VSL

FLOATING CONCRETE STRUCTURES

EXAMPLES FROM PRACTICE

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Preface

Two-thirds of the world's surface is covered with water. It is, therefore, not surprising that there has been much activity with concrete in the sea in recent decades. Numerous ideas and schemes have been promoted and many have been executed. Following the acceptance of concrete by the oil companies and the success of very large structures in the North Sea, the use of floating concrete structures, i.e. mainly temporarily floating structures, is continually growing.
The advantages of floating concrete structures lie in the economy of the materials used (concrete is very well suited to a marine environment), in the fact that it is easy to make concrete structures buoyant in the construction stage as well as permanently and for towing, whereas they are or can be made heavy enough for a safe permanent installation, and in the fact that they can also provide storage space.
Although concrete has been extensively used since 1900 for marine and coastal structures, post-tensioning has had a major, if not decisive, role in extending its full exploitation in marine structures. Post-tensioning creates a favourable state of stress.
Post-tensioned concrete is resistant to corrosion since the prestressing keeps the concrete in compression, thus limiting crack width. The material is highly resistant to fire, and post-tensioning improves watertightness and makes the structures well-suited forwithstanding the heavy wave and ice loadings encountered (for example in the North Sea and the Arctic). Finally, it does not suffer from the low-temperature fracture problems inherent in steel struc-tures.
Large structures can be built of cast-in-place concrete or may be assembled from precast components integrated by cast-in-place joints or by match-cast joints and by post-tensioning. A combined application of precast and cast-in-place elements is, of course, also possible. Precasting allows thin sections of high-strength concrete to be obtained. Post-tensioning can be provided in any desired direction to resist stresses from heavy and complex loads. Cyclic loading does not lead to fatigue failure. Cracks from overloading, for example possibly occurring during the delivery voyage, will close again after the loads are removed as a result of the post-tensioning.
The present report has been prepared with the objective of promoting floating concrete structures by illustrating their advantages, by providing a summary of the various possible applications and by explaining which VSL Special Construction Systems can be employed. Explanations of when, where and how these systems may be used are also given. The design of floating concrete structures will, however, not be treated here; design criteria for the determination of loads and forces can be found in the Literature (see bibliography).

The VSL Organizations will be pleased to assist and advise you on questions relating to the construction of floating concrete structures. These organizations hope that the present report will be helpful to you by stimulating new ideas, providing pointers and offering possible solutions. The VSL Representative in your country or VSL INTERNATIONAL LTD., Berne, Switzerland, will be glad to provide you with further information on the subject of floating concrete structures or the VSL Special Construction Systems.

1. Introduction

The history of concrete sea structures goes back to the Romans, who used pozzolanic cement concrete for the underwater piers of their river bridges. Some of these are still standing. In 1848 Lambot first used reinforced concrete for small boats; one of his later boats is still afloat. As mentioned in the preface, many marine structures have been built of concrete throughout the world since the early 1900's. In World Wars I and II, many hundreds of reinforced concrete ships were built, but their designs proved to be uneconomical. In the late 1950's, a number of prestressed concrete oceangoing barges were constructed in the Philippines. Concrete lighthouses were constructed as caissons in the 1960's; some of them were fixed in the sea bed by ground anchors. In the 1970's construction of offshore platforms for the exploration of oil started and by the end of 1986 eighteen concrete platforms have been installed in the North Sea.
Floating concrete structures are economical to build and maintain. To keep maintenance costs low, quality assurance during construction is very important. A quality assurance programme is, therefore, normally set up. Furthermore, tolerances must be kept small; it is important to limit closely the variation of the unit weights of the materials used and to observe rigorously the specified dimensions. A combination of maximum weight of components and maximum thickness tolerance must be avoided at all costs.
For cast-in-place structures, concrete with a minimum 28-day cube strength of 40 N/mm^2 is used, whilst in precast structures a strength of 50 to 60 N/mm^2 is the usual objective. The water/cement ratio should be low and good curing is of importance. The ratio can be minimized by using super

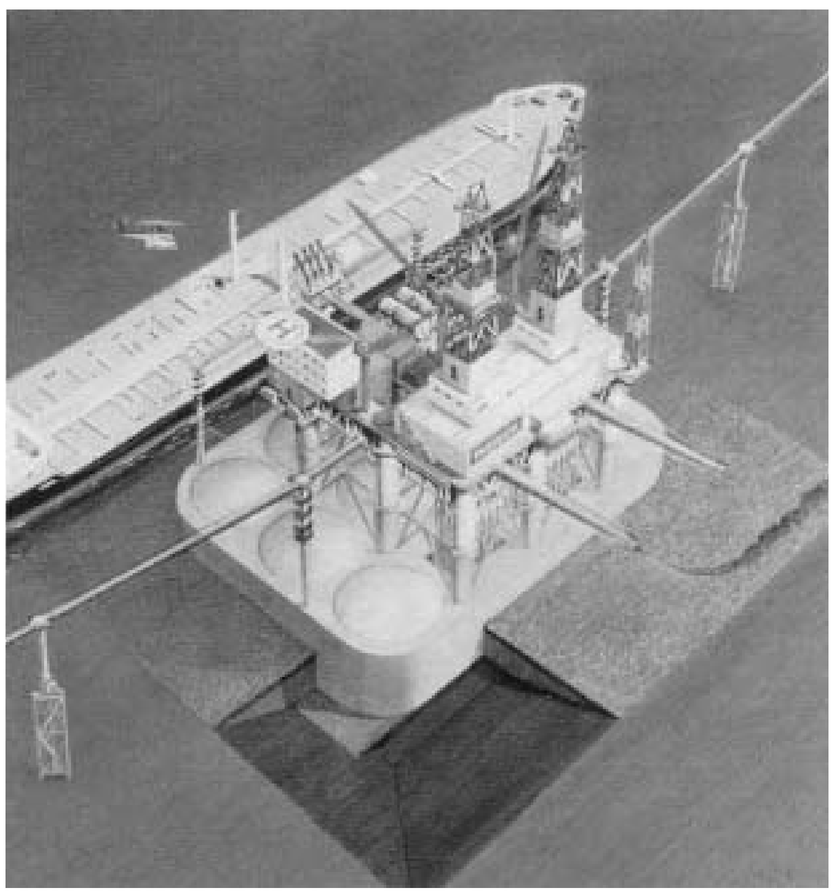

Figure 1 : Offshore oil production platform

plasticizers to make the fresh concrete workable. Lightweight concrete is attractive because it permits better buoyancy. It should be noted that the concrete strength is slightly reduced by the saturation of the material with sea water.

Concrete structures may be constructed in a convenient, protected area and then floated to the installation site. This method is used with advantage to avoid land reclamation, a costly and slow procedure, in which time must be allowed for consolidation of the fill, or to avoid the occupation of expensive, existing land. Even if the site is highly exposed to the weather, the structure can be quickly positioned in a short period of favourable conditions.

The range of applications of floating concrete structures that can be imagined is fairly large. It may include (Figs. 1 to 11):

- Oil exploration and drilling platforms,
- Oil production platforms,
- LPG terminals,
- Barges, ships and yachts,
- Floating docks,
- Floating gates for dry docks,

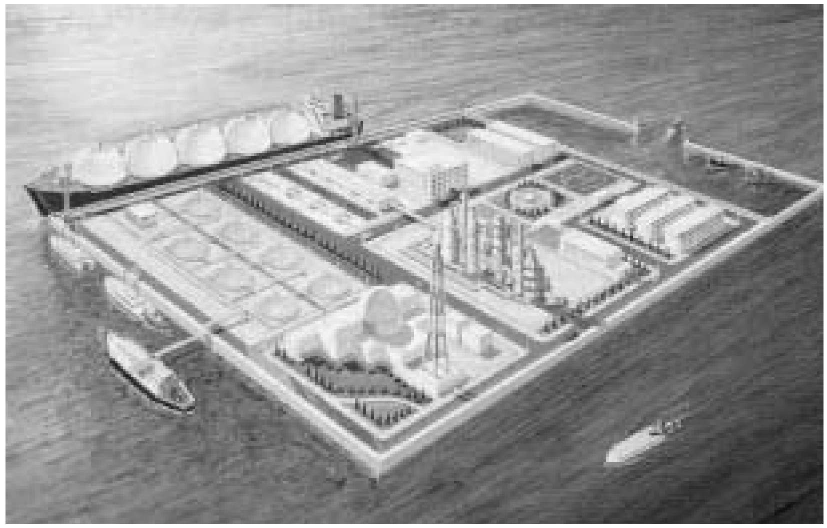

Figure 2: Floating LPG terminal

Figure 3: Floating concrete barge

Figure 4: Floating dock

- Floating airports,
- Floating power stations,
- Ocean thermal energy convertor (OTEC) plants,
- Rotating mooring structures,
- Floating hotels
- Floating shopping centres,
- Floating industrial plants,
- Floating jetties,
- Floating bridges,
- Floating bridges piers,
- Semi-submersible tunnels,
- Floating lighthouses,
- Floating breakwaters,
- Floating bridge girders,
- Semi-submersible towns, etc.

Floating concrete structures are the subject treated by a commission of the Federation Internationale de la Precontrainte (FIP) with the objective of preparing recommendations for the design and construction of such structures (see bibliography). This is a convenient place to summarize again the advantages of floating concrete structures:

- Durability (including high resistance to abrasion) and low maintenance,
- Excellent fatigue resistance,
- High resistance to compressive forces
- Excellent behaviour in cold weather and at low temperatures,
- Inherent rigidity,
- Good thermal insulating properties,
- High fire resistance,
- Utilization of mainly local materials,
- Economy.

Figure 5: Floating airport

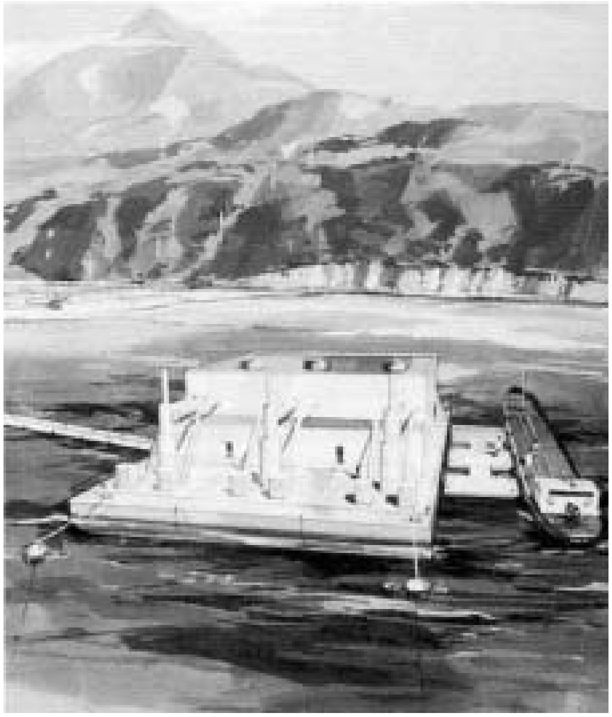

Figure 6: Floating power station

Figure 7: Ocean thermal energy convertor (OTEC) plant

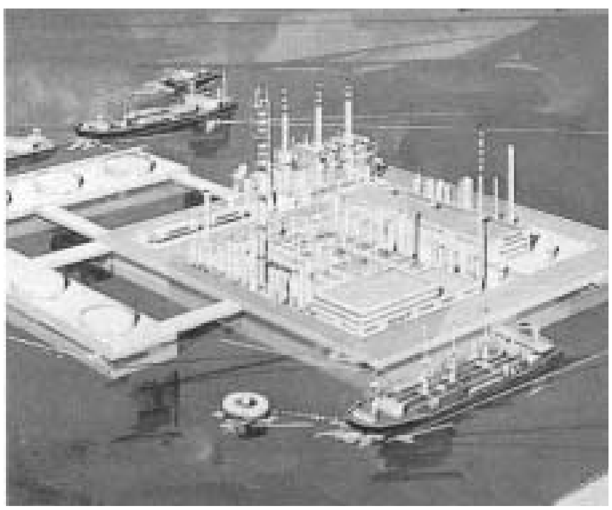

Figure 8: Floating industrial plant

Figure 9: Floating Brigde

Figure 10: Floating breakwater

Figure 11: Sea town of floating islands

2. VSL Systems and Service Range

2.1. VSL Systems

2.1.1. Introduction
In building floating concrete structures, the following VSL Special Construction Systems are of particular importance:
- VSL Post-tensioning,
- VSL Slipforming,
- VSL Heavy Rigging.

In many cases, these systems will be used separately, but it is especially advantageous if VSL is chosen for all the systems. By making use of the various VSL Systems in combination and by taking account of these facilities at an early stage in planning, the client will obtain substantial advantages. These may, for example, include the following:
- Optimized preparatory work, progress and use of personnel and materials,
- Simultaneous execution of post-tensioning and slipforming operations under coordinated control.

2.1.2. VSL Post-tensioning
The VSL Post-tensioning System (see pamphlet 'VSL Post-Tensioning Systems') with its wide variety of types of anchorage and cable units, is ideally suited for use in building floating concrete structures (Fig. 12). The methods adopted for assembling the tendons are also of particular advantage, since they can be adapted to the respective circumstances encountered.
The VSL Post-tensioning System uses, as tension elements, 7-wire strands of 13 mm (0.5") or 15 mm (0.6") nominal diameter,

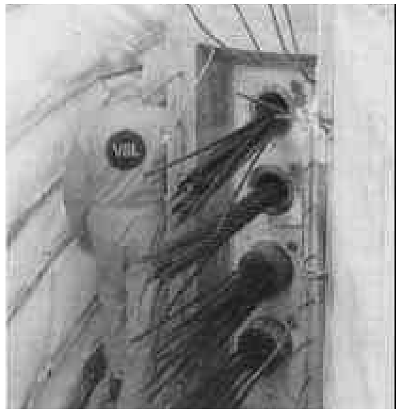

Figure 12: Application of the VSL Posttensioning System

with ultimate tensile strengths of 1,670 to 1,860 N/mm^2. In addition to the high strength and low relaxation, the great ease with which strands can be grouted (due to the screw action obtained from the twisted surface of the strands) should be emphasized. The strands of a VSL cable are stressed simultaneously, but individually locked in the anchor head of the stressing anchorage or coupler. Stressing can be carried out in as many steps as desired.
The VSL Post-tensioning System is approved in every country where the use of post-tensioning systems is subject to an official authorization. It also complies in all respects with the 'Recommendations for the Acceptance and Application of Post-tensioning Systems' of theFIP.

