THE DESIGN OF POD BOILER PRESSURE VESSELS
WITH PARTICULAR REFERENCE
TO HARTLEPOOL NUCLEAR POWER STATION

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ABSTRACT

This paper describes the design and development of the podded boiler type of pre-
stressed concrete pressure vessel now being constructed for both the Hartlepool and Heysham
Nuclear Power Stations in the United Kingdom. In this form of vessel two features of the
reactor layout govern the vessel design:

1. The adoption of a single channel fuelling arrangement with interstitial
   control rods and consequent close spacing of standpipes through the top
cap of the vessel.

2. The housing of the boilers in the wall of the vessel.

Prestressing of the top cap, therefore, has to be accomplished by circumferential
tendons located outside the perforated zone. The second feature restricts the space avail-
able for internal hoop tendons and therefore external wire winding was adopted.

The principal method used in analysing the Hartlepool vessel and model, based on
dynamic relaxation, is described. Various programs were developed in which plane stress and
strain solutions were expressed in cartesian and polar co-ordinates. Additional programs
using axi-symmetric and three-dimensional cylindrical co-ordinates were derived. An
ultimate load analysis is introduced which shows the response of the vessel to overload
pressures which would cause progressive cracking and rotation. The method is based on
finding the collapse mechanism associated with the minimum potential energy in the vessel.

Finally, the paper describes the experimental program which has been conducted on a
1/10th scale model concrete vessel of the Hartlepool design. The essential experimental
details, together with the degree of simulation achieved, are presented. The elastic
response, overload and ultimate load behaviour of the model are then correlated with two
methods of analysis described.

1. INTRODUCTION

The two nuclear power stations at present being constructed at Hartlepool and Heysham
in the United Kingdom have twin advanced gas-cooled reactors, designed to produce 660 MW per
reactor. They are being built by British Nuclear Design and Construction Limited for the
Central Electricity Generating Board and are the first commercial stations to use the podded
boiler concept. The boilers are housed within the pressure vessel wall and are removable
for maintenance if necessary. This advance in reactor layout has led to a more complex form of cylindrical pressure vessel which requires new methods of elastic and ultimate load design. The paper describes the basis of these methods and shows that they are in good agreement with experimental results from a model investigation.

2. VESSEL DESCRIPTION AND DESIGN CONSIDERATIONS

The pressure vessels are cylindrical in form with an external diameter of 85ft.(25.9m) and overall height of 96ft.(29.3m). The main pressure void is 45ft.(13.1m) in diameter and 60ft.(18.3m) high and encloses only the reactor core and its shielding (see Fig.1).

The boilers are contained in eight circular cavities of 9ft.(2.7m) diameter passing vertically through the full height of the vessel wall. They are linked by gas ducts to the top and bottom of the main void. Gas circulators are mounted below the boilers. An additional feature of the reactor layout is the use of single channel fuelling with interstitial control rods. The 324 fuelling standpipes are composed of three concentric tubes which form annular spaces for vermiculite insulation and cooling water. These standpipes are arranged on a square pitch of 16in.(458mm). The 81 control rod standpipes are similarly constructed and are placed centrally in patterns of four fuelling standpipes. These features of the reactor layout have influenced the vessel design. (see reference 3).

The choice of pile cap thickness and location and extent of prestress provided in the cap region of the vessel are governed by considerations of the ultimate strength of caps in flexure and shear. The small span of the main void in the pod-boiler vessels leads to a corresponding economy in the thickness of vessel caps.

The vessel wall thickness is controlled by the diameter of the boilers as well as that of the main void, and the most economical vessel is obtained by using a large number of small boilers. This minimises the thickness of the wall required and reduces the outside vessel diameter and the amount of circumferential prestress required. In the Hartlepool design, the final choice of wall thickness and the position of the eight boilers within the wall were mainly governed by the circumferential compressive stress arising between the boilers and the main void under conditions of initial prestress.

The circumferential prestress chosen for the Hartlepool vessel was designed and developed by Taylor Woodrow Construction Limited. It is based upon the use of concentrated bands of wire, wound under tension into channels preformed in the vessel walls. The wires are arranged in layers, each of which is separately anchored. Each layer is wound directly on to its predecessor in a carefully controlled way which ensures a regular arrangement of wires in the finished band. Continuous lengths of 0.2in.(5.1mm) diameter low relaxation wire sufficient for one complete layer of wire are produced by welding the rod stock prior to patenting. In the vessel each band will consist of approximately 35 layers of wire and exert a total force of approximately 9,500 tons (9,640 tonnes).

