DESIGN AND CONSTRUCTION OF A PRESTRESSED CONCRETE PRESSURE VESSEL FOR A WORKING PRESSURE OF 69 N/mm² (10,000 p.s.i.)

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SUMMARY

Construction is nearing completion of a pressure vessel with a chamber 9.15 m (30 ft.) high and 3.05 m (10 ft.) internal diameter for hydraulic tests on marine components up to 69 N/mm² (10,000 p.s.i.) working pressure.

The chamber comprises a steel cylinder, with independent end plates contained within a prestressed concrete structure. The cylinder is constructed in two halves, each consisting of three forged rings, 170 mm thick, shrink-fitted onto a 90 mm thick liner. It rests on a 100 mm thick bottom plate, provided with a band of hard-facing overlay on which the cylinder slides in response to changes of test medium pressure. Models to be tested within the chamber are hung from a removable 150 mm thick top plate. A central elliptical hatch provides access into the chamber.

Special sealing assemblies are fitted at the junction of the cylinder sections and between the cylinder and end plates. These seals are capable of accepting radial expansion of the cylinder and corresponding vertical movements at the upper seal arising from elastic movements of the enclosing structure.

The top plate is restrained by a wire-wound prestressed concrete closure plug, itself located by twelve bifurcated inclined steel struts which transfer the load on the top plate into the concrete structure. The struts are retractable to allow removal of the closure plug and top plate.

The enclosing concrete structure is 25 m (82 ft.) high and 11 m (36 ft.) diameter. It is vertically prestressed by 180 no. 540 Tonne tendons and circumferentially prestressed by 5 mm wire laid under tension in pre-cast concrete channels by the Taylor Woodrow Wire-Winding System. The structure was analysed, using limit state principles, by computerised elastic and non-elastic dynamic relaxation techniques. The results were evaluated against triaxial stress criteria established from relevant research work and experience obtained from nuclear prestressed concrete pressure vessels.

Model tests were carried out on components of the concrete structure to provide an essential supplement to the analytical studies. Initially, one-thirtieth scale tests were used to establish the primary dimensions, prestress level and mode of failure. These were followed by working and overload tests, including cyclic loadings, on separate one-fifth scale models of the bottom cap and the reaction ring/struts/closure plug assembly. These models successfully withstood pressure up to 190 N/mm² (27,000 p.s.i.) with no evident distress.

Construction of the concrete structure proceeded generally in 2 m (6 ft.) lifts using conventional techniques. The mean 28-day concrete strength was 60 N/mm² (8,700 p.s.i.) with a standard deviation of 3.9 N/mm² (570 p.s.i.). The bottom plate was installed prior to prestressing operations and grouted to the underlying concrete with high-strength grout in a two-stage operation. The plate is further secured to the bottom cap concrete by 30 No. 40 mm diameter vertical prestressed steel bars around its periphery.

At present, a steel-framed service building is being erected on top of the chamber after which the cylinder, seals and closure plug will be lifted into place and the strut assemblies installed. Following commissioning of auxiliary services, the initial proof pressure tests to 1.25× working pressure is scheduled for May 1978.
1. **INTRODUCTION**

In 1971, Taylor Woodrow Construction Ltd. were asked by the United Kingdom Naval Construction Research Establishment to investigate the possibility of designing a 69 N/mm² (10,000 psi) pressure chamber, 3.05m (10ft.) diameter and 9.15m (30ft.) long based on the structural design principles which had been developed for prestressed concrete pressure vessels for nuclear gas-cooled reactors [Ref. 1]. A number of ideas were considered from which the present design of an inner steel chamber housed within a prestressed concrete yoke structure was evolved.

Following a period of feasibility studies to prove particular features and to establish primary dimensions, detailed design commenced in 1974 and work commenced at site in 1975. At the present time, concrete construction and prestressing have been completed and the high-level building above the chamber is being erected in readiness for the delivery of the internal chamber and closure system components.