The VSL Post-tensioning System provides stressing anchorages types EC (Fig. 13) and E (Fig.14) and special centre stressing anchorages types Z (Fig.15) and ZU (Fig.16), which make buttresses unneces-sary as these anchorages can be stressed in a block-out. Of the dead-end anchorages, in addition to the standard types H (Fig. 17) and U (Fig. 18), special mention should be made of type L (Fig. 19), in which the tendon is returned through 180° within a small space. This type is especially suitable for vertical post-tensioning, since it enables the strands to be installed after concreting, which is a constructional advantage.
Horizontal tendons also are usually installed after concreting. The tendons may be completely prefabricated, but are nevertheless easy to install due to their flexibility. The VSL Push-through Method (Fig.20) is, however, most commonly used; if the tendons are installed by this method before concreting, even dead-end anchorages type H can be used, as these can be formed just after installation of the strands of a cable. The VSL Push-through Method consists of pulling the strand from a dispenser containing the strand coil and pushing it by means of a special device directly into the duct. When the strand has reached the necessary length, it is cut off and the pro-cedure is repeated until all the strands of the tendon have been placed in the duct.
In floating concrete structures corrosion protection of the post-tensioning cables and their anchorages is of special importance. Ducts should therefore be tight (for example according to the tentative German standard DIN 18553) and galvanized, while the anchorages may be recessed and epoxy-coated. The cement mortar used for grout

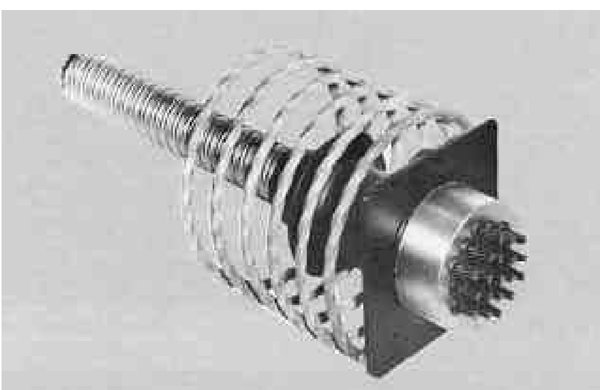

Figure 13: Stressing anchorage VSL type EC

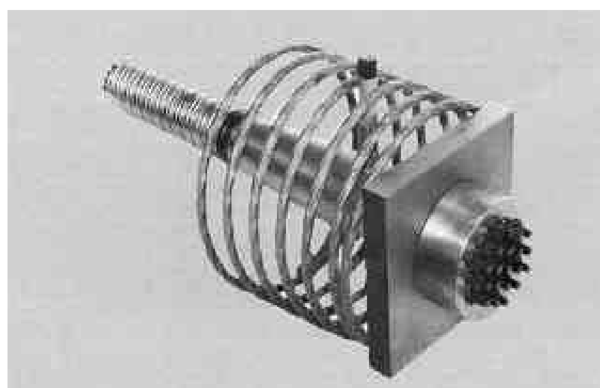

Figure 14: Stressing anchorage VSL type E

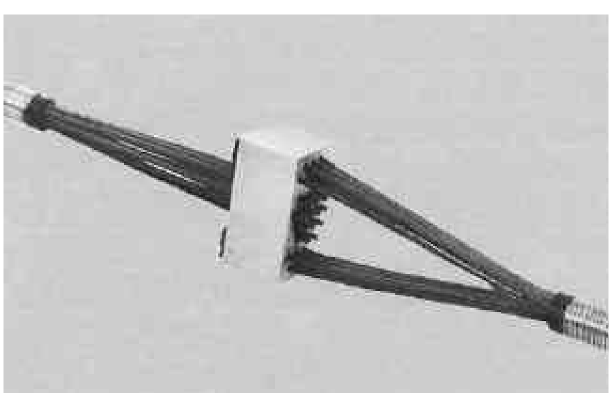

Figure 15: Centre stressing anchorage VSL type Z

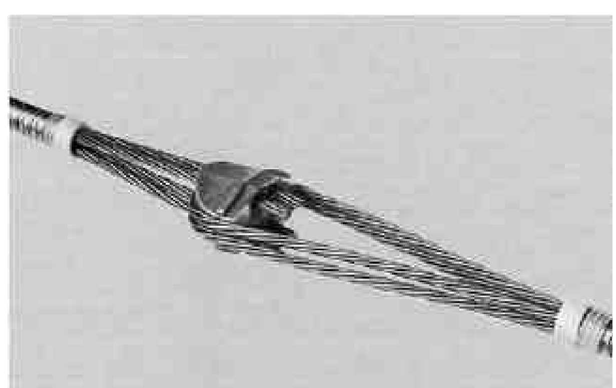

Figure 16: Centre stressing anchorage VSL type ZU

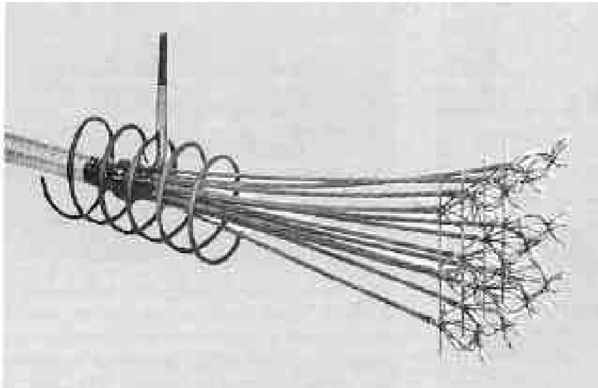

Figure 17: Dead-end anchorage VSL type H

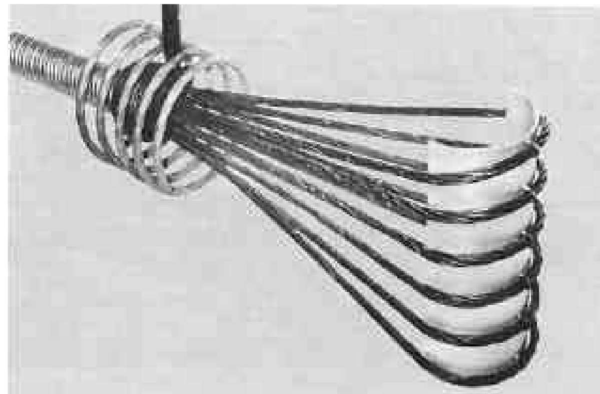

Figure 17: Dead-end anchorage VSL type U

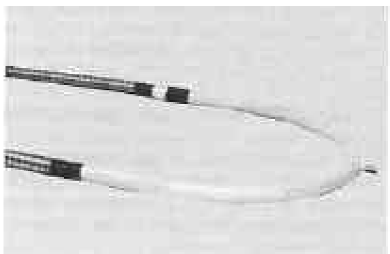

Figure 19: Anchorage VSL type L

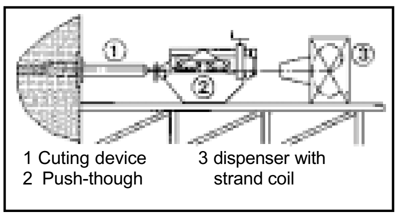

Figure 20: Diagrammatic representation of the VSL push-through equipment

Figure 21: Application of VSL Slipforming

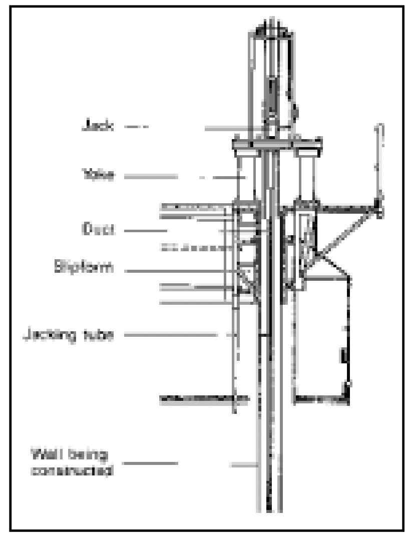

Figure 22: Basic construction of VSL Slipforming

ing constitutes a chemically active corrosion protection.
In floating concrete structures VSL tendons can be used for:
- Horizontal post-tensioning of foundation slabs, bottom slabs and walls,
- Vertical post-tensioning of walls,
- Post-tensioning of inclined elements, Fixing of steel parts to the concrete,
- Jointing of hinge systems, etc.

2.1.3. VSL Slipforming

Shafts, tanks and walls of floating concrete structures are especially well-suited to the use of slipforming during construction, since the preconditions for economic use of this construction method exist to a particularly high degree in these structures (Fig. 21).
The advantages of slipforming include the short construction time resulting from continuous working, monolithic construction without construction joints and of high dimensional accuracy and cost savings even where the height is moderate.
The slipforms of the VSL Slipforming consist of 1.25 m high elements of steel. They are standardized components, from which any desired plan form can be made up. Steel was chosen as the formwork material because it guarantees the highest dimensional accuracy in construction. The inner and outer forms are connected together by transverse yokes. At the upper edge of the forms, working platforms are located and scaffolds for finishing the concrete surface are suspended beneath them.
The forms are raised by hydraulic jacks of 30 or 60 KN lifting force moving on jacking tubes. The jacking tubes are positioned inside the wall under construction and transfer the load from the formwork equipment to the foundation. In the wet concrete zone the jacking tubes are encased in ducts which are connected to the slipform. These ducts provide antibuckling guidance to the tubes and prevent them from being concreted in, so that they can be recovered and used again (Fig. 22).
VSL Sipforming can also be used with special forms, by which conical walls and walls of variable thickness or special shapes can be produced. The speed of progress depends upon many factors, such as size of structure, dimensions, reinforcement, concrete quality, temperature, etc. The rate normally varies from 2 to 6 m per 24 hours.
Slipforming is a largely mechanized construction procedure. A trouble-free and therefore economic sequence of work requires certain preconditions in respect of design, organization and construction. Cooperation should therefore be established as early as possible between the project designer, main contractor and slipforming contractor. This will then guarantee rational and co-ordinated construction.
The simultaneous use of VSL Post-tensioning and VSL Slipforming is especially advantageous because:
- the placing of the cable fixings and ducts during the slipforming operations is car ried out simultaneously with the fixing of reinforcement and can be continually monitored by the VSL Slipforming personnel,
- the formwork panels to which the anchorages of the VSL cables are fixed can be re-used during slipforming,
- the preparatory work, progress and the use of personnel and materials are all under one control, which considerably simplifies co-ordination.

Information about the use of VSL Slipforming for floating concrete structures will be found in the next chapter. Attention is also drawn to the publication 'VSL Slipforming', which contains further details and examples of use.

2.1.4. VSL Heavy Lifting

In the construction of floating structures, whether of concrete or steel, large or heavy components such as berth elements, platform decks and modules, platform elements, etc. may have to be lifted, lowered or horizontally jacked due to their weight or size (Fig. 23). For such work, VSL can offer two heavy lifting systems:
- VSL Strand System,
- VSL Rod System.

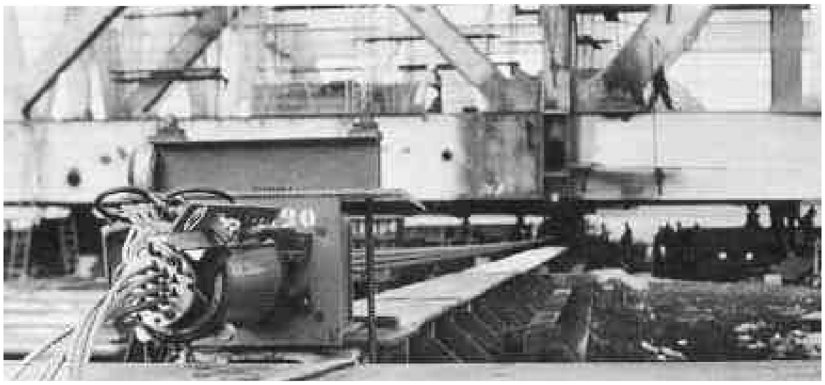

Figure 23: Application of VSL Heavy Lifting for an offshore structure

The feature common to both systems is the principle of their operation. The load is moved by electro-hydraulic jacks with a regripping mechanism, which enables the load to be moved rapidly, smoothly and accurately over almost any distance. The motive units of both systems are characterized by their comparatively small dimensions in relation to the force developed. They can be arranged in groups to distribute the load over a number of gripping points.
The VSL Strand Lifting System (see also brochure 'VSL Heavy Lifting') has been developed from the VSL Post-tensioning System. Its essential components are the motive unit, the strand bundle and the anchorage at the load to be moved. The motive unit consists of a VSL centre-hole jack and of an upper and a lower strand anchorage (Fig. 24).
For the load-bearing element, 7-wire prestressing steel strands Ø 15 mm (0.6") are normally used. Advantages of strands over other load-bearing elements are that their specific carrying capacity is particularly high and that they can be cut to any required length. The number of strands per cable is adapted to the load to be moved, so that within the scope of the six existing basic VSL motive units any force between 104 and 5,738 KN is possible (the safety factor with respect to U.T.S. of strands is 2.50). The motive units are driven by high-pressure pumps, usually operated from a central control console. The speeds that can be achieved depend upon the delivery of the pumps. Lifting speeds of up to 10 m/h and horizontal jacking speeds of up to 20 m/h can be attained. In exceptional cases the speed may be even higher. In the normal case, however, it is not the speed but the reliable execution of the rigging operation that is of prime importance.
The VSL Rod Rigging System uses tubular rods, equipped at regular intervals with cogs, as the load-carrying elements (Fig. 25). Due to their stiffness they are especially well-suited for pushing. The two motive units available have capacities of 300 and 800 KN respectively.
In addition to these two heavy lifting systems, VSL can also provide various types of jacks, for example for small lifting or levelling operations.

2.9.5. Other VSL Systems

There may be occasion to use other VSL Systems in the construction of floating con-crete structures. These include:
- VSL Soil and Rock Anchors,
- VSL Measuring Technique,
- VSL Fabric Formwork,
- VSL Flat Jacks.

VSL Soil and Rock Anchors may find appli-cations, for example, for anchoring sheet pile walls of a temporary dry dock used for constructing a floating concrete structure, or for anchoring structures such as lighthouses, for example (see chapter 3.2.6.), in the ground.
Specially engineered and tailored VSL Fabric Formwork is extensively used in the offshore industry for grouted supports,

Figure 26: Application of VSL Fabric Formwork

protections and stabilization of pipelines and other subsea installations (Fig. 26).
Other typical uses for VSL Fabric Formwork are the sealing of grouting compartments on the underbase of concrete gravity structures and grouted flexible mattresses, used for scour prevention around all types of under-water structures subjected to scour due to strong currents and/or soft soil conditions.
These other systems may be used in particular cases. Information about the systems will be found in the relevant brochures. In this context, reference may again be made to those VSL publications which are of interest with regard to the construction of floating structures:
- Brochure 'VSL Post-tensioning Systems'
- Brochure 'VSL Slipforming'
- Brochure 'VSL Heavy Lifting'
- Brochure 'VSL Soil and Rock Anchors' Brochure 'VSL Measuring Technique'
- Brochure 'VSL Flat Jacks'
- Technical Report 'Soil and Rock Anchors - Examples from Practice'
- Technical Report 'Concrete Storage Structures'
- Job Reports
- VSL News

2.2. Services offered by VSL

It will be apparent from the preceding chapters that the VSL Organizations can offer a very comprehensive range of services in the construction of floating concrete structures, namely:
- Consultancy service to owners, engineers and contractors,
- The carrying out of preliminary design studies,
- Assistance with the preliminary design of floating concrete structures,
- Preparation of alternative designs in cluding cost comparison,
- The development of complete projects,
- Detailed design of post-tensioning,
- Carrying out of post-tensioning work,
- The design and manufacture of slip-forms,
- The execution of Slipforming work,
- The design of rigging operations,
- Carrying out of rigging operations,
- The use of other VSL Systems.

The VSL Organizations* are in a position to provide these services on advantageous terms; for each case the possibilities and extent of services will usually need to be clarified in discussions between the owner, the engineer, the contractor and the VSL Organization.

*The addresses of the VSL Representatives will be found on the back cover of this report.

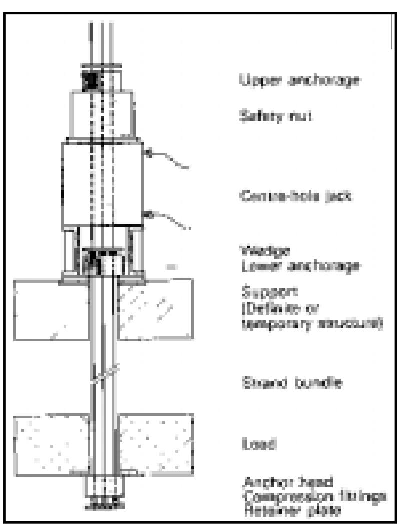

Figure 24: Basic construction of the VSL Strand Lifting System

Figure 25: Motive unit of the VSL Rod Lifting System

3.Examples from Practice

3.1. Introduction

In the following you will find a series of descriptions of floating concrete structures, for which VSL Systems were used. The chapter is divided into two parts. The first part presents structures which floated during construction and tow-out but were finally placed on the sea bed or on a firm structure. while the second part is devoted to permanently floating concrete structures. These structures were all built either in Europe or the Far East, with one exception. Indeed, the discovery of oil and gas deposits in the North Sea has created a boom in the construction of floating concrete structures. In the Far East, also, the use of concrete is common and is known to be economical. Floating concrete structures will certainly find increasing applications in other parts of the world as well. At this point it should be mentioned that the VSL Heavy Lifting Systems, in particular, have also been used on a number of floating steel structures. These structures are, however, not presented in this report as it is devoted exclusively to concrete structures.

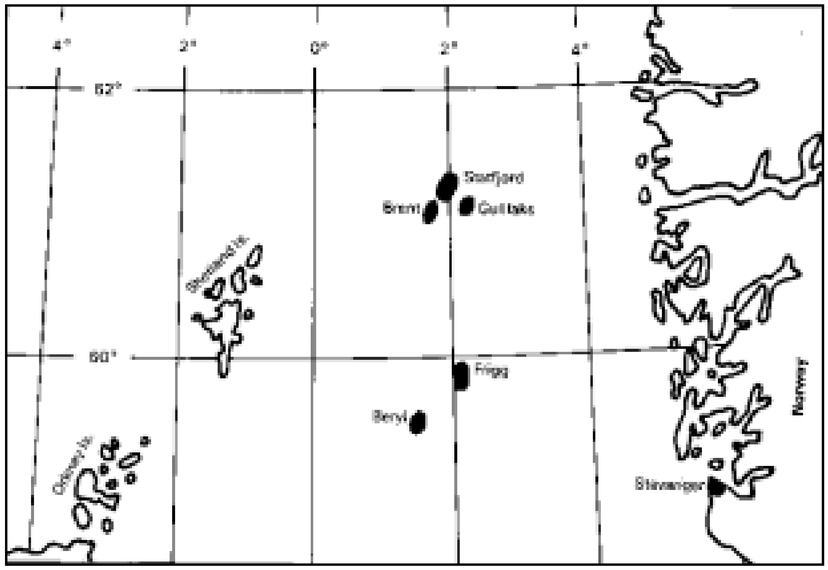

Figure 27: Location of Condeep platforms in the North Sea

3.2. Temporarily floating structures

3.2.1. 'Condeep' offshore oil production platforms

Owner (Operator) Mobil Exploration, Oslo, Norway (Platforms 'Statfjord A', 'Statfjord B', Statfjord C')
Statoil, Oslo, Norway (Platforms 'Gullfaks A', 'Gullfaks C')
Shell U.K. Exploration and Production Ltd., London, UK (Platform 'Brent D')

Engineers and Contractors,
Norwegian Contractors
Oslo, Norway

Post-tensioning
VSL Norge A/ S (formerly VSL Offshore Ltd.), Stavanger, Norway

Years of construction
1974-1989

Introduction

In the mid-sixties, large oil deposits were discovered in the North Sea, mainly in the British and Norwegian sectors. The oil companies showed enormous interest in these fields, in spite of the fact that the exploitation techniques that would be required were comparatively unknown, since previously oil had been extracted only on land and inshore in shallow water, but not in water 150 m and more deep. Furthermore, some of the most promising oil fields are situated quite far north, some 150 to 250 kilometers offshore and exposed to storm waves of up to 30 m and gale force winds of 240 to 260 km/h.