This system was developed partly because the interstitial control rods make it impractical to pass tendons through the standpipe zone; also the podded boiler layout leaves the outside of the vessel entirely free from major penetration. Wire wound tendons removed the need for ribs and anchorages, eliminated friction losses and improved the construction time.
The CCL system is used for the longitudinal prestressing of the vessels. Each tendon consists of 28 strands of 0.7 in. (18 mm) diameter, having a guaranteed ultimate strength of 1,040 tons (1,056 tonnes) and a working load of 780 tons (792 tonnes). About two-thirds of the longitudinal tendons are located in groups between the boiler pods, and a proportion of these deviate towards the vessel axis to maintain satisfactory levels of compressive prestress adjacent to the haunches of the main liner, and to improve the shear strength of the vessel caps. The remaining tendons are grouped around the boiler penetrations to provide maximum integrity for the boiler and circulator closure anchorages.

The gas ducts which connect the main and boiler pod liners have been designed to ensure compatibility of strain between the components. It is important to ensure that the compressive strain in the main liner is not concentrated into the area penetrated by the ducts. This is achieved by an array of anchors below the top ducts and above the bottom ducts.

The remainder of the paper explains in more detail the techniques which were used for the elastic and ultimate load design, and compares the theoretical results of the analytical methods with experimental results from a tenth scale model of the vessel.

3. ELASTIC ANALYSIS

The principal method used in analysing the Hartlepool vessel and model is based on dynamic relaxation (DR). Various programs were developed in which plane stress and strain solutions were expressed in cartesian and polar co-ordinates. Additional programs using axi-symmetric and three-dimensional cylindrical co-ordinates were derived.

DR is a finite difference method employing dynamic forms of the stress-strain equations and the equilibrium equations. The latter include damping terms which cause the structure to settle down to its static position. These damping terms are chosen so as to give an acceptable accuracy in a minimum computation time.

In cylindrical co-ordinates, the six basic stress-deflection equations are differentiated with respect to time. This results in equations giving the rate of change of stress in terms of velocities. The rate of change of velocities in terms of stresses is obtained by re-arranging the equilibrium equations. The resulting nine equations are changed into finite difference form, the position of the variables being on an interlacing net as shown in Fig. 2.

Special forms of the finite difference equations are required for boundary grid points in areas of varying elastic properties and where boundaries do not coincide with the grid system. (see reference 1).

3.1 Application of DR to pressure vessel design

The ideal design approach would be to carry out full three-dimensional analyses for all loading cases. This is impractical, except for the final design analysis, because of the large amount of data preparation and computer costs involved. Hence, great use is made of the more simple forms of DR. Two or more forms are used in conjunction to give reasonably accurate assessments of the vessel response. In addition, various 2 and 3-dimensional finite element analyses were carried out which could be compared directly with the DR analysis. Each of the design stages is dealt with below.
3.1.1. Standpipe Analysis: The standpipe region has effective elastic properties which depend on the position and orientation within the standpipe array. The overall vessel response depends on the mean stiffness of the standpipe zone and, therefore, a property termed the effective modulus was derived. The portion shown in Fig. 3 was analysed using cartesian co-ordinates under a uni-axial stress field, all four boundaries being constrained to remain straight. By knowing the applied stress and the deflections in the two orthogonal directions the effective $E$ and $\nu$ were obtained, say $E'$ and $\nu'$. For various reasons, it was decided that in the vessel analysis the value of $\nu$ in the standpipe zone would equal that for concrete, $\nu_c$. This resulted in a different value of effective $E$, namely $E''$, given by:

$$\frac{E''}{1 - \nu_c} = \frac{E'}{1 - \nu'} = \frac{KE}{1 - \nu_c}$$

Typical values of $E'$, $\nu'$ and $K$ are plotted in Fig. 3. An analysis was also carried out with loading at 45° to that above. Although producing different $E'$ and $\nu'$ values, the $K$ value was similar.