The primary functional design requirements are as follows:

<table>
<thead>
<tr>
<th>Requirement</th>
<th>Specification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Operating pressure</td>
<td>69 N/mm² (10,000 psi)</td>
</tr>
<tr>
<td>Annual proof test pressure</td>
<td>96.25 N/mm² (12,500 psi)</td>
</tr>
<tr>
<td>Test medium</td>
<td>Fresh or sea water</td>
</tr>
<tr>
<td>Temperature</td>
<td>5°C - 25°C (25°C)</td>
</tr>
<tr>
<td>Design life</td>
<td>20 years</td>
</tr>
</tbody>
</table>

A general arrangement of the chamber is shown in Fig. 1. This identifies the major structural components whose design is described in this paper.

2. **CONCRETE STRUCTURE**

2.1. **General**

The concrete structure surrounding and containing the inner pressure chamber, although continuous, can be regarded for design and descriptive purposes as three separate components, the bottom cap, the barrel and the reaction ring. The design of each component was investigated separately followed by analytical studies to confirm the overall behaviour of the structure.

The primary objective, for both the bottom cap and reaction ring was to achieve a design which took maximum advantage of the strength of concrete under triaxial stress conditions.

The absence of recognised codified criteria for this type of design necessitated a cautious approach to the preparation and evaluation of analytical data. In addition it was considered essential to confirm the predicted behaviour of the structure by operational and overload tests on large-scale fully-representative models of the bottom cap and reaction ring and closure system.

2.2. **Small Scale Model Tests**

Tests on 1:30 scale models of a range of bottom cap and reaction ring designs were carried out to establish the primary dimensions and optimum level of circumferential prestress. The models were loaded to failure in a compression testing machine with continuous measurements being taken of their deflection under increasing load.
As a result of the tests on bottom cap models, the depth of the cap, the intensity of circumferential prestress and the preferred layout of the lower anchorages of the vertical prestressing system were established. These were chosen to give a minimum ultimate Factor of Safety of 3 against shear failure for the full-scale structure, based on a concrete design strength of 45 N/mm² (6500 psi).

Similar parameters were established for the reaction ring. In addition the angle of inclination of the struts was fixed at 30° to the vertical and the need for an inner liner was confirmed.

2.3. Analysis

Separate analyses of the bottom cap and reaction ring were carried out using a "Dynamic Relaxation" computer program developed for axisymmetric structures by Taylor Woodrow Construction Ltd. Stresses, strains and movements were evaluated for these components under prestress only, operating, and proof pressure loading conditions, at early and late life. The complete concrete structure was also analysed for the same loading conditions using similar analytical techniques but with a coarser grid.

The effective value for the elastic modulus of the concrete used in these analyses was estimated from specific strain curves prepared by TWG from the results of creep test data for the type and strength of concrete to be used for construction. Estimates of concrete shrinkage were obtained and used in a similar way. A value of 0.16 for Poisson's ratio was used for all analyses.

Vertical tendon prestressing loads were calculated on the basis of a tendon-duct friction coefficient of 0.20 and a wobble factor of 0.0015/m and assumed that the tendons were stressed to 75% GUTS before lock-off with a 5mm anchorage pull-in after lock-off. Late life prestressing loads allowed for strand relaxation losses as estimated by the manufacturer. Wire-winding prestressing loads were calculated on the basis of a winding tension of 70% GUTS, again with appropriate relevant allowances for relaxation.

Stresses in the concrete structure obtained from the D.R. analyses were initially compared with the permissible limits allowed in B.S. 4975 "Specification for Prestressed Concrete Pressure Vessels for Nuclear Reactors" [Ref. 2]. In some areas the stresses were slightly outside the conservative limits of this standard and in these cases they were assessed against the criteria recommended for triaxial compression by J.B. & K. Newman in their CIRIA document "Criteria of Concrete Strength". [Ref. 3] In all cases the stresses were comfortably within the "envelope of initial cracking" recommended in this document.

Tensile reinforcement was provided in all areas where the analyses identified tensile concrete strains. In addition, reinforcement was provided for control and distribution of surface shrinkage cracking and also around the prestressing anchorages in accordance with normal practice.