Such geographic and climatic conditions demand that site installation work be kept to a minimum and storage capacity be created, since production must continue even when tankers cannot load and where there is no pipeline to the shore. These requirements led to the development of completely new production platforms.

Concrete gravity structures with steel decks appeared to be very well suited to these conditions. Designs for different types of

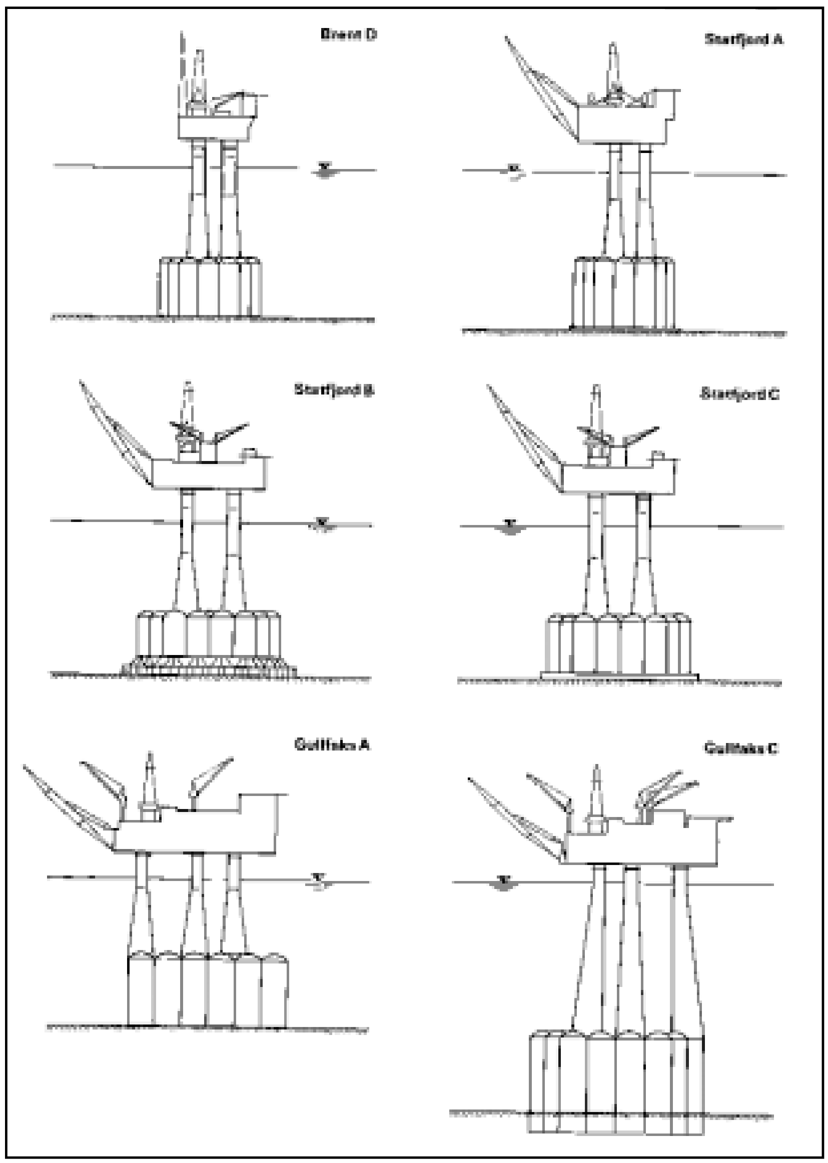

Figure 28: Sections of various Condeep platforms

platforms were rapidly prepared and submitted to theinterested oil companies. Most designs are similar in overall size and have basically the same concept. They comprise a cellular concrete caisson, acting as foundation and providing oil storage capacity, but also providing buoyancy during construction and tow-out. The foundations support one or more concrete towers, with a steel deck carrying the process plant and equipment spanning between them. The platform is used as a drilling platform, with derricks, storage for mud, cement and pipes, power generation and living quarters, and as a production unit providing cooling and separation facilities and storage for the crude oil, together with generators, pumps and re-injection equipment, living quarters, rescue and fire-fighting plant, communications, gas flare off and so on.

The Condeep design

The most successful type of gravity structure so far used is that known as the 'Condeep' design (concrete deepwater); this was originally developed by A/S Hoyer-Ellefsen. Further development and construction of the first platforms was made in the Condeep Group with Norwegian Contractors (Hoyer-Ellefsen, F. Selmer and T. FurUholmen) for civil works and the Aker Group for mechanical works. The design is based upon model tests performed in specialized laboratories and universities, to assess the effects of the natural forces encountered in the North Sea. With the advanced state of mathematics and the science of the strength of materials available, the very complex calculations were made in a short time, using extensive computer capacity (Fig. 27).

A Condeep platform structure essentially consists of three parts: the storage cells, the towers and the deck structure. The cells and towers are of reinforced and post-tensioned concrete, while the deck is a steel structure. The first Condeep platforms built had 16 cylindrical storage cells and three towers. This design was subsequently modified, to meet the clients' requirements for larger structures. The next type had 20 storage tanks and four towers to carry the deck. Not all the structures built to this design were identical, since the foundation as well as the tower arrangements varied. Recently, construction of a new type with 26 cells and four towers has started. The latest platform, 'Gullfaks C', now under construction, is of extraordinary size; its total height will be approx. 380 m (Fig. 28).

Construction procedure

Once a deposit has been discovered and assessed as worth exploring, the order for constructing a platform can be placed. It is most important for construction then to start quickly, to enable exploitation to commence as soon as possible and thereby obtain an early return on the huge investment.

All Condeep platforms have been or are being built by Norwegian Contractors (NC), who have a construction yard in Stavanger on the west coast of Norway. This is a most favourable region for offshore construction, which demands deep, sheltered water close inshore near to a site. The Norwegian fjords perfectly fulfil these requirements. Commercial as well as geographic reasons, however, determined the choice of the Stavanger site.

The site at Stavanger offers all the facilities required. The construction yard comprises two dry docks, which were created by driving sheet piles into the sea bed, dewatering and excavating the rock to a certain depth. In this way two sites with a total area of about 75,000 m^2 were obtained.

The general construction procedure for a Condeep structure is as follows (Fig. 29). First the steel or concrete skirts are placed or constructed in the construction yard. Then the foundation slab, lower domes and cell walls are built (Fig. 30). The walls are slipformed at a rate of 1.5 to 2.0 m per day. As soon as the structure has reached a stage where it will float, if necessary with the help of an air cushion inside the skirts, construction of the cell walls is interrupted. The dry dock is then flooded and the piling removed. The structure is now brought into deeper water and there construction of the walls is continued (Fig. 31). After the cells have been closed by their upper domes, slipforming of the tower shafts is started (Fig. 32). slipforming is exacting work, as diameter and wall thickness vary and reinforcement, post-tensioning cables and concrete must be placed in simultaneous working. When the concrete structure is complete, it is towed out to deeper but still sheltered water (Fig. 33), where deck mating is carried out (Fig. 34).

Post-tensioning

The Condeep platforms contain a considerable quantity of post-tensioning, which is required mainly for structural reasons. Depending upon the type of platform, post-tensioning may be used in the following parts of a structure (Fig. 35):

- Skirt walls,
- Bottom slab,
- Retaining and plinth walls rising from the bottom slab,
- Inclined struts between bottom slab and cell walls (Fig. 36),
- Ring beams of lower domes,
- Drill domes,
- Moorings,
- Lower and upper parts of cell walls,
- Ring beams of upper domes,
- Area of upper domes around shafts,
- Walls of shafts,
- Ring beams at the top of the shafts.

The cable units used in the first platforms ('Statfjord A' and 'Brent D') consisted mainly of the VSL type 5-19 (breaking load 3,500 KN), which replaced the 5-12 tendons originally specified at the tender stage. The adoption of the larger unit made the prestressing work more economical. With a few exceptions, the tendons were generally made up by the VSL Push-through Method.

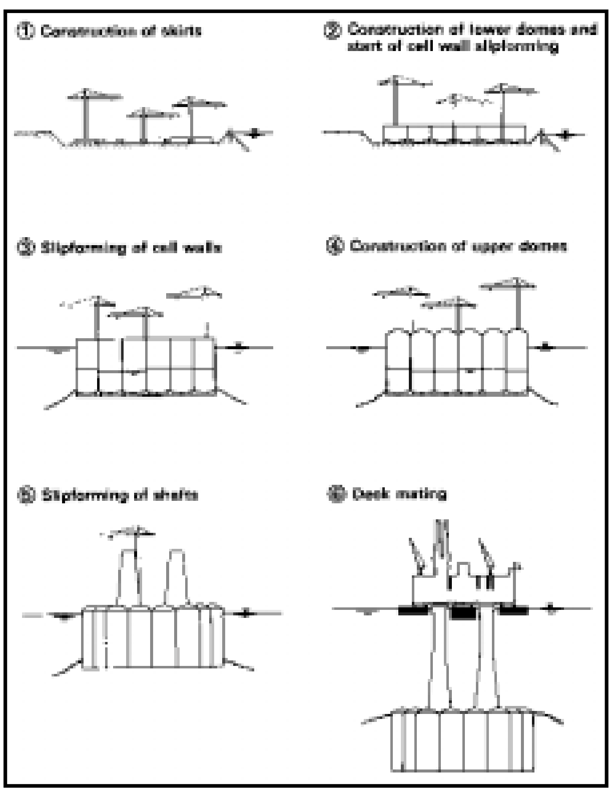

Figure 29: Typical construction procedure for a Condeep platform

Figure 30: Condeep platform under construction in the dry dock

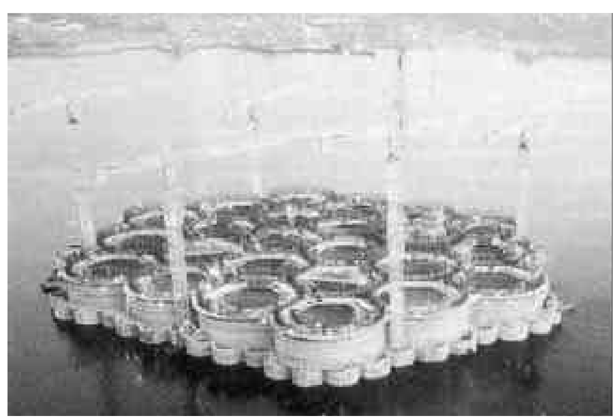

Figure 31: Condeep platform during construction of cell walls

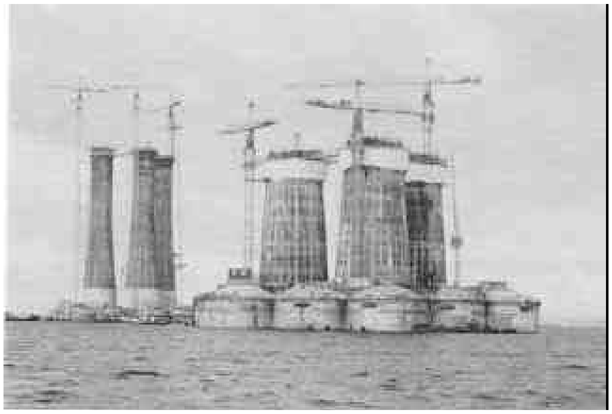

Figure 32: Con deep platforms during construction of tower shafts

Figure 33: Tow-out of the concrete strucuture of a Condeep platform

Figure 34: Deck mating, i.e. of deck structure onto the concrete structure

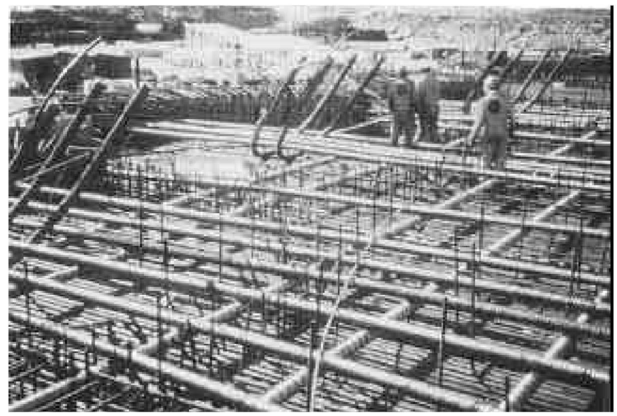

Figure 36: VSL cables in bottom slab and loop ducts for cables in struts

The major part of the post-tensioning is installed in the shafts. The stressing anchorages used are of the EC-type, while the dead-end anchorages are mainly of the H-type. The latter was developed especially for the first Condeep jobs in which VSL was involved, since it perfectly suits the pushthrough method; the strands can be installed by push-through and the H-anchorage can be formed immediately afterwards. The shaft cables were prefabricated in the workshop, using the push-through method. In the first two platforms the H-anchorages were formed in the workshop, while in the others they were made up after installation of the cables. Both the anchorage of type H and the anchorage body of type EC had been subjected to rigorous system tests, to meet the approval required by the Norwegian administration for the Condeep jobs.

From the 'Statfjord B' platform onwards, the VSL cables used were mainly of the 6-19 unit (breaking load 4,986 KN). Low-relaxation strand was used throughout. All the cables were cement-grouted after stressing (Fig. 37). Grouting of the long vertical tendons required particular attention and therefore full-scale tests with different grout mixtures and additives were performed in advance. The most convenient mixture was found to be a cement grout with a water / cement ratio of 0.45, containing a thixotropic additive. With this grout and the use of a special colloidal grout mixer, it was possible to grout all the vertical tendons in

Figure 37: Stressing of a VSL cable

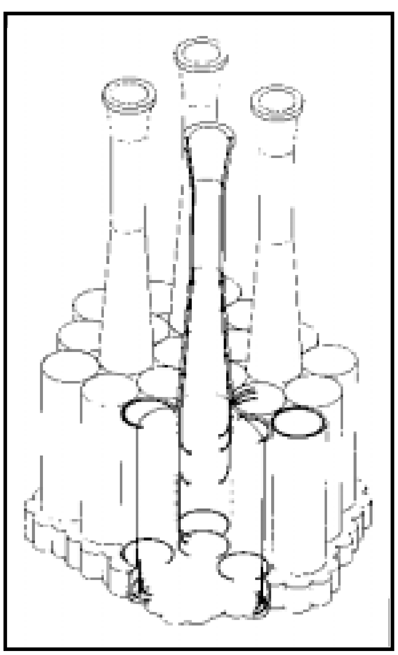

Figure 35: Cable layout scheme

Name of structure	Base area (m²)	Number of cells	Number of towers	Total height (m)	Water depth (m)	Storage capacity (barrels)	Years of construction	VSL services provided
Brent D	6,400	19	3	171.3	140	1 million	1974–76	VSL Post-tensioning: 1,400 tonnes of p.t. steel Deck fixing by means of p.t. cables
Statfjord A	7,900	19	3	176	145	1.5 million	1974–79	VSL Post-tensioning: 1,300 tonnes of p.t. steel Deck fixing by means of p.t. cables Balancing of load between deck and utility shaft
Beryl A	6,300	19	3	149	120	0.9 million	1976	Levelling of deck by means of VSL jacks
Statfjord B	18,200	24	4	174	145	2 million	1978–81	VSL Post-tensioning: 2,800 tonnes of p.t. steel Lowering of dowels Deck fixing by means of p.t. cables
Statfjord C	13,000	24	4	174.4	145	2 million	1981–84	VSL Post-tensioning: 1,737 tonnes of p.t. steel
Gullfaks A	11,000	24	4	158.6	133	1.95 million	1983–85	VSL Post-tensioning: 1,265 tonnes of p.t. steel
Gullfaks C	16,000	24	4	262.4	215	2 million	1985-89	VSL Post-tensioning: 3,500 tonnes of p.t. steel Deck fixing by means of p.t. cables

Table I : Main of Condeep platforms in the construction of witch VSL was involved

one or two steps (according to length) and without bleeding occurring

Friction and elongation tests

In the stressing of the tendons (up to 170 m long) of the foundation slab of the Statfjord B' platform, elongations larger than the calculated values and exceeding the tolerances permitted by the standards were systematically observed. This naturally raised the question of the possible reasons for these results, particularly as the same trend (although within the 5% tolerance band) had also been noticed in the shorter tendons.