3.1.2. Analysis of boiler rod effects: Analysis of sections taken through the equator and cap under internal pressure, boiler pressure and prestress were carried out using polar co-ordinates. From the analysis of the equator section, average deflections at the inside and outside surfaces were obtained. It was then necessary to simulate this section with an axi-symmetric structure, such that the vertical direct and flexural stiffnesses would be little affected. In the axi-symmetric program it is possible to change the $E$ value from block to block and to designate zero hoop strength to any block. In practice it was found that, by varying the $E$ value of those blocks in the boiler region and giving one or more blocks zero hoop stiffness, an axi-symmetric structure was obtained which behaved very similarly to the actual structure.

A second result of these analyses was that more accurate assessments of the stress concentration effects were obtained since a finer grid could be employed than in the three-dimensional analysis.

3.1.3. Axi-symmetric and three-dimensional analysis: During the initial period of any design the basic parameters are liable to change rapidly. Hence, the axi-symmetric analyses, incorporating effective stiffnesses obtained as above were used. These runs enabled the designer to assess the effect of changing the vessel dimensions, prestress arrangements and thermal distributions.

Stress concentration effects around the haunch were also investigated. In successive analyses the grid size was made finer as the area of haunch examined was progressively reduced.

The final analyses carried out were full three-dimensional solutions of the top and bottom halves of a 1/16th segment of the vessel. A typical three-dimensional grid is shown in Fig. 4.

3.2 Creep

The effect of concrete creep on the stress re-distribution and vessel movements was examined by using appropriate values of modulus in the analysis previously described. The
E values selected were dependent upon the temperature distribution, the age at loading and its duration. The E value of each element is the reciprocal of the specific strain. Values of this parameter were obtained from a comprehensive experimental program into the creep of concrete at both ambient and elevated temperatures.

4. REVIEW OF DESIGN PROBLEMS

The stresses calculated using the various methods outlined in section 3 were judged in relation to the following allowable stress levels, where the cube strength \((U_w) = 7000\) lb./sq.in. \((48.3\) N/mm\(^2\)).

<table>
<thead>
<tr>
<th>Nature of Loading</th>
<th>Permissible Compressive Stress</th>
<th>Permissible Tensile Stress</th>
</tr>
</thead>
<tbody>
<tr>
<td>Construction, prestress, proof test</td>
<td>0.4 (U_w)</td>
<td>200 lb./sq.in. ((1.38) N/mm(^2))</td>
</tr>
<tr>
<td>Early and late life operation start up/</td>
<td>0.4 (U_w)</td>
<td>400 lb./sq.in. ((2.76) N/mm(^2))</td>
</tr>
<tr>
<td>shut down. Fault conditions.</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Local stress concentrations</td>
<td>0.8 (U_w)</td>
<td></td>
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<tr>
<td>Prestressing anchorage stresses</td>
<td></td>
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</tbody>
</table>

There are a number of zones in the pressure vessel where the analysis has shown that these allowable stresses were exceeded. These are:

a) In the centre line of each boiler at the equatorial plane. The hoop stresses adjacent to the boiler pod under the action of prestress only peak to values on the inner ligament of 3900 lb./sq.in. \((27\) N/mm\(^2\)) and on the outer ligament to 2900 lb./sq.in. \((19.3\) N/mm\(^2\)).

b) Around the boiler penetrations at the top corner of the vessel in the outer ligament. These stresses are also due to prestress load only and occur on the centre line of the boilers. The values peak to 3300 lb./sq.in. \((22.8\) N/mm\(^2\)) at the outer curved surface of the vessel and act in combination with a radial stress of 900 lb./sq.in. \((6.2\) N/mm\(^2\)) and a vertical stress of 800 lb./sq.in. \((5.5\) N/mm\(^2\)).

c) At penetration liner anchorages and discontinuities in the liner profile. The values of peak stresses occurring in such areas are difficult to predict realistically, but their influence on the vessel behaviour is considered to be small. Stress concentrations at penetrations are generally controlled by the choice of liner thicknesses and, in some cases, local bonded reinforcement is incorporated in areas where required.