2.4. Reaction Ring Steel Components

The reaction ring liner provides a reaction to the circumferential prestress to ensure that the concrete is in a state of substantially uniform radial compressive stress. It is fabricated from steel to B.S.4360 Grade 50E, having a minimum yield stress
of 325 N/mm² (21 T/eq. in.). The maximum hoop stress in the liner, derived from the
L.R. analysis of the reaction ring, is about 150 N/mm² for the late-life prestress
only condition. Shear keys, formed from 50mm square bar welded to the concrete face
of the liner, ensure compatibility of vertical strains.

Twelve strut bearing plates are provided to distribute the load from the inclined
struts into the body of the reaction ring concrete. The bearing plates are 125mm
thick with two 100mm thick flanged radial webs and were cast in steel to B.S. 1456
Grade A with a minimum yield stress of 340 N/mm² (22 T/eq. in.).

A detailed finite element analysis was carried out to evaluate the behaviour of
the plates under maximum loading conditions. This showed that the stresses in the
steel were within permissible limits, the maximum stress being about 170 N/mm² com-
pression near the base of the webs. The analysis also showed that the pattern of
stresses in the surrounding concrete was acceptable to B.S. 4975 criteria with the
addition of some reinforcement to control secondary tensile effects. In this area, as
elsewhere, reinforcement detailing took account of the need to avoid such high densities
of reinforcement that there would be a risk of inadequate concrete compaction during
construction.

2.5. Fifth-Scale Model Tests

When the design of the concrete structure was substantially complete, fifth-
scale models were constructed of the bottom cap and the reaction ring to confirm the
behaviour of these components under operating and overload conditions. These models
replicated in as great a detail as possible the essential features of the full-size
structure. In addition the reaction ring model included a scaled closure system to
ensure representative loading on the reaction ring and to enable parallel tests on the
closure plug to be carried out. (See Section 4.3 below).

The models were vertically stressed against a reaction base and loaded by a pressure
cell to simulate the hydraulic loads from the main chamber. An extensive array of
embedded strain gauges, surface deflection gauges and prestressing tendon load cells
was incorporated in the models to record their behaviour. Tests included static loading
to chamber operating and overload pressures, and sustained and cyclic load tests.

The test results substantiated the earlier analytical studies. Seal failures in
the hydraulic system precluded continuing the 4ultimate4 load tests to their intended
limit of three times chamber operating pressure; the bottom cap model test was
discontinued at 140 N/mm² (20,000 psi) and the reaction ring test at 190 N/mm² (27,500 psi).
Neither model showed any sign of structural distress or significant non-linear tendencies
at the maximum load reached and each subsequently underwent a test to chamber operating
pressure, repeating its previous elastic behaviour. In view of this, further attempts
to obtain the intended load were not pursued.

2.6. Prestressing Systems

Vertical prestress is provided by 180 No. prestressing tendons arranged in
five rings, anchored at the top and bottom surfaces of the reaction ring and bottom cap
respectively. Each tendon is made up of 19 No. 18mm diameter "Dyform" low-relaxation
strand and has an ultimate load capacity of 720 tons. They are anchored by FSG 19/18
Monogroup anchorages and protected from corrosion by cement grout pumped into the ducts from the lower ends after prestressing.

Circumferential prestressing is provided by the Taylor Woodrow Wire Winding System [Ref. 4]. Layers of 5mm diameter low-relaxation wire are laid under tension in steel lined channels formed in pre-cast concrete shutter units. Each channel is 615mm wide and 200mm deep and contains up to 24 layers of wire. On completion of winding operations, fibre-glass covers are fitted to the channels and the void between the covers and the wire filled with grease.

3. INNERR CHAMBER

3.1. Bottom Plate

The primary functions of the bottom plate are to provide a water-tight membrane at the bottom of the chamber, to transfer chamber hydraulic loads into the bottom cap concrete and to provide a flat level surface on which the cylinder rests. In addition hydraulic and instrumentation services enter the chamber through penetrations in the bottom plate.

The plate is 4.2m (14ft.) diameter 100mm (4in.) thick, fabricated from steel to B.S. 1501 - 161 Grade 26A. The area of the plate on which the cylinder and bottom seal (See Section 3.4, below) rest is overlaid with Deloro Alloy C. This is a nickel-based alloy used to provide a hard wearing, flat, smooth, highly corrosion-resistant surface. Elsewhere the plate is coated with Epoxy paint. Heavy radial and circumferential stiffeners are provided on the underside of the plate to limit deflection during handling and to reduce distortion as the overlay was applied.