The participating firms concerned (Norwegian Contractors, the structural engineering office Dr. O.Olsen, the technical control company Noteby A.S. and VSL Offshore Ltd.) decided to carry out a comprehensive test programme to investigate the reasons for the excessive elongations. The tests were performed on the Stavanger site within a year starting from the end of August 1979 (Fig.38). Further investigations continued until December 1980 in the 'Physikalisch-technische Bundesanstalt', in Brunswick, Federal Republic of Germany.

A considerable time was spent in preparing a special computer programme for evaluating the approx. 40,000 readings. The following conclusions could be obtained from the evaluation:

1. In normal conditions, i.e, with tendons of sufficiently large curvature which have been correctly installed, with no corrosion of the duct and strand, the commonly used v values of the friction coefficient are correct.
2. The friction coefficient increases with decreasing radius of curvature. This effect is even more pronounced for materials with corroded surfaces.
3. Corrosion increases the friction coefficient much more than has usually been assumed to date. Therefore, latest possible installation of the strands before stressing and good protection of the ducts during construction is recommended.
4. For a small force, the friction coefficient is smaller than for a large force. A maximum friction coefficient therefore corresponds to a maximum value of the force. If the elongation is calculated with this latter value, on the assumption that it is constant, then the calculated elongation is smaller than the measured value.
5. Deviations of at least 5% must be expec ted between measured force and effective force under site conditions. Higher accuracy could only be achieved with disproportionate expenditure.
6. The procedure for testing the material properties of the prestressing steel should be improved.

Figure 38: Friction and elongation tests being carried out

The latest Condeep platform

The 'Gullfaks C' platform will be used, together with platforms 'A' and 'B', in the Gullfaks Field. This is the first field wholly in Norwegian ownership. 'Gullfaks C' will be the world's largest offshore production platform, with a final installed weight of approx. 1.4 million tonnes and total height of about 380 m. Tow-out is scheduled for summer 1989, five years after the go-ahead from the Norwegian Parliament. The cost of the complete platform will amount to some US $1,500 million.

'Gullfaks C' will be an integrated drilling, production and accommodation platform, with a gravity base structure comprising 20 storage cells, four shafts and a steel deck (Fig. 39). The production capacity will reach some 245,000 barrels or 33,000 tonnes of crude oil per day. Gas from 'Gullfaks C' will be transferred through a separate pipeline to 'Gullfaks A', to be taken ashore. Oil will be loaded into tankers via an offshore loading buoy. 'Gullfaks C' will be equipped to handle oil from the 'B' platform, if necessary.

'Gullfaks C' will be placed on the western slope of the Norwegian Trench, and not on the North Sea plateau like earlier Norwegian platforms. Sea bed conditions here are poorer than in shallow water, as there is an upper layer of about 40 m of relatively loose sand and clay. To prevent the platform from subsiding, its base is specially designed, with long concrete skirts designed to cut into the sea bed under the weight of the platform. To install a platform with skirts 27 m long, of which 22 m are intended to penetrate into the seabed, an entirely new installation technique is required. Firm seating on the seabed is obtained mainly by the platform's own weight and by the suction effect created by the difference in pressure inside and outside the skirts. To test this technique and to measure the stress on the base structure during installation, Statoi! conducted a test in the early Autumn of 1985. The model used consisted of two steel cylinders, 25 m high and connected together by a 0.40 m thick concrete wall. The test was carried out offshore in block 34/ 10, where 'Gullfaks C' will be positioned.

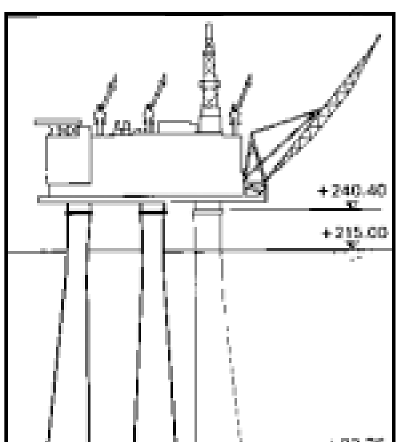

Figure 39: Cross-section of Gullfaks C platform

The skirts are located below the perimeter cells and below the two cells in the centre. They consist of 0.40 m thick walls, which are constructed by slipforming and posttensioned. The cells, more than 82 m high, have walls between 0.60 and 2.30 m thick. The latter value corresponds to the intersection of neighbouring cells. The distance between the axes of the cells is 28.00 m. The shafts are 162.80 m high and the total height of the concrete structure is, therefore, 262.40 m. The central shaft is vertical and the axes of the other three are inclined at about 3.5 ° towards the centre.

Construction of 'Gullfaks C' started at the end of 1985 with the building of the skirts (Fig. 40), Completion of the concrete structure is scheduled for Autumn 1988 and deck mating for February 1989.

Approx. 3,500 tonnes of post-tensioning strand will be required. The skirts are posttensioned by horizontal VSL cables EC/EC 6-6, 6-7 and 6-9. In the other parts of the structure, that is the lower domes, cell walls, upper domes and shafts, tendons of type VSL 6-9, 6-19 and 6-31 are used (Fig. 41). The post-tensioning of these parts is similar to that in previous Condeeps, except in the zone lower part of cell walls/ring beams of lower domes. There, additional vertical cables join the cell walls to the ring beams. These cables have a dead-end similar to the bond length of a rock anchor, located in the ring beams.

Condeep T300

The next generation of offshore platforms is already appearing on the horizon. The design for a platform intended for a water depth of 300 m - the 'Condeep T300' model (Fig. 42)-is complete. Although no contract has yet been let for building such a platform, Norwegian Contractors have already done preparatory work for its construction. The T300 model will have three inclined shafts of varying diameter. To make sure that these can be constructed without problems by the slipforming method, a 53 m high test shaft was built in March/ April 1984. All the work went ahead to the complete satisfaction of everybody concerned. The test piece has been named the 'Leaning tower of Stavanger'. It is now being used as a cement silo for the platforms presently under construction, including 'Gullfaks C'.

For the working condition, stability of the silo had to be assured by anchoring it at one side into the rock. For this purpose, 36 VSL Rock Anchors 6-19 were installed early in 1984. Each anchor was stressed to a test force of 3,550 KN and locked off at 3,300 KN (Fig.43).

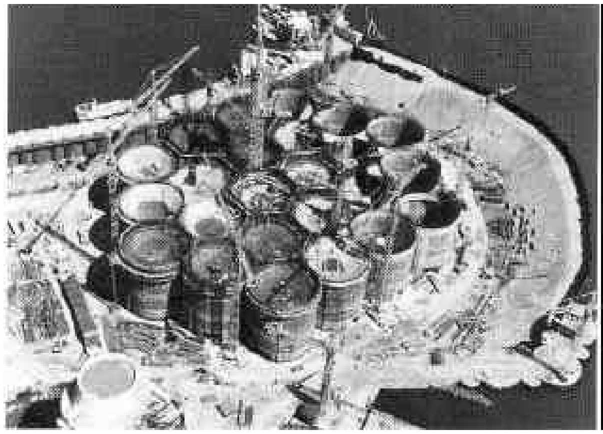

Figure 40: Gullfaks C platform under construction

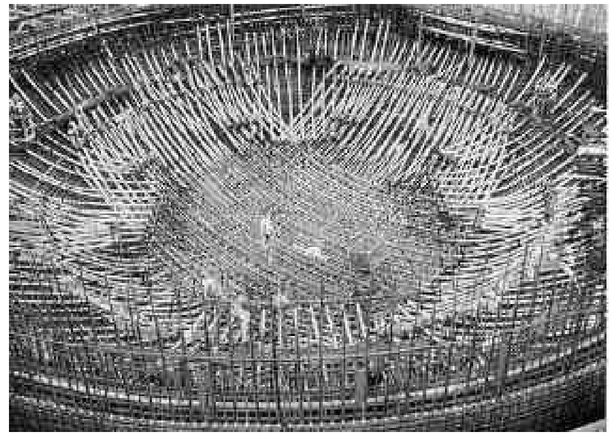

Figure 41: Cell base with VSL tendons

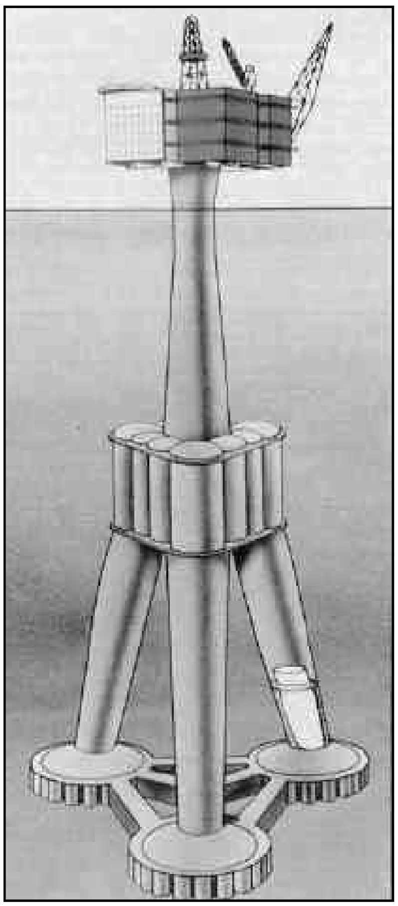

Figure 42: Artist's impression of Condeep T 300

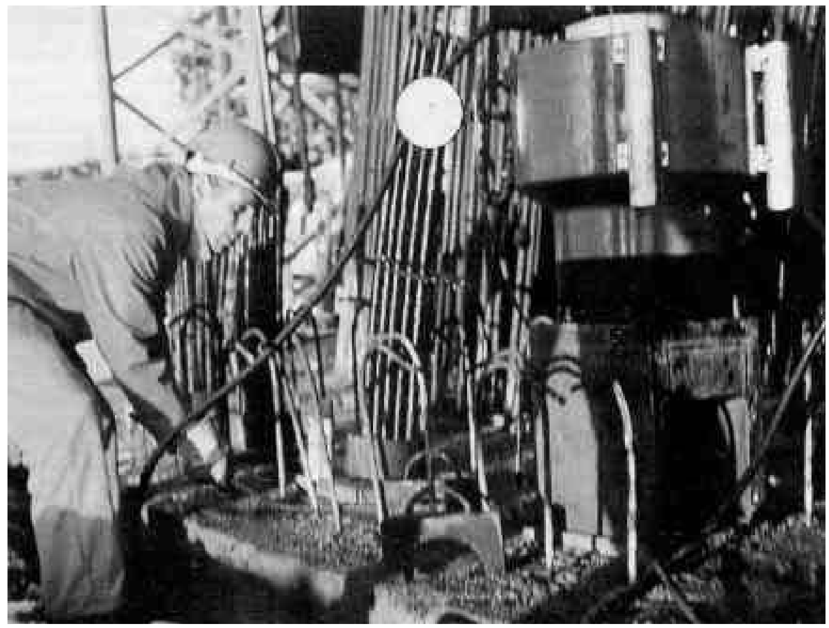

Figure 43: VSL Anchors to secure the 'Leaning tower of Stavanger' being stressed

3.2.2. Offshore oil production platform for the Dunlin Field

Owner Shell U.K. Exploration and Production Ltd., London, UK

Engineer ANDOC (Anglo-Dutch Offshore and Concrete), a joint venture of two English and four Dutch construction groups

Slipforming
Joint Venture Gleitbauge sellschaft m.b.H., Salzburg, Austria/ mleitschnellbau G.m.b.H., Dusseldorf, Federal Republic of Germany/ IGA AN & Co., GmbH, Cologne, Federal Republic of Germany/VSL INTERNATIONAL LTD., Berne,Switzerland

Years of construction 1974-1976

The ANDOC platform consists of a rectangular base, 104 x 104 x 32 m, composed of 81 cells and of four towers which reach a height of 144 m above the bottom of the base (Fig. 44). The structure has a storage capacity of 1.4 million barrels.

The platform was built on a construction site located in the Maasvlakte near Rotterdam, Netherlands, and covering an area of 500,000 m^2 with a twin dry dock and an adjoining, well-sheltered water area for completion of platforms afloat.

Construction of the ANDOC platform started in May 1974. Up to a height of 9 m, the cell base was built in the dry dock. This was then flooded and the upper 23 m of the cells, which have a minimum thickness of 450 mm, and the approx. 110 m high towers were subsequently added using the slipforming technique (Fig. 45).

Slipforming of the towers started on March 10, 1976 and lasted until June 3, 1976. The performance achieved was 3.20 to 3.60 m per 24 h, all four towers being slipformed simultaneously. Laser equipment was used for checking and assuring verticality. VSL's essential contribution consisted of providing personnel for the slipforming joint venture.

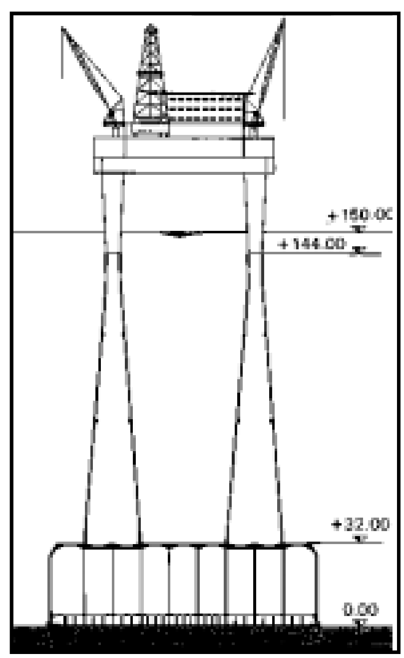

Figure 44: Cross-section of ANDOC platform

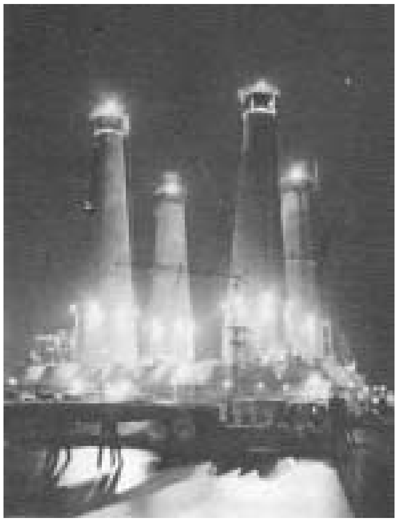

Figure 45: Night view of ANDOC platform under construction

3.2.3. Field Control Station, North-East Frigg Field

Owner Petronord/Statoil Group/ Esso Expro Norway
Engineer Kvaerner Engineering A.S., Oslo, Norway
Contractor Skanska AB, Gothenburg, Sweden
Post-tensioning Internordisk Spannarmering AB, Danderyd, Sweden
Years of construction 1981-1982

Elf Aquitaine Norge A/ S, as the operator of the North-East Frigg Field in the Norwegian sector of the North Sea, had a tubular structure built between November 1981 and August 1982 at Gothenburg, Sweden. The structure is the base for a 140 m high, articulated steel tower which carries a control and helicopter deck. The tower, positioned 15 km from the platform TCP-2, is used as a field control station (FCS) and houses equipment to convert electrical signals into hydraulic pressure for operating the subsea production valves, for controlling the wells, checking for leaks and injecting hydrate inhibitor into the wells (Fig. 46). The concrete base structure has the form of a catamaran and consists of two double tubes, each 41.50 m long and 6.50 m in internal diameter. These are connected together at each end by a tube of 5.70 m internal diameter and at the centre by a boxsection transverse beam. The beam contains the hinge for the steel tower. The distance between the external faces of the double tubes is 45.75 m.

The foundation structure was built from high-quality concrete and is very heavily reinforced. It was built in several stages in the old 400x60 m dock of Eriksbergs Mekaniska Verkstad (Fig.47). Before the actual work commenced, an 8.40 m long pipe was constructed to natural scale as a model. Pressure tests were carried out on the model and the cracks which were produced were subsequently grouted and the model again pressurized up to failure.

The tubes are longitudinally post-tensioned with VSL cables EE 6-12 and 6-15 (breaking load 3,000 and 3,750 KN respectively). A total of 130 tendons of 41.50 and 45.00 m length, requiring approximately 78 tonnes of prestressing steel, were used (Fig. 48).