The stress levels described in a) and b) above were considered acceptable because:

i) The liner and bonded reinforcement around the boiler was not taken into account in the analysis.

ii) The stress levels quoted are those immediately on transfer of the total prestress. Because of elastic deformation and creep, gradual losses occur as the prestress is applied over a period of months. In addition, further creep losses occur as the vessel is operated. Thus, the quoted levels diminish by approximately 10-15\% over the life of the vessel.

iii) The peak stresses occur on planes at the equator, or radially through the centre line of a boiler and diminish rapidly away from these locations.
iv) The stresses are combined with two other compressions or, for stresses immediately around the boiler, restraint is offered in the third direction by the liner and the curvature of the concrete profile. The authors have carried out considerable experimental work to examine the effect of high local stresses around the single penetrations. Field stresses of at least twice those normally used in vessel design did not appear to produce any significant structural damage in the area around a penetration. In addition, the diametral deformations were linear, even though elastic theory predicted that these stresses exceeded the linear elastic range for concrete.

A further area of complex local stress occurs in the haunch of the vessel, where the gas ducts are located. To analyse this zone, a three-dimensional analysis was first completed with the gas duct geometry not represented. General field stresses were then identified at the location of the gas duct and stress concentration factors for a single lined hole lying in an infinite plate were then applied.

The diameter of the gas duct was fixed from gas flow considerations, but the liner thickness of the penetration was chosen to withstand ultimate pressure at yield. In this situation, the stress concentrations do not exceed the general field stresses quoted previously. Thus, the major problem in this area was to predict the movements of the vessel concrete as these control the stresses in the steel liners, particularly at the junction of the duct and main vessel envelope.

5. **ULTIMATE LOAD ANALYSIS**

The purpose of this ultimate load analysis is to find the continuing response of the vessel to pressure after initial cracking and up to final collapse. It is based upon determining the mechanism associated with minimum potential energy in the vessel.

Various failure conditions, such as tendon and liner strain or concrete shear stress, are plotted against increasing pressure to examine the progressive failure. Failure is assumed to occur when any of the individual parameters exceeds its relevant criterion.

Two fundamental assumptions about the behaviour of the cracked vessel are made. These, together with the empirical values used for the material properties, are examined.

5.1 **Principles of Analysis**

The method of analysis is based upon the principle of stationary potential energy (\(U\)), which states that of all geometrically possible deflected shapes, the structure will adopt the one with minimum potential energy.
Thus for equilibrium:
\[ \frac{\partial U}{\partial x} = 0 \]
where \( x \) is the displacement function

Therefore,
\[ \frac{\partial^2 U}{\partial P \partial x} \cdot \frac{\partial P}{\partial x} = 0 \]
where \( P \) is the internal pressure load

The principle is, therefore, satisfied if:
\[ \frac{\partial P}{\partial x} = 0 \]

By plotting \( P \) against assumed hinge positions \( x \) inches from the equator, a value of \( x \) is found such that \( P \) is minimum for a given deflection at the equator.

The number of geometrically possible mechanisms may be reduced by considering the symmetry of the vessel, which is assumed to split into a shape indicated in Figure 5. Only a sixteenth segment bounded by the vessel equator and two boiler pods need be considered.

The first fundamental assumption is that a horizontal crack forms at the equator on the outside, to form a "plastic hinge". A second inclined crack forms from the inside of the haunch between the barrel and cap, and finishes at some height \( x \) above the equator. The end of this crack is the neutral axis of a second "plastic hinge" about which the barrel and cap rotate.

The second assumption is that the vessel deflections are rigid-plastic after cracking. Elastic changes of strain are then ignored compared with rigid-body movements of the barrels or cap segments. The net strain value at a cracked section is the sum of the initial prestress strain, less losses, plus elastic strain recovery up to the elastic limit of the vessel, plus strain increase due to rigid-plastic movement after cracking.

On the basis of these two assumptions, the vessel deflections and strains may be defined at any point, in terms of \( x \) and the barrel and cap deflections, \( \Delta B \) and \( \Delta C \). By using suitable stress/strain relations for the material properties, these strains may be converted into forces acting at the boundaries of the sixteenth segment, which are used to form equilibrium equations for the barrel and cap. These are solved separately for \( PB \) and \( PC \), the pressures acting on the barrel and cap portions of the sixteenth segment. As \( PB \) and \( PC \) are both functions of \( x \), \( \Delta B \) and \( \Delta C \), it is convenient to temporarily fix two variables, \( x \) and \( \Delta B \), and to vary \( \Delta C \) iteratively until \( PB \) and \( PC \) are equal. This ensures that a single compatible pressure acts in the vessel and that any temporary boundary conditions assumed between barrel and cap are satisfied.