To provide further resistance against possible bottom plate deflection during service the bottom cap concrete under the plate is stiffened by a large quantity of vertical reinforcement. This represents about 9% of the concrete cross-sectional area under the plate and effectively stiffens this zone against vertical deflection by about 50%. In addition, the edge of the plate is prestressed to the bottom cap by 30 No. 40mm diameter 2m long MacAlloy prestressing bars around the periphery of the plate.

3.2. Cylinder

The steel cylinder is designed to withstand the radial bursting forces applied by the test medium. It consists of an inner liner with a yield stress of 410 N/mm² (27 T/sq. in.) onto which forged hoops, with a yield stress of 860 N/mm² (57 T/sq. in.), are shrink fitted. At zero chamber pressure the hoop stresses are 200 N/mm² compressive in the liner and 110 N/mm² tensile in the hoops. At chamber operating pressure these stresses increase to 220 N/mm² tension and 500 N/mm² tension respectively, with a corresponding radial expansion of about 3.4mm. At proof test pressure the stresses are 330 N/mm² and 600 N/mm² respectively with a radial expansion of about 4.2mm.

A fracture mechanics analysis was carried out on the forged hoops supported by fracture toughness tests on samples cut from a prototype hoop. A 100% ultrasonic examination was made of each production hoop using direct and skew probes in accordance with ASTM A 398-71. The specified minimum acceptable equivalent defect diameter was 4mm. No defects were found.
The cylinder is constructed in two sections so that the maximum lift is within the capacity of the permanent craneage installed over the chamber. Each section will be handled with an external friction grip steel belt. The seal between the two sections is recessed into the liner to avoid restricting the available working area with the chamber. The seal consists of an 8mm diameter neoprene rubber O-ring with a 2mm thick scarfed P.T.F.E. anti-extrusion ring.

The areas of the cylinder on which the top and bottom seals will bear are overlaid with a nickel-based alloy for the same reasons as those described above for the bottom plate. Elsewhere the inside of the cylinder is coated with an epoxide paint and the outside with a high durability chlorinated rubber compound. A 2mm thick layer of bronze-filled P.T.F.E. will be positioned under the lower section of the cylinder to reduce friction as the cylinder expands and contracts in response to changes in chamber pressure.

Detailed design and construction of the cylinder has been carried out by B.V.S. Grenoble, France.

3.5. Model Support Plate

The model support plate forms the top closure to the pressure chamber and transmits hydraulic loads into the concrete closure plug. Models to be tested in the chamber are bolted to the plate before it is lifted into position. At the centre of the plate a removeable elliptical hatch is provided for access to the inside of a model for inspection and instrumentation purposes. Pipe work to models under test pass through the plate alongside the access hatch.

The design of the model support plate was primarily influenced by the need to minimise deflections and stresses in the central area around the elliptical hatch opening where the plate is not supported by the closure plug. After studying several alternative schemes, a flat plate design was selected. The nominal plate thickness is 150mm and the steel has a minimum yield strength of 550 N/mm², which is equivalent to HY 90 (U.S.A.) or Q345 (U.K.). The plate is formed as a flat forging and hard-facing overlay is applied over the areas on which the top seal, the models and hatch will bear.

The plate was analysed using "PAFAC", a finite element computer programme, with axisymmetric elements. The closure plug and model support system were included to ensure adequate representation of the plate boundary restraints and loading conditions. Analyses were performed for a range of closure plug concrete E-values, to study the effect of varying support stiffness on the unsupported central area of the plate. Plate stresses calculated by the analyses were compared with design criteria based on ASME Boiler and Pressure Vessel Code, Section VIII, Division 2 [Ref. 5], using a permissible design stress intensity equal to two-thirds of the nominal tensile yield stress of the material.