The 6,800 tonnes weight structure was towed out from the yard in Gothenburg, Sweden, to Stavanger, Norway, in September

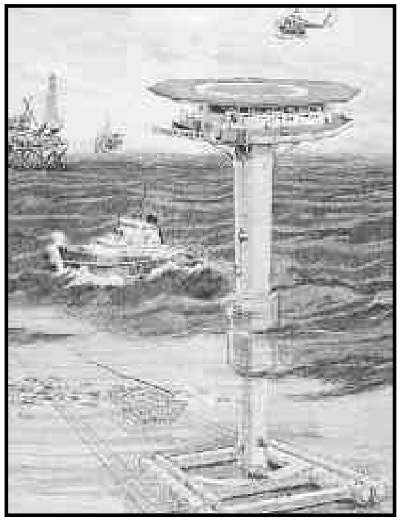

Figure 46: Artisi's impression of the Field Control Station

ber 1982. In Stavanger, it was connected by the cardan joint to the steel tower in a tricky operation, which was carried out in June 1983. The set-down at the intended location in the Frigg Field was performed in August 1983. The FCS came into operation at the end of 1983.

3.2.4. Concrete Island Drilling System (CIDS)

Owner Global Marine Development, Inc., Newport Beach, California, USA
Engineer Alfred A.Yee, Honolulu, Hawaii, USA
Contractor Nippon Kokan K.K.,Tokyo, Japan
Subcontractors Joint Venture Penta-Ocean Construction Co. Ltd./Shimizu Construction Co. Ltd., Tokyo, Japan
Post-tensioning PS Concrete Co. Ltd., Tokyo,Japan
Years of construction 1983-1984

In August 1984, the world's first mobile drilling island was placed on the sea bed in Northern Alaska's Harrison Bay (Beaufort Sea, USA). The island is used for oil and gas exploration, first in a sector owned by Exxon (Fig. 49).

Compared with gravity-based islands, which have traditionally been used for offshore drilling in shallow water, the CIDS features lower construction and operation costs as well as safer operation, since it is not subject to environmental destruction and can be refloated to other drilling locations. Because of these economic and operational advantages, the new mobile CIDS is expected to find wider application in the future.

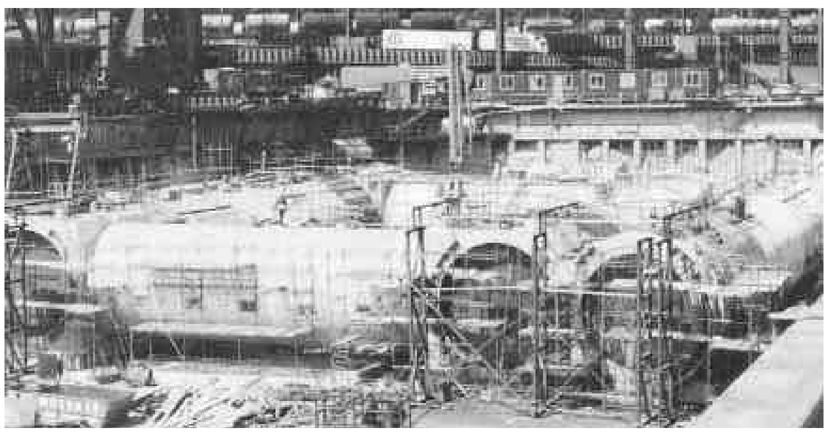

Figure 47: Bottom structure being constructed in the dry dock

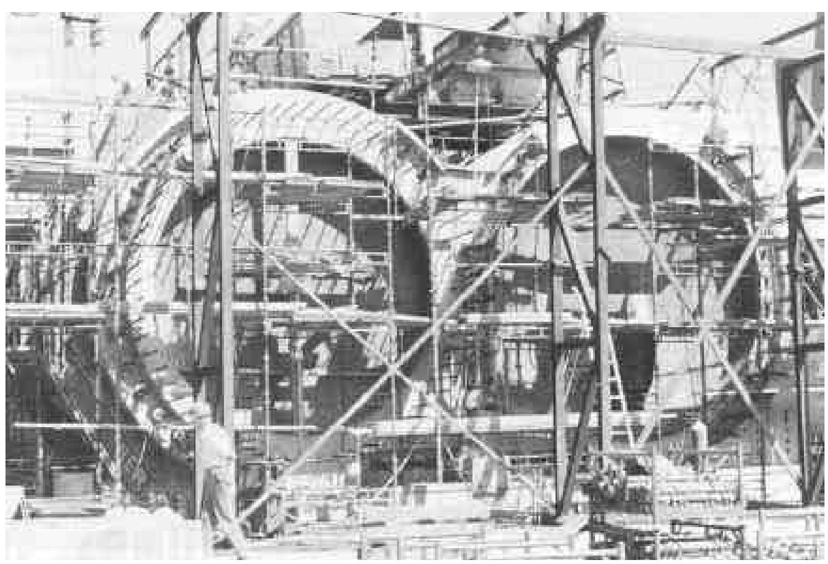

Figure 48: View of end part with VSL anchorages

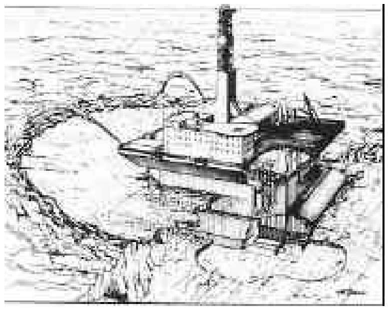

Figure 49: Artist's impression of CIDS in working position

The CIDS is a hybrid structure consisting of three basic sections:

- a steel mud base to sit on the sea bed, 95.27 x 89.94 x 7.62m, weighing 13,000 tonnes,
- a concrete structure to rest on the steel base, 71.34 x 71.34 x 13.42 m, weighing 35,000 tonnes,
- two upper steel deck barges, of 88.57 x 41.46 x 7.93 m, totalling 8,200 tonnes, which were made of special steel for low temperatures.

The function of the steel mud base is to increase the footprint to allow the system to function on poor (soft) soils. The primary function of the concrete module is to resist the ice forces that are to be expected in the Beaufort Sea and to provide support for the system. The CIDS is planned for operating in water depths of 10.70 to 18.30 m. The deck storage barges carry drilling equipment, supplies, repair shops and living quarters for the crew (up to 93 persons).

The design of the CIDS is based mainly on the utilization of appropriate materials for the various functions. The mud base is of structural steel to minimize weight and draft during mobilization and towing. The concrete module functions as the primary iceresistant element, as mentioned above. It is located in the splash zone and will be subjected to severe corrosion, ice pressure and low temperatures. Structural steel was again chosen as the framing material for the deck storage barges, to minimize draft.

The US $ 42 million structure was built between August 1983 and April 1984 in Japan, at the Tsu Works repair dock of Nippon Kokan K.K. (NKK) in Tokyo. NKK, one of Japan's largest steelmakers, constructed the steel parts while the joint venture of Penta-Ocean and Shimizu built the concrete part under a US $ 12 million subcontract (Fig. 50). The three parts were assembled at Isewan Bay, Japan, in April 1984 and the CIDS was then towed to the Beaufort Sea (Fig. 51).

The concrete middle section has heavily reinforced and post-tensioned perimeter walls which are supported by a series of closely spaced, highly reinforced and prestressed shear walls of 5.79 m depth, spanning vertically 13.42 m between top and bottom slabs. These shearwalls and the internal walls were made of high-strength concrete (f_c=55 N/ mm^2). The central core area is a honeycomb structure of lightweight concrete (f_c= 44 N / mm^2). This part, which is filled with water, was made up of precast cylinders of 3 m diameter.

The VSL Post-tensioning System was used to prestress the top slab, the bottom slab and the external walls in the horizontal plane (Fig. 52). This horizontal post-tensioning is applied for crack limitation. The cables are of type 5-7, 5-12 and 5-19.

Acknowledgement:
Drawings and photographs kindly provided by Shimizu Construction Co. Ltd.

3.2.5. Immersed shore approach concrete tunnel, Karmey, Norway

Owner statpipe Company, Oslo
Engineer Dr. O.Oleen, Oslo
Contractor Enginor F.Selmer A/S,Oslo
Post-tensioning
Internordisk Spännarmering AB, DaaderyK, Sweden
Year of construction
1982

The Statpipe Development Project includes a shore approach structure for two gas pipelines on the exposed western coast of Karmoy, an island north-west of Stavanger (Fig. 53). The pipelines are part of Statpipe's 850 km long gas line in the North Sea, which partly comes from the Statfjord Field, where three Condeep platforms are installed (see chapter 3.2.1.) The pipelines are placed on the bottom of a concrete tunnel which, in operation, is water-filled.

The tunnel project was selected after the main alternatives tendered for, like blasting out a sea bed trench or driving a subterranean tunnel, had been discarded by the client as too time-consuming. However, the technical solution of such an immersed concrete tunnel had never been tried before. The time available for design and construction was only 9 months. The geographical location is one of the most exposed areas on the south-west coast of Norway. Although a rush job, nothing was left to chance in the process; model tests were conducted, concrete mixtures of unusually high quality were developed and so on.

The sloping concrete tunnel is 670 m long, starting in 30 m deep waterand ending with an 80 m long cast-in-situ section a shore (Fig. 54). The immersed part consists of 5 precast elements of 90, 110, 120, 120 and

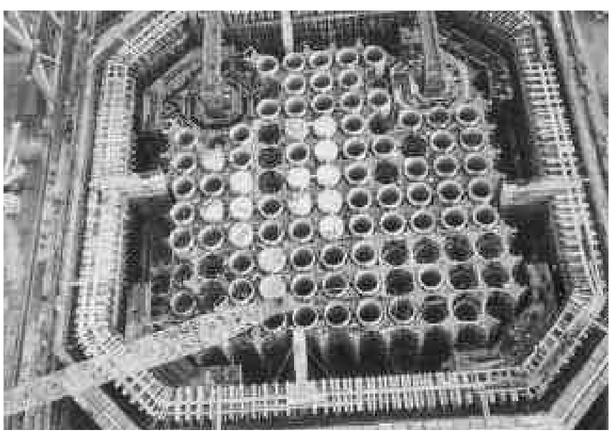

Figure 50: CIDS under construction

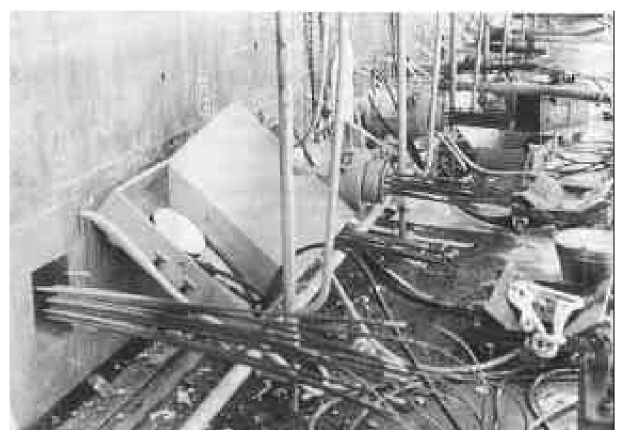

Figure 52: Stressing of VSL cables

Figure 51: Tow-out of the CIDS

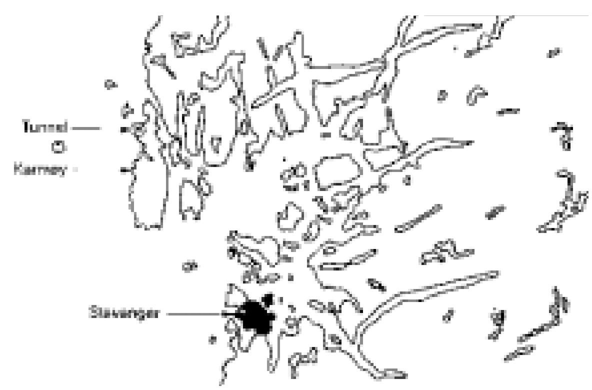

Figure 53: Location of Shore Approach Tunnel

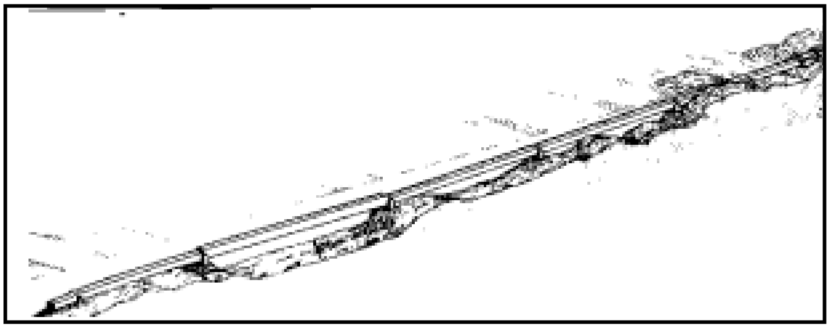

Figure 54: Sketch of Shore Approach Tunnel

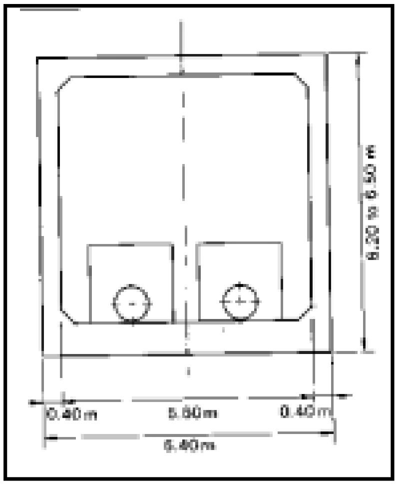

Figure 55: Cross-section of Shore Approach Tunnel

150 m length. The box-shaped tunnel is internally 5.60 m wide and 4.90 m deep, while the outside width is 6.40 m and the depth 6.20 to 6.50 m resulting in a crosssectional area of 30 to 45 m^2 (Fig. 55). The elements rest on 6 heavy foundations, the lower parts of which were cast under water, while the upper parts were prefabricated ashore (at Kalsto, approx. 50 km from Stavanger) and positioned by a floating crane. The weight of each upper part was 400 tonnes.

The five tunnel segments, each one divided into three chambers, were fabricated in dry docks at Kristiansand and Stavanger (Fig. 56) from high quality concrete having a cube strength of 60 to 65 N / mm^2, then sealed, ballasted, tested and towed to Karmoy (Fig. 57). The 90 m and 110 m long elements were constructed at Kristiansand and the others at Stavanger. The weight of the largest element was 7,000 tonnes. Using tugs for positioning and winches for hauling, the segments were then successfully pulled onto their supports against their buoyancy and placed within tolerances of a few tens of millimeters (Fig. 58). All activities were monitored on TV.

Weight control is a very important aspect in the prefabrication industry. For elements to be towed to a site, a high degree of accuracy in fabrication is very important. An increase in the wall thickness of 20 mm would have a dramatic effect on the draft conditions. The specific weight of the materials, therefore, had to be carefully checked. As a result of well-organized planning and thorough control, the deviations from the theoretical values remained very small (less than 1%).

All the cables used for post-tensioning the tunnel elements and the tops of the foundations were of the VSL type 6-19 (breaking load 4,953 KN). In the tunnel elements, cables with an E-anchorage at both ends and lengths of 25 to 116 m as well as EPtype and EH-type cables, 5 m long, were installed. The latter are used as shear tendons at the end of some elements; they are inclined at 45 ° . For the tops of foundations, 129 VSL cables EP 6-19 of 4 to 12 m length were required.

3.2.6. Swedish lighthouses in the Baltic Sea

Owner Swedish Board of Shipping, Norrköping
Engineer Vattenbyggnadsbyran (VBB), Stockholm and Swedish Board of Shipping, NorrKöping respectively
Contractor Swedish Board of Shipping, Norrköping
Drilling Contractor Stabilator AB, Danderyd
Post-tensioning and Rock and Soil Anchors Internordisk Spannarmering AB, Danderyd
Years of construction 1975, 1985-1987

Figure 58: Shore end of the approach tunnel

Figure 56: An element under construction

Figure 57: Towing of an element

Introduction

At the end of the fifties, the Swedish Navigation Authorities drew up a comprehensive programme for the replacement of the majority of the lightships and buoys along the east coast of the country, i.e. in the Baltic Sea, by stationary equipment. Since the weather in this region is frequently very severe, it was found more economical to carry out the building and equipping work as far as possible on land and then to tow out the completed structures to the appointed position.