The variable \( x \) is then altered until a minimum compatible pressure \( P \) is found. Figure 6 shows \( x \) plotted against \( P \) for the Hartlepool vessel model. The third variable \( \Delta B \), which corresponds to strain in the hoop tendon at the equator, is successively increased and the whole process repeated. It can be seen for the model, that \( x = 45 \) inches corresponds to minimum pressure, and that this value of \( x \) increased slightly for higher values of \( \Delta B \).
It will be shown that the plastic hinge moments are themselves functions of the internal pressure, so that a further iterative process is necessary for finding the depth to the neutral axis.

The flow diagram for the computer program (Fig.7) explains the logic for the analytical procedure. The plots of pressure against increasing deflections, shears and strains (Fig.18, 19, 20) are shown only for the mechanism with minimum pressure. By invoking the principle of minimum potential energy and accepting the assumptions, this will be the mechanism the structure will adopt. Section 8 examines some of the effects of the two assumptions.

5.2 Material Properties

For the bonded steel, a typical crack width/stress relationship of high yield steel is shown in Fig.8. Using this relationship, strain hardening can be accounted for; failure of a bar is assumed if the width of a crack exceeds some arbitrary limit - say 2 in. (50 mm).

The stress-strain curves for the hoop and vertical tendons are stored in the computer by defining the stress and strain at eight points on the curves. As the vertical tendons are un-grouted, it is assumed that the tendons are free to strain over their entire length. For the hoop tendons, friction between the tendon and vessel modifies the stress in the tendon for a given average hoop strain.

5.3 Plastic Hinge Moments

The value of bending moment at a cracked section or "plastic hinge" is based upon two assumptions:

a) Plane sections remain plane and rotation occurs about the neutral axis.

b) The compressive stress block is rectangular. The stress level is based upon tests by Hognestad, Hanson and McHenry (ref. 2).

The forces acting across the plastic hinge and the expression for the plastic hinge moment are shown in Fig.9. The plastic moment (M) is a quadratic function of the total compression (N) across the section. As N is the net compression from prestress and internal pressure, M is also a quadratic function of the pressure in the vessel. M does not reach any given "plastic" value, but constantly alters as the internal pressure and deflections of the vessel increase. This expression for M is valid, provided the section is under-reinforced and not in net tension. A check that the depth to the neutral axis is between zero and 30% of the total depth of the section is, therefore, made.

6. MODEL VESSEL DESCRIPTION

Whilst the cylindrical pressure vessel is not uncommon, the type developed for the Hartlepool and Heysham Nuclear Power Stations contained sufficient advanced features to justify undertaking a program of research. The major item of this program was the construction and testing of a tenth scale concrete model of the complete vessel.

The primary objects of this model work were to observe the performance of the structure as cracking developed, to demonstrate the mode of failure and to establish the minimum ultimate load factor. In addition, adequate instrumentation was included to provide strain and deflection information during prestressing and under design pressure loading.
Since the model was constructed in 1966, there have been some modifications incorporated into the vessel design. However, comparison of Figs. 1 and 10 shows that the model is still substantially representative of the current design.

The significant differences between model and prototype are:

i) The standpipes in the model each incorporated external lobular cooling tubes. These produced a more severe stress condition in the model cap, since the lobes acted as local stress raisers. The prototype standtubes incorporate annular cooling tubes.

ii) The interior of the model had 45° haunched corners, compared with square corners in the prototype. Both forms of corner generate local stress concentrations under pressure loadings and first flexural cracks initiate from these regions when the vessel is subject to overpressure. The development of these cracks is subsequently governed by the overall geometry and applied forces.

iii) A 14in. (356mm) internal diameter hole was included in the bottom cap of the model for construction access. This was placed centrally and had a steel liner with a wall thickness sufficient to compensate approximately for the concrete displaced. The cavity was not pressurised.

iv) The standpipe array was twice the model scale to facilitate construction whilst maintaining a similar relative rigidity.

The model was constructed from limestone concrete with 3/8in. (9mm) maximum aggregate size; the average cube strength was 8800 lb./sq.in. (60 N/mm²). Because of the degree of construction in the pile cap zone, the aggregate size was dropped locally to 1/4in. (6mm).