The domed elliptical access hatch at the centre of the model support plate is formed from a steel casting to B.S. 3100 Grade 37 which has a yield stress of 695 N/mm² (45 T/eq. in.). It is designed to deform elastically to accommodate the differential deflections around the corresponding opening in the model support plate.
3.4. Scale

The design of sealing systems for the upper and lower ends of the pressure chamber proved, as expected, to be one of the most severe problems of the project. The bottom seal is required to accommodate the radial movement of the cylinder as it expands and contracts in response to chamber pressure. The top seal has to contend not only with this movement but also with the corresponding vertical movement of the model support plate caused by the elasticity of the closure system and the concrete structure. This movement is difficult to predict accurately; 30mm is expected for a pressure change of 69 N/mm² and a target of 60mm has been adopted for the top seal design.

Early attempts to devise a seal test rig at half-scale were abandoned because full representivity of the design conditions for the chamber seal could not be achieved and the value of the results which could have been expected did not justify predicted rig construction and operating costs. The designs which have been finally selected, in preference to a number of promising alternatives, are therefore based on a careful theoretical appraisal of their functional suitability associated with an assessment of manufacturing feasibility and cost.

The bottom sealing system consists essentially of a continuous ring of rubber, approximately 100mm square in section. At the angle formed by the cylinder and bottom plate the seal fits over a segmented lead-bronze anti-extrusion ring. The seal is pre-loaded against steel stops bolted to the bottom plate to ensure it is effective at low chamber pressures. These stops also provide a means of centring the cylinder as it contracts during falling chamber pressure.

At the top of the chamber a seal carrier ring is fitted in a housing bolted to the underside of the model support plate. Under operating conditions the carrier ring is forced into the angle formed by the cylinder and model support plate and sealing is effected by elastomer seals fitted in housings in the carrier ring. The space between the carrier ring and its housing can be pressurised to ensure effective sealing at low chamber pressures. This provision allows the outside diameter of the carrier ring to be slightly smaller than the corresponding internal diameter of the cylinder so that the model support plate can be removed and replaced without difficulty.

A step is formed at the top of the cylinder by setting back the seal bearing surface by about 30mm from the nominal internal face of the cylinder. During chamber pressurisation the test medium will exert a downward force on this step to counteract the upward frictional force exerted by the top seal as it follows the movement of the model support plate.

4. CLOSURE SYSTEM

4.1. Closure Plug

The concrete closure plug is basically a deep high-strength concrete disc circumferentially prestressed by wire-winding. Its design is based on Taylor Woodrow Construction Limited’s earlier work on the concrete boiler closures for Hartlepool and Heysham P.C.P.V.’s [Ref. 6]. By uprating the circumferential prestress and adjusting the geometric proportions it is possible to take advantage of the high tensile load
factor of the boiler closures to achieve a satisfactory design for the much higher loading on the NGRE closure plug.

The top surface of the plug is formed from twelve inclined steel castings which transfer the load from the struts into the body of the concrete. A face plate is fitted to the underside of the plug to provide a uniform bearing surface with the model support plate. A central 880mm diameter shaft allows access to the hatch in the model support plate when the plug is in position in the chamber. The shaft has a 65mm thick steel liner to provide a reaction to the circumferential prestress and maintain the concrete in radial compression.

Circumferential prestress is provided by 30 layers of close packed 2.65mm diameter low-relaxation wire laid under tension. The wire is anchored by standard barrel and wedge grips bearing against cast steel anchor blocks at the upper and lower ends of the plug. The anchor blocks are supported on a 6mm thick steel outer liner which also serves to distribute the high local stresses under the first layer of wire into the concrete. On completion of winding the wires are encased by a steel outer cover and the space between the wires and the cover is filled with grease.

The basic parameters of the closure plug were initially established on the basis of the results of ultimate load tests on 1:18 scale models of a range of possible designs. From these tests the depth of the plug and the quantity of prestress were determined. Computer analyses were carried out to determine the required wire-winding tension, taking account of concrete creep and shrinkage and steel relaxation. As a result of this work a winding tension of approximately 5 KN (1100 lbs.) = 48 GUTS, was specified.

Both the model tests and analytical studies indicated that the concrete strength should be as high as possible. Following a series of laboratory trial mix designs, it was established that it was realistic to specify a characteristic strength of 75 N/mm² (11000 psi), and this was subsequently achieved at site.