The structures

The lighthouses presented in this paragraph (Fig. 59) are those at Kullagrund (about 10 km south-west of Trelleborg in the south of Sweden), at Southern Kvarken and at marketskallen (both on the navigable route between the Stockholm archipelago and the Aland Islands). These lighthouses were built at javrebyn, south of Lulea in northern

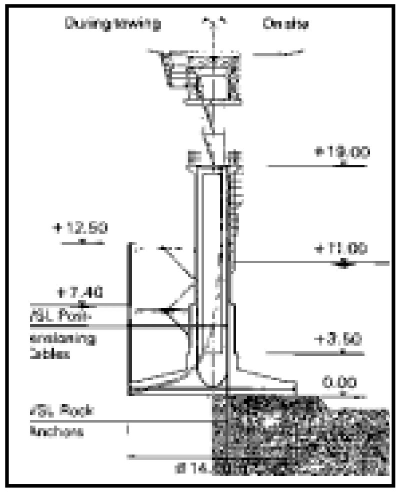

Figure 60: Section through lighthouse

Figure 61: Lighthouse under construction

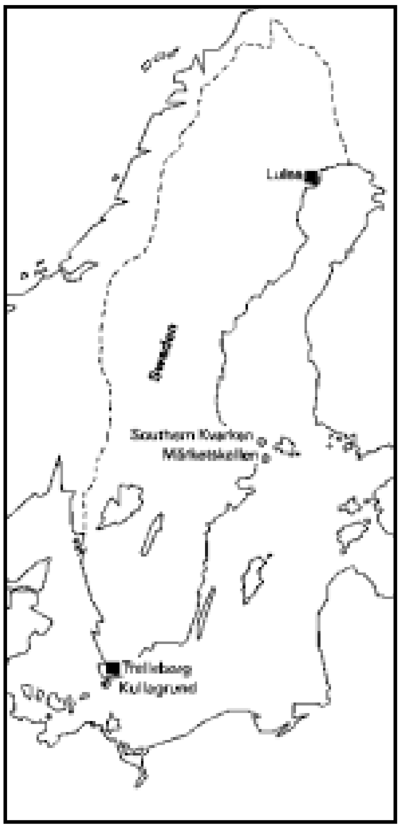

Figure 59: Location of lighthouses

Sweden, where the owner has a permanent yard in a protected bay for production and maintenance. From there the structures were towed to their final positions and anchored to the sea bed.

All three lighthouses are similar in shape. They have a circular foundation slab of 14 to 16 m diameter, supporting a vertical concrete cylinder of 1.60 to 2.40 m internal diameter. A cylindrical steel tower, incorporating the lighting equipment and, in the case of the two newer structures, helicopter platforms as well, tops the concrete cylinder (Fig. 60).

A wall, temporarily erected around the perimeter of the foundation, provided buoyancy during towing (Fig. 61). At their permanent location, the lighthouses stand in 11 to 13 m of water (mean level) on a flat ballast bed laid on the hard moraine or rock of the sea bed.

Post-tensioning

the concrete cylinders and foundation slabs of the lighthouses at Southern kvarken and Marketskallen are each post-tensioned by means of eight L-shaped VSL cables EE 6-19, with a radius of only 2 m. The strands were pushed into the ducts from the top and the cables tensioned simultaneously from both ends before tow-out (Fig. 62).

Anchoring

The stability of a lighthouse during use can be assured either by a suitably large and heavy foundation slab or by anchoring the structure into the sea bed with soil and rock anchors. The anchor method enables the design to be simplified and material to be saved, as the size of the foundation slab is reduced. In the case of the kullagrund tower, the savings amounted to 14% of the net construction costs. At the same time, anchoring provides the additional safety required for extreme conditions caused by waves of 9 to 10 m height and by ice pressure, especially with regard to stability against sliding.

The kullagrund tower is anchored with six permanent VSL Soil Anchors, which are vertical and uniformly distributed in the concrete cylinder wall. Each anchor consists of seven strands 0.5" Dyform and has an ultimate capacity of 1,463 kN. To protect the strands against corrosion, they are greased and individually sheathed in a polyethylene duct, except in the bond length.

For each of the lighthouses at Southern kvarken and Marketskallen, four VSL Rock Anchors E_R 6-19 dyform, consisting of unbonded strands (monostrands), were used. The rock anchors of the Marketskallen tower were stressed in September 1985 to a test force of 75% of ultimate and anchored at 50% of ultimate. Twelve hours after the crew had left, however, a 40,000-tonne icebreaking tanker rammed the lighthouse (Fig. 63), rupturing all the post-tensioning cables (Fig.64) but not the unbonded ground anchors. The lighthouse therefore had to be completely removed and a new one was built which was placed and anchored in July 1987. The structure for Southern Kvarken was launched in Spring 1986.

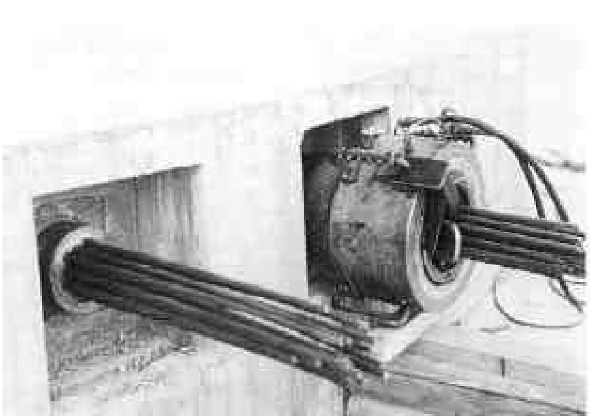

Figure 62: Stressing of VSL cables

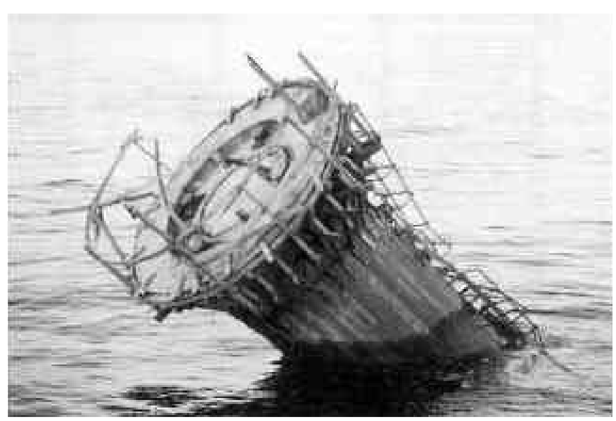

Figure 63: The destroyed lighthouse of marketskallen

Figure 64: Torn cables

Figure 67: Form of bottom part on lowering platform

3.2.7. Caissons for harbour quay, Venice, Italy

Owner Genio Civile Opere Marittime, Venice
Engineer Ufficio Tecnico Adriatica Lavori Marittimi S.p.A.,Milan
Contractor Adriatica Lavori Marittimi S.p.A., Milan
Heavy Rigging and Slipforming VSL Italia s.r.l. (now PRECO S.r.l.), Milan/VSL INTERNATIONAL LTD., Berne, Switzerland

Years of construction 1973-1974, 1976-1977

For the extension of the Isonzo jetty in the harbour of Venice, reinforced concrete caissons were built in two phases. In the first stage, carried out in 1973/ 74, sixteen caissons were constructed, while the second stage (1976/77) comprised ten caissons.

All the caissons are identical in dimensions, 13.50 m high, 6.60 m wide at the bottom, 4.70 m wide at the top, and 15.25 m long. Each comprises eight vertical compartments. The thickness of the outer walls is 250 mm and that of the internal bulkheads 200 mm. Two metres above the bottom there is a horizontal slab 250 mm thick, separating the (lower) compression chamber from the (upper) compartments (Fig. 65). The caissons constitute the basement for a concrete superstructure which carries quay service equipment.

The caissons were built on a steel platform;

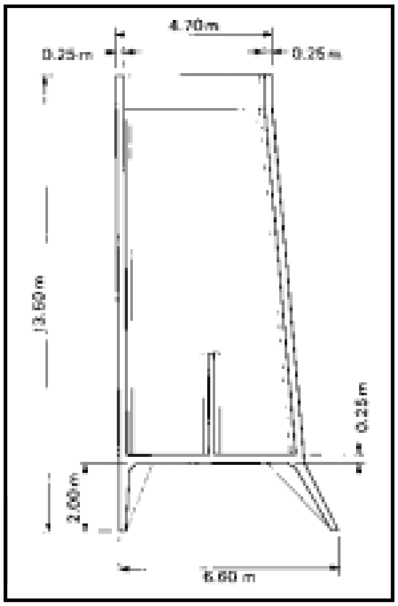

Figure 65: Cross-section of caisson

this was suspended from a steel frame structure standing on the sea bed close to the existing jetty. The suspension consisted of rods of the VSL Rod Lifting System. Six VSL motive units H 80 were installed on top of the frame structure, with the rods passing through them.

Construction of a caisson took about one month and was carried out in the following sequence (Fig. 66):

- A portion of 2.40 m height, i.e. the compression chamber including the horizontal slab, was first constructed in a steel form (Fig. 67).
- The walls of the upper part were then raised by VSL Slipforming. While the slipform rose, the platform with the already constructed part of the caisson was progressively lowered into the water (Fig. 68). The maximum net weight to be carried by the lifting equipment was approximately 166 tonnes.
- When a caisson was completed, the steel platform was lowered and the caisson was floated off and towed out laterally. In this phase, the slipform was temporarily suspended from the frame. After towout, the caisson was ballasted with gravel and sunk in place on the harbour bed.
- Finally, the platform, together with the form work for the first part of the caisson resting on it, was lifted to the starting position for the construction of the next caisson.

By constructing the caissons one after another in a single temporary installation, simple and economical construction and handling were achieved.

The VSL Rod Lifting System was used in this case because it provided the required rigidity and it was well suited with regard to the unit force required. For larger forces, units of the VSL Strand System will have to be used.

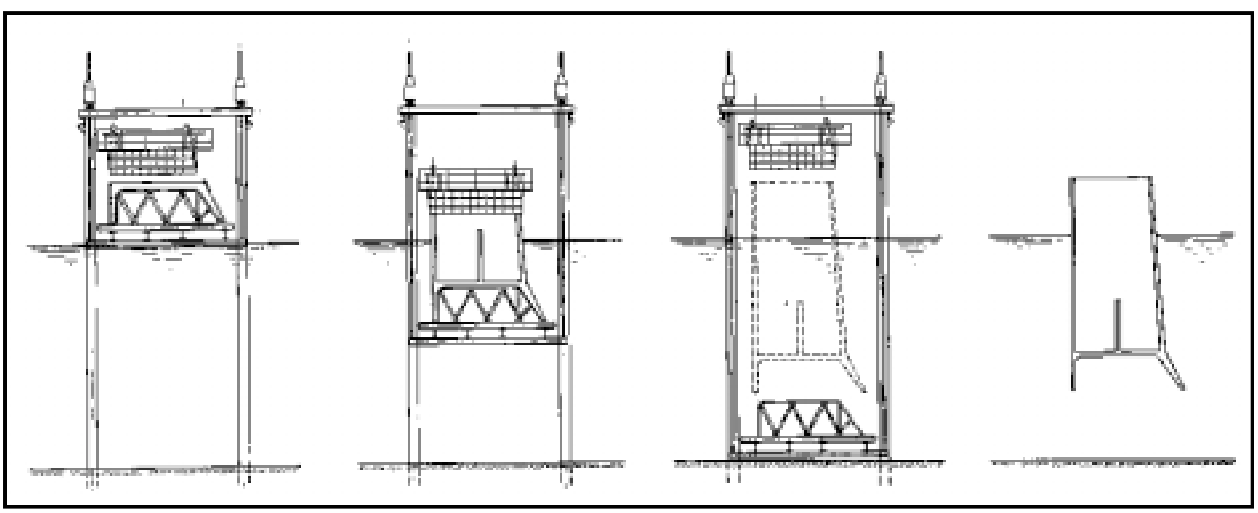

Figure 66: Schematic representation of construction procedure

Figure 68: View of Caisson uncler construction

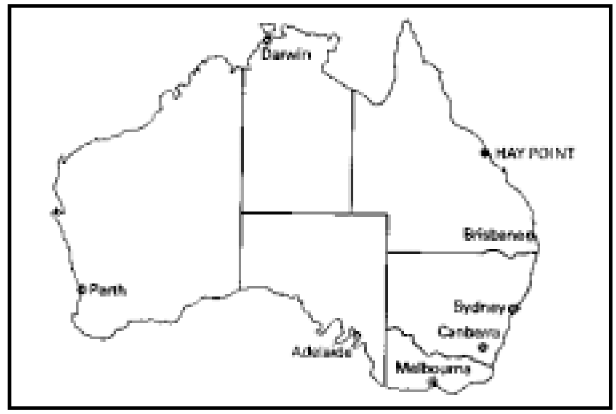

Figure 69: Location of Hay Point

3.2.8. Caissons for berth at Hay Point, Queensland, Australia

Owner Utah Development Co. (now owned by Broken Hill Proprietary Co. Ltd., Melbourne)
Engineer Rendel Scott Furphy, Melbourne
Contractor Joint Venture of John Holland Constructions Pty. Ltd., Melbourne/ Christiani & Nielsen, Denmark
Post-tensioning VSL Prestressing (Aust.) Pty. Ltd., Thornleigh
Years of construction 1972-1975

Introduction

The first berth at Hay Point, capable of accommodating bulk carriers up to 120,000 deadweight tonnes (dwt) was commissioned in September 1971. It was constructed as a steel piled trestle to carry the shiploader and conveyor, with structurally separate berthing dolphins forward of and underneath the trestle. This berth was the only outlet for the export of about 10 million tonnes per annum of coal from Utah Development Company's mines at Goonyella and Peak Downs. With the introduction later of a third mine at Saraji, detailed studies indicated that a second berth would be required to handle the combined production of 15 million tonnes per annum (Fig. 69).

Fundamental Considerations

It is of interest that the first and second berths are totally different in their concepts, because the problems presented to the designers were also totally different. For the first berth, the problem was to design and construct a coal loading facility 2 km out in the open sea, in a very isolated location, whereas for the second the problem was to design and construct a coal loading facility 2 km out in the open sea, immediately adjacent to an existing installation, with a ship coming in and departing approximately every two days.

The new berth is immediately north of the existing one, from which laden ships depart northwards (Fig. 70). I n view of the exposed conditions in which the operations take place, it was evident that a partly completed structure built by conventional methods in this position would have presented a serious shipping hazard for a period of about two years until it could have been completed with all its fendering. The partly completed structure would itself also have been very vulnerable during this period.

Another major consideration was that construction of a berth piecemeal in the open sea necessitates access for men and materials in an exposed situation every day for up to two years. This proved to be a major problem during the construction of berth No. 1, as there were many days when the conditions for access were marginal or impossible. Consideration was of course given to eliminating this problem by constructing berth No. 2 using the existing approach trestle and berth head for access purposes. However, this did not offer a solution to the potential shipping hazard problem and would have resulted in serious congestion on the existing installation.

The scheme finally selected

A number of schemes were considered in detail, including caissons, jackets and a duplication of berth No. 1 using sea access only. After a detailed analysis of comparative capital cost and construction risks, concrete caissons were finally selected, some of their main advantages being as follows:

1. Absolute minimum of work to be carried out at the exposed offshore site and consequently a maximum of work in the sheltered conditions of MackayHarbour.
2. Minimum risk to shipping during the construction period.
3. Low maintenance cost.
4. A heavy caisson floats and sinks in a stable, upright position. The caisson scheme therefore allowed the fabricated steel shiploader to be transported to Hay Point on board one of the caissons after being completely erected and assembled in the calm conditions of Mackay Harbour. Elimination of the very difficult task of erecting a shiploader in the exposed conditions represented a considerable saving.

The scheme finally adopted is shown in Fig. 71. Five caissons were needed to support the approach trestle. These caissons became known as the Approach Caissons (AC). Each one consists of a prestressed concrete cellular box 17.37 m square in plan and 7.62 m deep, divided into sixteen cells. The three berth caissons (BC) have a 12.19 m square and 18.29 m high reinforced concrete cellular column on each corner. The bases of BC 1 and BC3 are 45.72 m long, 38.71 m wide and 7.92 m deep, each divided into 99 cells in plan. BC 2 is 3.96 m wider, having 110 cells to provide extra buoyancy to transport the 1,000 tonnes weight shiploader to Hay Point (Fig. 72). North of berth No. 2 two mooring dolphins were required; these have concrete bases identical to those of the AC's.

Figure 70: Hay Point berths

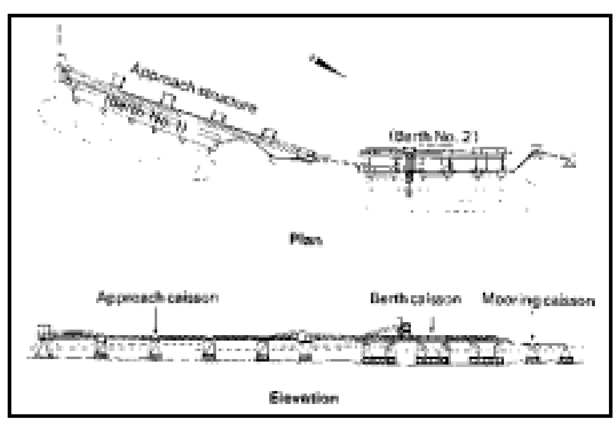

Figure 71: Plan and elevation of berth no. 2

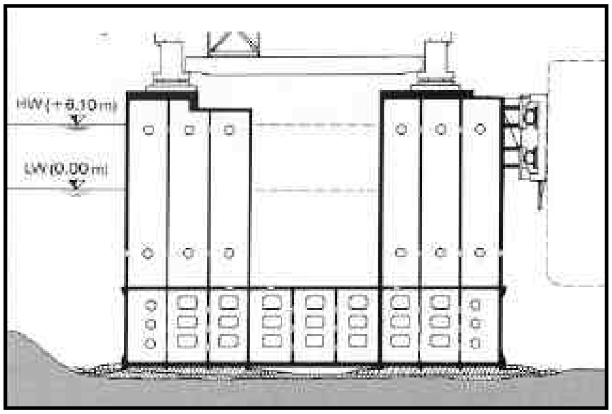

Figure 72:Cross-section of a berth caisson (BC)

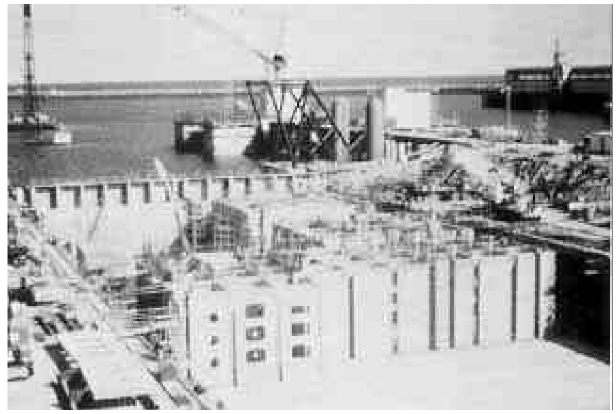

Figure 74: Construction stage (dry dock)

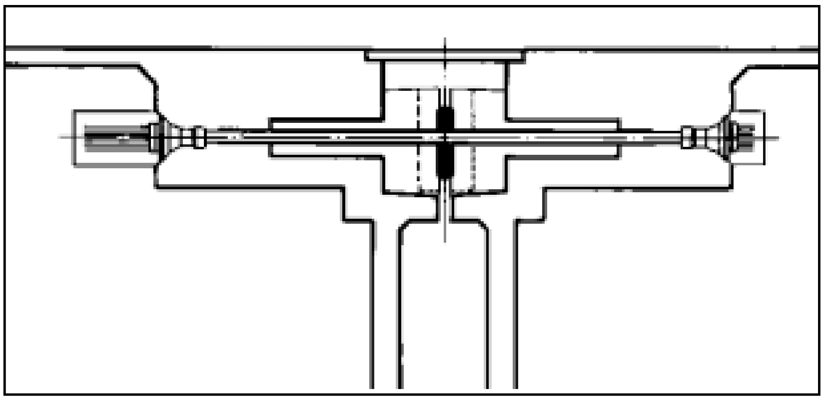

Figure 73: Section through hinge

Method of construction

The caissons were partly completed in a specially constructed temporary dry dock at the western side of Mackay Harbour. The dock was designed for use with a straddle carrier and a purpose-built overhead gantry crane, each of 350 KN capacity, to handle the precast wall units of the caissons. For obvious economic reasons the depth of the dry dock was limited to that required for partial completion of the caissons. At the fitting-out berth, at which the caissons were completed, it was necessary to seat the BC's at such a level that after completion they could be floated off at their final draft (10.67 m) at high tide.

The caissons were assembled from precast wall units varying in thickness from 203 to 406 mm with cast-in-place floors and roofs (Fig. 74). The principal reason for choosing precasting for the wall units was the existence of the somewhat conflicting requirements for thin units, to limit the draft of the caissons when afloat, and very durable concrete, to ensure the necessary life of the structure in sea-water. For substantially the same reasons, it was decided to post-tension the caissons horizontally, except for the BC columns.

Undoubtedly the principal criterion underlying the whole design was the assurance of absolute safety during the towing and sinking operation. This led to an early decision that the caissons must remain afloat and stable even in the event of major damage to an outer wall, or to the floor and roof.

The wave forces on the caisson were estimated by means of a hydraulic model at the University of Queensland. Two other model tests to investigate the floating and towing characteristics of the caissons were undertaken at the Universities of N.S.W. and Melbourne respectively. The structural design of the caissons required the checking of a large number of different loading cases.

Post-tensioning

An average prestress of about 5.3 N/mm^2 was used in the horizontal direction to cater for the various load cases. A loss of 1.4 N/ mm^2 in the floor and roof and a similar gain in the walls was calculated on the basis of the differential shrinkage effects.

In total 1,885 VSL cables of types 5-12 and 5-19, of 18 to 45 m length, were installed. Thus 680 tonnes of strand were used. The cables were either pre-assembled and fed into the ducts by winches or the VSL Pushthrough Method was applied. The latter method was judged to have particular advantages for the shorter cables. The tendons were stressed from one end only, 50% of them from one face and 50% from the opposite face (Fig. 75).

3.2.9. Caisson pile caps for Yokohama Bay Bridge, Japan

Owner and Engineer

Metropolitan Expressway Public Corporation, Yokohama

Contractor and Post-tensioning

Joint Venture of Taisei Corporation /Maeda Construction Co. Ltd./Shimizu Construction Co. Ltd., Tokyo

Years of construction

1982-1986

On September 27, 1981, the foundation stone was laid for the Yokohama Bay Bridge. This will come into service in 1989. It is a two-storey motorway bridge with six traffic

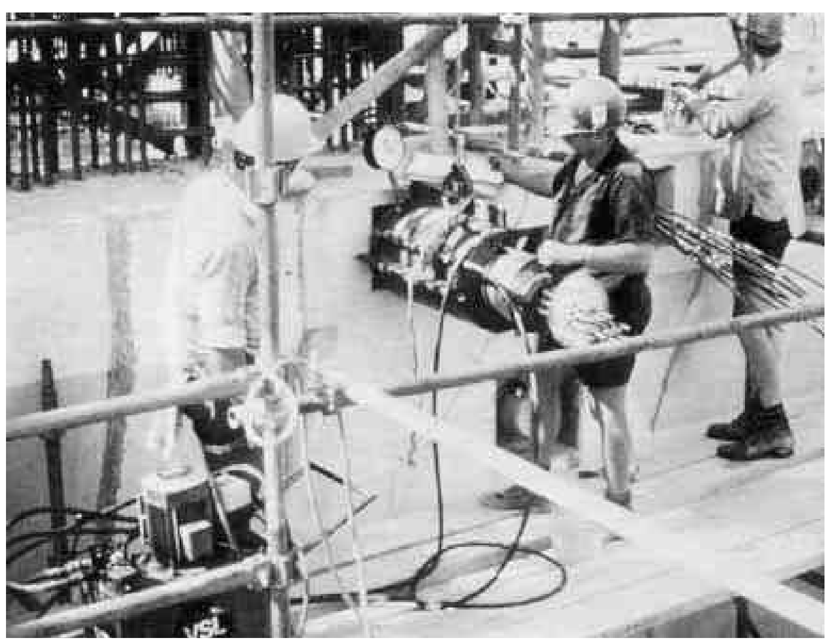

Figure 75: Stressing work being carried out

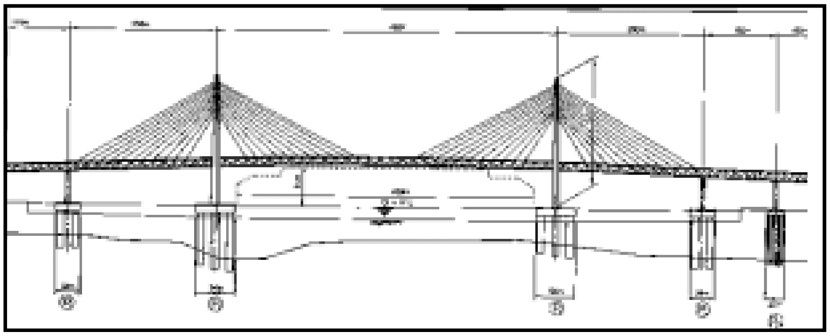

Figure 76: Longitudinal section of bridge

Figure 80: Post-tensioning cables

lanes on each level. The superstructure consists of a steel lattice girder with spans of 3X116-200-460-200-3X100 m. The three main spans are cable-stayed (Fig. 76).

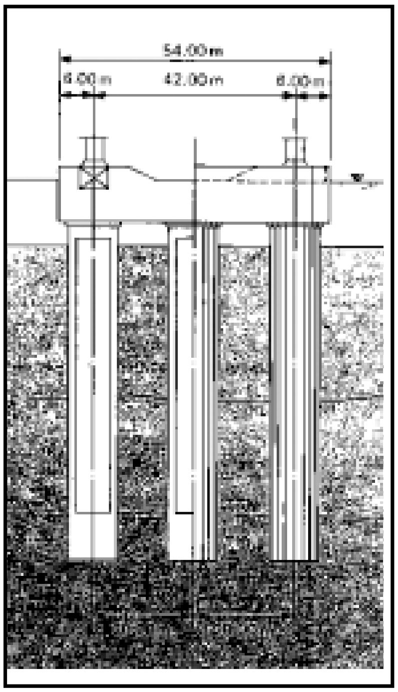

Figure 77: Section through foundation

Each of the two pylons from which the bridge girder is suspended is 172 m high and is founded on nine circular caisson piles, which are fixed in a box-shaped pile cap supporting the two pylon legs. The two end piers of the central part of the bridge are founded in the same manner also, but for these only six caisson piles were necessary. This type of foundation is entirely new in bridge construction. It is comparatively light and therefore more economical than other methods (Fig. 77).

The pile cap structures, when completely finished, measure 56 X 54 X 12 m (large caps) and 54 X 34 X 12 m (smaller caps). The thickness of the outer wall is 800 mm. Twin inside walls are provided in the spaces between the caisson piles. The pile cap structures were built in a dry dock up to a height of 9.25 m at the outer wall and to a lower level than the full height also for the inner walls (Fig. 78). They were then towed a distance of 15 km to the site (Fig. 79), where they were sunk onto temporary piles. There, the wall construction was completed. The structures were first used as working platforms, formwork, water retaining walls, reaction tables for sinking the caisson piles, etc. VSL Heavy Lifting equipment was used to assist in sinking the piles.

The use of a prestressed concrete barge as a working platform minimizes the construction area and reduces the working period on site. Finally, the pile cap structures were filled with concrete to a determined height in order to form the permanent pylon supports.

Element	Part	VSL Anchorage Type
Pile cap	External walls	E 6-19 P 6-19
	Twin inside walls	E 6-12 P 6-12
	Base slab	E 6-19 P 6-19
Footing of pier foundation	Concrete poured into the pile caps after completion of caisson piles sinking operation	Z 6-14 V 6-14 K 6-14
In addition:		
Caisson piles	Walls	E 5-19 K 5-19 V 5-19 P 5-19

Table II: VSL Cables and anchorages used

Figure 78: Pile caps under construction in the dry dock

Figure 79: Tow-out to the bridge site

3.2.10. Floating concrete batching plant, Japan

Owner Honshu-Shikoku Bridge
Bisan Seto Ohashi Sub-structure Joint Venture Company

Engineer, Contractor and Post-tensioning Taisei Corporation, Tokyo

Years of construction 1981-1982

Construction of the North and South Bisan Seto Bridges on the Kojima-Sakaide Route, linking Honshu and Shikoku, started in 1978. Substructure work for the bridges was finished on schedule in 1985. Completion of the crossings is anticipated for 1988. Section 4A of this work is a 250,000 m^3 concrete structure for anchoring the main cables of the adjacent suspension bridges. For supplying this concrete, a posttensioned concrete barge equipped with a concrete batching plant of 150 m'/h output was constructed. It was used in this section from August 1982 to January 1985.

The barge has a length of 62 m, a width of 23 m and a depth of 10 m. It is a box-like structure, with spaces for storing approx. 16,000 tonnes of aggregates, 1,000 tonnes of cement and 2,000 tonnes of pure mixing water. It also carries aggregatereceiving equipment, muddy water treatment equipment, ice-making plant, air and water supply equipment, disposal equipment for surplus concrete, a central control room, concrete testing room, emergency generator and so on (Fig. 81).

The barge was built of lightweight concrete (γ= 1.9 t/m^3) made from artificial lightweight aggregates and normal Portland cement. The design concrete strength was f_c=45 N/mm^2. As the barge has a dual function, i.e. to float during towage to the site and to rest on the sea bottom during plant operation, the following design requirements had to be met:

- Considerably reduced weight for floating (draft factor 0.55),
- The large load of about 20,000 tonnes had to be supported with the barge resting on the sea bed during plant operation,
- Stability against slipping on the sea bed when subjected to ocean waves, tidal current and earthquakes.

The barge was built in a dry dock (Fig. 82). In order to establish the method of construction and tocontrolstandards,aprogrammeof experimental work was carried out before construction of the main body of the barge commenced. The programme included checking the strength and

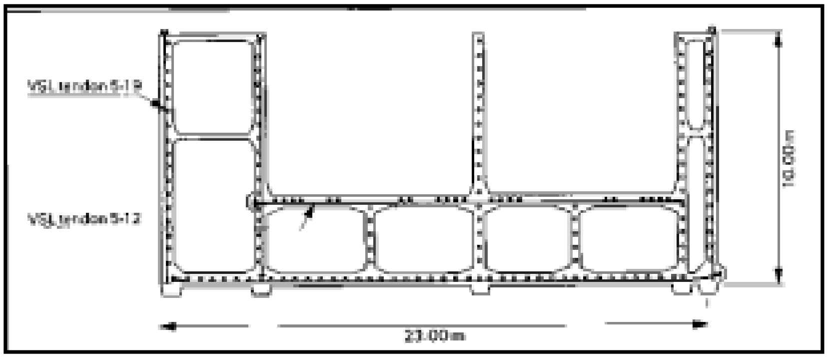

Figure 81: Section through the barge

pumpability of the lightweight concrete and determining the amount of plasticizer to be added. In a structure of complicated form, many cracks affecting its durability will normally occur, due to thermal stress and shrinkage. The cracks were therefore controlled by dividing the barge into several blocks. In view of the construction conditions in the existing dock, the barge was cast in place, although precasting would also have been possible. Thermocouples and high-sensitivity strain gauges were embedded in the concrete for monitoring the temperature and confirming the stage-by-stage increase of prestress. The results of the observations were used for determining the curing method and period and the amount of primary prestress. As each box was completed, it was filled with water to test and confirm the watertightness of its inner and outer portions.

The barge was launched with parts of the plant already installed. The centre of gravity of the structure is eccentric from that of the equipment and sea water was therefore used for ballasting. A perfect balance was maintained during floating by finely adjusting the volume of ballast by the slope control system using a microcomputer. The tow was started immediately after the barge had left the dry dock. The fleet comprised three main tug-boats and several auxiliary tugboats and guard-boats, in orderto cope with fast tidal currents (4 to 5 knots) in the Setonaikai Inland Sea. The towing distance was about 90 km and the draft of the barge during towing 5.45 m (Fig. 83).

The barge is horizontally and vertically posttensioned. VSL cables EE 5-12 and 5-19 were used longitudinally and VSL tendons EP 5-12 and 5-19 transversely in slabs and walls. The longitudinal cables were installed after concreting and stressed from both ends. The transverse tendons were placed before concrete was poured. To prevent cracks during construction, a partial prestress of 1 to 2 N/mmz (i.e. one-fifth of the total amount) was applied in all directions. The longitudinal cables were stressed two days, the transverse tendons four days after concreting.


Figure 83: Barge being towed to the operating location


Figure 82: Barge during construction in dry dock


Figure 84: Barge in use at bridge construction site

3.3. Permanently floating structures

3.3.1. Concrete barge 'C-Boat 500'

Owner, Engineer, Contractor and Post-tensioning Taisei Corporation, Japan

Year of construction 1982

Taisei Corporation, in conjunction with the Japan Ship's Machinery Development Association, have developed a marine floating structure of lightweight concrete. A prototype barge, of 37 m length, 9 m beam and 3.10 m depth and of 4,900 KN (500 dwt) loading capacity was first built to this design (Fig. 85). The barge is not self-propelled but is designed to be towed. A service speed of 6 knots can be achieved.

Up to 95% of the hull can be precast. The hull is stiffened longitudinally and transversely by ribs like a steel vessel. The barge does not require any special corrosion protection, it is fire-resistant, shock-resistant and thermally insulating and it needs virtually no maintenance. All these features contribute to its economy in use. The design may be applied to floating berths, cold food storage, LNG storage, leisure facilities and so on. Large floating concrete structures can be assembled by joining individual barges together (using VSL cables and joining material).

For the prototype, the parts of the hull were precast from lightweight concrete (density γ=1.8 t/ m^3, design strength 50 N / mm^2) in the workshop and were trucked from there to the assembly plant (Fig.86). The segments were of two types: firstly, units comprising a part of the bottom, of the side wall and of the deck as well as vertical diaphragms and, secondly, units forming a part of the bottom only (in total 40 units). They were joined by strips of cast-in-place concrete. The stern and bow were also cast in place. In the deck, two openings of 11.0X5.0 m were left and were covered with removable roofs. Longitudinally, the structure is post-tensioned by VSL monostrand cables.

The development of the barge included various tests (watertightness and strength of lightweight concrete, actual construction procedure, etc.) for determining the behaviour of the materials and of the finished structure. The barge therefore combines the benefits of experience gained in many years of concrete technology with many of the features specific to marine technology and shipbuilding.