The final analysis of the closure plug was carried out using a non-linear dynamic relaxation computer programme developed by Taylor Woodrow Construction Limited [Ref. 7]. The stress-strain relationship for triaxially stressed concrete was assumed to be parabolic with the maximum allowable stress being equal to the uniaxial stress plus two and a half times the average restraining stress. The maximum principal stress was then used to determine the position on the corresponding stress-strain diagram and hence the individual secant modulus to be used in the computation for each element. Stress patterns were established for a range of chamber pressures up to three times operating pressure. At this level the analysis showed that the closure plug would be exhibiting significant overall non-linear behaviour, but would not have reached a state equivalent to total collapse. With the type of computer programme used the procedure is liable to become locally unstable close to limiting stress levels. It was judged that this situation was being reached at three times chamber operating pressure and therefore analysis was not pursued beyond this point.
4.2. Struts

Chamber pressure loads on the closure plug are transmitted into the reaction ring by twelve inclined steel struts. Each strut is bifurcated, having one bearing at the closure plug end and two at the reaction ring end to ensure maximum stability under load. The struts are high strength steel castings to B.S. 3100 Grade 693, with a minimum yield stress of 695 N/mm² (45 T/eq.in.). They were subjected to 100% ultrasonic examination to ensure freedom from unacceptable defects.

The ends of the struts are fitted with spherical seatings to enable the strut system to tolerate the small relative vertical and horizontal movements of the closure plug and reaction ring during chamber pressurisation. The corresponding spherical housings have a slight interference fit with the ball-ended struts to ensure maximum uniformity of load transfer.

When not in use the struts hang vertically from pivots at their upper ends. They are raised into their operating mode by hydraulically operated linkages housed in a recess in the concrete barrel wall behind the struts. A second hydraulic system then drives wedges under the closure plug end of each strut to ensure there is no initial slack at the start of chamber pressurisation and also to provide a means of taking up small variations in the nominal strut length.

The operating linkages and their hydraulic control system are arranged so that each strut may be moved independently of the others. The power pack and control panel are situated above the pressure chamber and all normal operations on the struts can be initiated and controlled from this position.

Detailed design of the struts and their control system is by Vickers Design & Projects Division, Southampton, England.

4.3. Fifth-Scale Model Test

As mentioned above (Section 2.5.), a fifth-scale model of the closure system was incorporated in the reaction ring fifth-scale model and was satisfactorily subjected to a series of normal operating, sustained, cyclic, and overload tests culminating in a test to 190 N/mm² (27 500 psi). The closure plug exhibited no signs of distress whatsoever at the end of this severe sequence of tests.

For this model the strut system could not be modelled to scale in all respects as its design factor of safety is lower than that for the concrete components. However, the essential features of rotating end seatings and wedge tightening were retained.

Strain gauges near the centre of the closure plug concrete recorded a vertical strain change of about 6 000 microstrain at the maximum pressure reached. This is very close to that predicted by the non-linear analysis referred to above. Radial and circumferential strains were also close to their predicted values.

Measurements of strain on the surface of the struts showed that all twelve struts were carrying an almost identical share of the applied load and that bending moments in the struts were not only small in magnitude but tended to decrease for successive tests.
5. **CONSTRUCTION**

5.1. **Foundations and Bearings**

The foundation for the chamber is a heavily reinforced concrete slab, 2m thick and approximately 13m x 9m in plan. It is carried on 114 No. 150 ton capacity steel H-piles driven about 50m into the ground.

The chamber itself stands on a 550mm thick, 3m high circular support wall structurally continuous with the foundation slab. Between the chamber and the support wall 40 No. 500mm diameter 50mm thick laminated rubber bearing pads are provided to allow for small radial movements of the chamber during prestressing and during its operating life. Rotation and lateral movement of the chamber is prevented by castellations formed in the top of the support wall and matching recesses in the bottom cap soffit. Further bearing pads are fitted on the vertical end surfaces of the castellations.