3.3.2. Floating Concourses, Brighton Marina, UK

Owner The Brighton Marina Company Ltd., Brighton

Engineer Ove Arup & Partners,London

Contractor Taylor Woodrow Construction Ltd., Southall

Post-tensioning VSL Systems Ltd. (formerly Losinger Systems Ltd.),Thame

Years of construction 1976-1978

Introduction

Brighton Marina is a privately developed boat harbour near Brighton, Sussex, about 100 km south of London. The harbour is wholly artificial and has required the construction of a major sea wall together with reclamation and associated works. The scheme provides moorings for 2,047 boats and allows development of up to 1,250 apartments and maisonettes, a 500bed hotel, a conference centre, a boatyard, social and yacht clubs, car parking for 4,648 cars, public entertainment zones and shopping and exhibition areas. The water area of the Marina is divided into a locked basin and a tidal basin, separated by a spine of reclaimed land (Fig. 89).

The tidal harbour can accommodate 1,300 boats. The maximum design tidal range is 8 m. It was specified that amenities such as access for emergency vehicles, water, electricity, telephone and sewerage be provided close to the points where individual boats are moored. In order to avoid large fixed quays, it was decided to provide two floating concourses, which would give:

- Access for boat owners to their vessels in the tidal harbour,
- Access to the moorings for emergency vehicles, such as fire tenders and ambulances,
- Extension of water, electricity and telephone services to moorings,

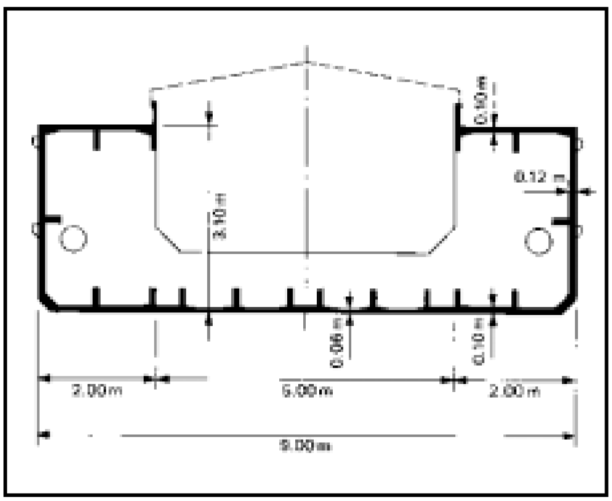

Figure 85: Cross-section of 'C-Boat 500'

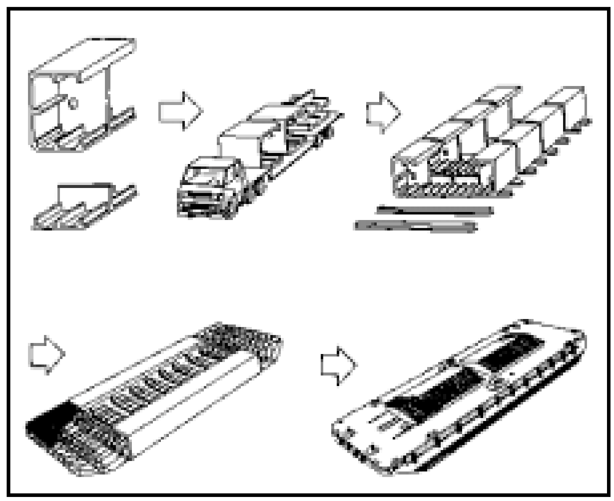

Figure 86: Schematic representation of construction procedure

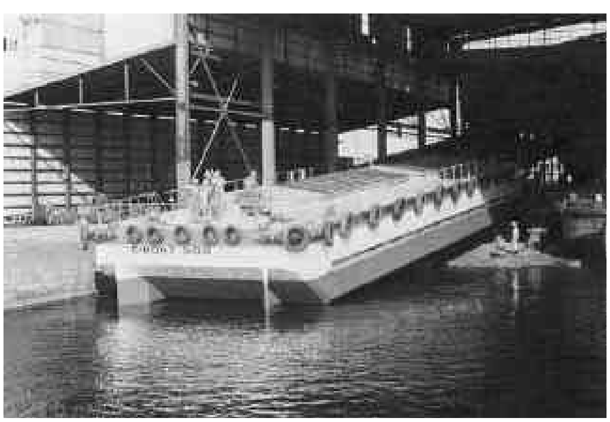

Figure 87: Launching of the 'C-Boat 500'

Figure 88: View of the 'C-Boat 500' during towing

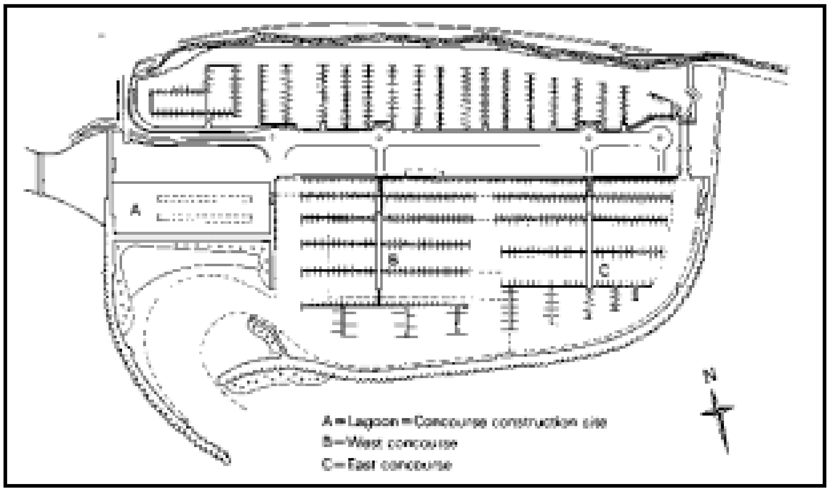

Figure 89: Plan view of the marina

- Normal washing and toilet facilities near the moorings,
- An elevated deck available as a public area or for holiday cabins/housing.

The structure

The concourse substructure is a cellular box 159.60 m long, 9.60 m wide and 1.80 m deep. The hull is divided longitudinally and transversely by diaphragms into bays not exceeding 15 m length. Ribs at 3 m centres provide a secondary slab support system and lateral stiffening (Fig. 90). The concourse superstructure is a rigidly jointed steel frame, divided into three approximately equal bays, clad with lightweight concrete. A 30 m span bridge links each concourse to the shore. The concourses are moored at each end of the leading pile of each dolphin.

Dense concrete made from sea-dredged aggregates and having a minimum design compressive strength of 50 N/mm^2 at 56 days was used for the substructure. Sulphate-resisting Portland cement and granular blast furnace slag with a Superplasticizer were used. The water/cement ratio was 0.42. The specific characteristics required for the concrete of the hull were:

- Minimum drying and thermal shrinkage,
- Minimum permeability,
- Maximum workability,
- Maximum cohesiveness.

Static analysis

The basic global static analysis for dead and live loads was executed with the use of a finite difference programme for beams on an elastic foundation. Transformed section properties were used. The effect of a change in the modulus of elasticity and moment of inertia (EI) was investigated. Whereas the deflection of the structure was found to be relatively insensitive to variations in section stiffness, bending moments varied at about 40% of the change in EI.

The wide range of load intensities from small uniformly distributed loads to relatively large point loads caused some difficulties with accuracy in computation, point loads near the ends being a particular source of difficulty. A total of 18 dead load and live load conditions and 44 combined load cases were considered. A rectangular grid frame model with longitudinal members 4.80 m apart and transverse members at 15 m spacings was developed for analyzing the effects of waves; these were represented by nodal forces varying as the wave profile, assuming the concourse remains rigid and calculating the equilibrium position iteratively. The elastic foundations were modelled as springs with vertical and rotational stiffnesses assuming that a standing wave exists; water pressures are hydrostatic and rotational spring stiffnesses are given by calm water conditions.

Construction

The concourses were constructed in a temporary dry dock formed by dewatering the lagoon (Fig. 91). Work commenced in July 1976. The lagoon was flooded in January 1978 and the concourses floated out in February 1978. The west concourse was completed in June 1978, the eastern one in September 1978 (Fig. 92). The basic crosssection was divided into 8 pours of approximately 15 m length. The problem of weight control and assessment was continuous. The initial target was a total selfweight of 1,965 tonnes. The final design weight was 2,140 tonnes. The final as-constructed weights were 2,220 tonnes and 2,260 tonnes for the west and east concourses respectively.

Post-tensioning

The post-tensioning cables are of the VSL type 6-12, dyform low-relaxation strand being used. The tendons (6 at each lower and upper end of a web) are straight, except at theirends, where they are curved inwards to bring the anchorages within the basic cross-section. The duct diameter was minimized (Ø 70/77.mm, galvanized spiral wound steel) to reduce the wall and haunch thickness necessary to contain it. Peak stress within the anchorage was 0.85 f_u at the end of stressing to achieve 0.70 f_u minimum at mid-span. The tendons were partly installed by pushing-through, partly by winching them through as complete tendons.

Figure 91: The concourses under construction

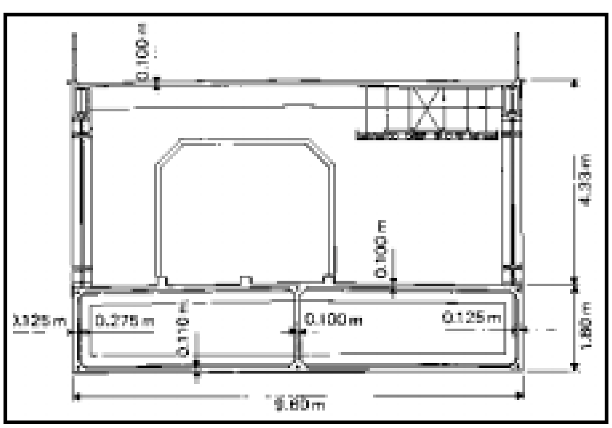

Figure 90: Cross-section of the concourse

Figure 92: View of the completed marina

3.3.3. Third Lake Washington Floating Bridge, Seattle, USA

Owner and Engineer Washington State Department of Transportation, Olympia, WA

Contractor Joint Venture Kiewit Pacific Company, Vancouver, WA/ Ernest L. G rice & Sons, Federal Way, WA

Post-tensioning VSLWestern, Campbell, CA

Years of construction 1984-1987

On Lake Washington, a fresh water lake forming part of the non-tidal harbour of Seattle, eight pontoons have been installed as part of the new Interstate 90 Roadway Bridge. Growth of the area surrounding the City of Seattle has led to the existing roadway becoming heavily congested with commuter traffic twice a day. The owner expects to have the full Interstate 90 project completed some time in 1992. The floating bridge was selected as the best method of carrying traffic across Lake Washington. The distance across the lake is in excess of 1.6 km (1 mile). The lake bed, some 60 m (200') below the water surface, is of soft mud, which is unsuitable for piles as foundation for bridge structures.

The eight concrete pontoons are 22.86 m (75') wide; six of them are 106.68 m (350') and the other two 57.91 m (190') in length. These pontoons were attached to existing pontoons on Lake Washington and a superstructure was built on several of them. The two short pontoons are 8.38 m (27.5') deep and the six long ones range in depth from 7.62 m (25') to 10.97 m (36'). Each pontoon is divided by internal walls into cells 8.84 m (29') long and 4.27 m (14') wide. When the eight pontoons were in position, cantilevered wings 4.57 m (15') wide were added on each side. This provides a roadway surface 32 m (105') wide (Fig. 93). It was not possible to place the wings while the pontoons were being built, because the pontoons would have been too wide to pass through the lock to reach their destination. Pontoon construction started on July 9, 1984, four weeks after the joint venture had been awarded the contract. The eight pontoons were built on a site of 89,000 m^2 (22 acres) located in the Port of Tacoma. This site is about 65 km (40 miles) by road from the final location of the pontoons. It took almost 14 hours to tow the pontoons from the pit to the lake site. Four pontoons were constructed at a time, resulting in two

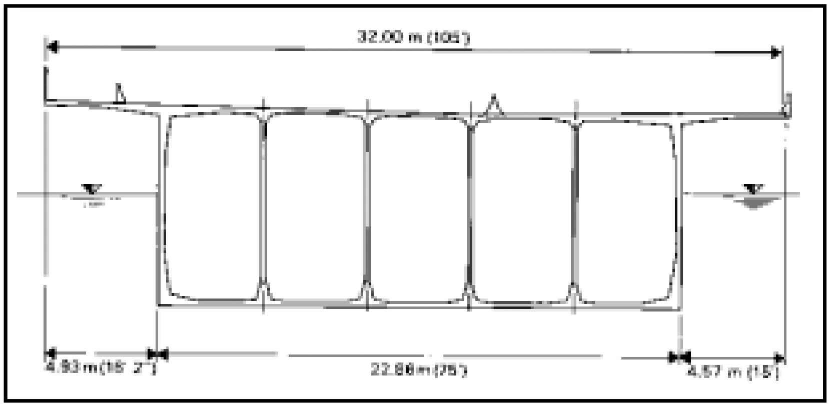

Figure 93: Cross-section of structure with location of longitudinal post-tensioning cables

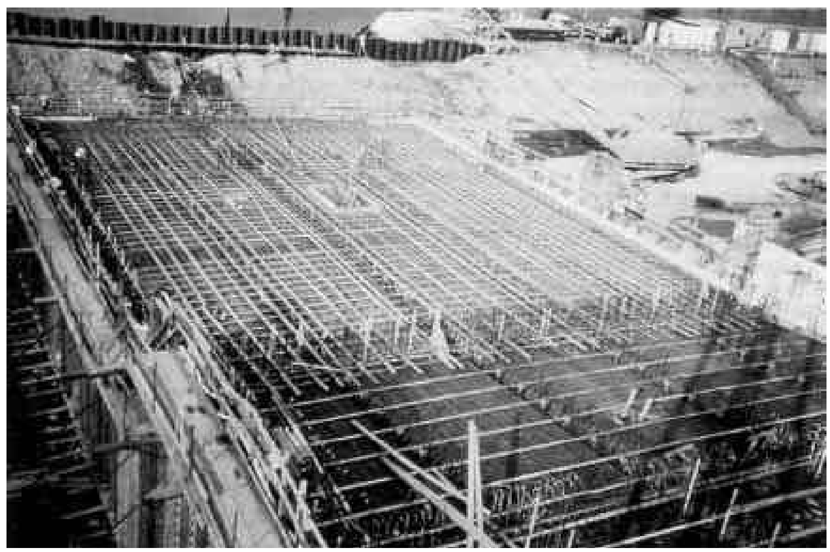

Figure 94: Pontoon under construction in the dry dock

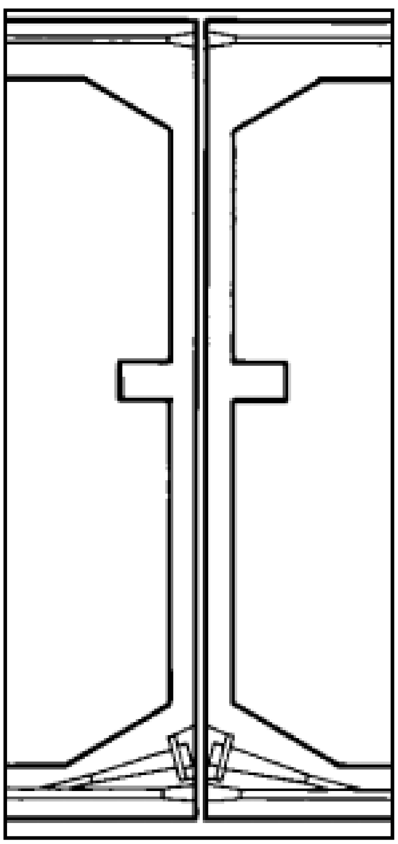

Figure 95: Detail of pontoon joint

Figure 96: Third Lake Washington Floating Bridge under construction

construction cycles to build the total of eight (Fig. 94).
The pontoons were heavily post-tensioned longitudinally, transversely and vertically. Longitudinally 984 VSL cables EE 5-27 (breaking load 4,960 KN) were installed, while the top and bottom decks were transversely post-tensioned with 2,852 VSL tendons SO/SO 6-4 (breaking load 1,043 KN). Approximately 1,680 tonnes of strand in total were used. Vertical post-tensioning was by means of threadbars. The bottom and top deck transverse tendons were stressed first, then the vertical web tendons and finally the longitudinal web tendons. At the lake site, the pontoons were connected by a 102X 102 mm (4"X4") rubber gasket, which is attached to one end of the outer post-tensioning connecting ducts (Fig. 95). The two pontoon ends were then aligned and pressed together. This was accomplished by tensioning EE 5-27 tendons with 500-tonne jacks until the gap formed by the gasket was reduced to 51 mm (2"). The two smaller pontoons were also secured to the ends of the larger pontoons with post-tensioning strands.
The second four pontoons were floated out in May 1986; the pontoon work was substantially completed in January 1987.

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Tel 31-71-768 900

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