5.2. **Concrete**

Concrete was placed by crane and skip, generally in 2m lifts, there being 15 lifts in all. The concrete mix design had a water/cement ratio of 0.44 and an aggregate cement ratio of 3.96 using 40mm maximum size dolerite aggregate. The mean 28-day cube strength was 60 N/mm² (8700 psi) with a standard deviation of 3.95 N/mm² (570 psi). Pehflo Standard was added to the concrete to improve workability. In areas where the reinforcement was severely congested a 20mm maximum size aggregate was used, the concrete having similar properties to that described above.

Great care was taken to ensure accurate location of the twelve cast steel reaction ring strut bearing plates. These weighed over 3 tonnes each and were supported on a temporary structural steel framework incorporating facilities for alignment and positioning of the plates. The same framework also supported the 75mm thick reaction ring liner before concreting. Due to construction constraints, this liner was constructed and lifted into place in two halves, each about 2m high, and the joint between the two halves was filled with Epoxy putty.

About 150 strain gauges were cast into the concrete structure during construction. These will be used to monitor the behaviour of the chamber during the annual proof pressure tests.

5.3. **Bottom Plate Grouting**

The bottom plate was positioned in a recess left in the bottom cap concrete before prestressing commenced. Six 50mm diameter threaded studs were accurately located and grouted into pockets, the plate was fitted over these and levelled and secured by nuts on the studs above and below the plate.

The 200mm deep void between the plate and the underlying concrete was filled with high strength cement grout in a two-stage operation. Initially the void was "flood" grouted, and during this stage 50mm diameter rubber Ductubes fitted to the underside of the plate were inflated. When the first stage grout had set these tubes were deflated and withdrawn to provide paths for the second stage grout. This was injected at a pressure of 0.35 N/mm² (50 psi). The grout for both stages contained a double dose of Cormix F1 additive to improve workability.
On completion of pressure grouting the location of the bottom plate was again checked and found to be level to within 0.4mm (0.016 in.) as measured on the cylinder bearing overlay surface.

5.4. Prestressing

The vertical 19/18 prestressing tendons were fabricated alongside the chamber and pulled through temporary ducting up into ducts into the concrete structure. They were prestressed from their lower ends using a PBC T 700 prestressing jack.

The cement grout used to fill the ducts after prestressing had a water cement ratio of 0.45 and contained Chemical Building Productives additive No. 208 to minimise bleed. Two full height mock-ups of the ducts were used to develop and prove the grouting procedure and to identify potential difficulties.

The circumferential wire windings around the bottom cap and reaction ring were laid by a purpose-built Taylor Woodrow wire-winding machine designed to cope with the relatively small outside diameter of the chamber (11m, 36ft.). This machine was similar in all essential respects to those used for wire winding the gas-cooled reactors at Hartlepool and Heysham Nuclear Power Stations. [Ref. 4].

5.5. Inner Chamber Assembly

On completion of prestressing, four corbels were lifted into position and prestressed to the concrete structure. Erection of the high-level building steelwork and 120 tonne capacity E.O.T. crane is currently in progress.

When the crane is available for use, the cylinder, seals and model support plate will be lifted into position, followed by assembly of the closure system and chamber services. On completion of satisfactory commissioning trials of individual components and equipment the chamber will undergo its first proof pressure test and then be ready for operational service.

6. CONCLUSIONS

A wide range of challenging engineering problems have had to be faced and solved by both the design and construction teams engaged in the realisation of this exciting and unusual project which represents a significant advance in the integration of steel and concrete technology for pressure vessel construction.

The knowledge gained on this project will be extremely valuable in meeting demands for similar facilities which may arise in the nuclear and chemical fields and is already being used for offshore and seabed construction studies. The inherent redundancy of prestressed concrete is a particularly valuable feature, as it embodies an exceptional level of structural safety with an in-built environmental shield.

7. ACKNOWLEDGEMENTS

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Finally, he would like to pay tribute to his colleagues within TWC for their individual contributions and continued support throughout this project.
REFERENCES


5 Boiler and Pressure Code, Section VIII, Pressure Vessels; Division 2, Alternative Rules. American Society of Mechanical Engineers.


FIG. 1  SECTION THROUGH LARGE FATIGUE CHAMBER