Representative reinforcement was provided in the form of ribbed high tensile bar. The eight boiler pods and gas entry ducts had representative steel liners.

Vertical prestress was applied to the model by 240 pairs of 0.276in.(9mm) diameter wires, each pair simulating a full scale tendon. Hoop prestress consisted of 0.104in. (2.6mm) diameter wire which had a similar stress/strain characteristic to the prototype 0.2 in (5mm) diameter wire. The vessel was rotated on a turntable and drew wire from a pre-wound drum. Tension was produced by braking the drum with an electro-magnetic clutch and controlled by monitoring pre-calibrated load cells anchored below the drum housing.

Initially, 700 transducers were included in the model to record the behaviour under test. The general disposition of these is shown in Fig.10. The main strain profiles were measured by electrical resistance strain gauges embedded within the concrete adjacent to the vessel surfaces. Supplementary vibrating wire gauges were included at critical sections, i.e. the haunches and at the equator. Further resistance gauges were bonded on two boiler pod liners, four standtubes and on reinforcing bars at the critical tensile zones. Copper-constantin thermocouples were distributed throughout the model to provide information for any thermal correction required to the recorded strains. Load cells were incorporated at each end of ten typical vertical tendons to monitor changes in anchorage load. Overall deformations were measured externally by dial gauges and linear potentiometers mounted on an independent reference frame.

Pressure loading was applied hydraulically to the main cavity and boiler pods. Reinforced rubber liners were used to prevent premature leakage through small cracks.
Between the design pressure of 644 lb./sq.in. (4.4 N/mm²) and 1000 lb./sq.in. (6.9 N/mm²) small increments of pressure were applied in order that the "elastic limit" and initiation of cracking could be defined. Successive tests in this range were arranged such that any one pressure was achieved at least three times before progressing to a high value. Above 1000 lb./sq.in., larger steps in pressure were taken in successive tests, and a nominal pressure of 50 lb./sq.in. (0.34 N/mm²) was maintained at all times to ensure full contact of the lining membranes with the vessel surfaces.

7. ELASTIC BEHAVIOUR - CORRELATION OF EXPERIMENTAL AND THEORETICAL RESULTS

An accurate knowledge of the elastic modulus (E) of the concrete is essential for translating theoretical stresses into strain for the purpose of correlation with experimental data. To obtain this information, a large number of control cylinders were cast during the model construction. These were loaded cyclically to a stress of 3000 lb./sq.in. (20 N/mm²) at the time of prestressing. The value of E varied from 5.03 to 5.75.10⁶ lb./sq.in. (34.5 to 39.5.10³ N/mm²) and a mean figure of 5.38.10⁶ lb./sq.in. (37.10³ N/mm²) was initially taken for the correlation of theoretical with experimental results. Poisson's ratio was measured and found to 0.21.

This value of E derived from the control cylinders was substantially lower than the value of 6.3.10⁶ lb./in.² (43.4.10³ N/mm²) from the Wyfia 1/12th scale model which was constructed from a similar mix. The theoretical strains based on the lower modulus were, therefore, compared with experimental strains when the model was subject to design pressure. This comparison is presented as a histogram of percentage variation between observation and theory in Fig.11. The histogram indicates that compatibility of the two variables requires a modulus of 6.2.10⁶ lb./in.² (42.7.10³ N/mm²). Since this modulus was derived from insitu tests on the actual model, it was considered the most realistic and, therefore, employed in the overall correlation. A value of 4.28.10⁶ lb./in.² (29.5.10³ N/mm²) was adopted for the perforated region of the pile cap to simulate its lower stiffness.

Possible reasons for the difference in E value between the model and control cylinders may depend on their relative sizes. Drying conditions and the moisture state would be affected as well as the temperature at early life due to heat of hydration. In addition, the model concrete was generally under triaxial stress conditions.

7.1 Behaviour under Prestress

Initially, it was hoped to measure the strains in the model induced by the hoop windings at 1/3, 2/3 and full hoop prestress. However, during the period taken to wind the first increment of prestress, in addition to creep strains, random errors were produced by disconnecting and re-connecting the cables to allow rotation of the model.

On completion of prestressing, the two hoop bands nearest the equator were cut off, strain readings being taken immediately before and after. Results obtained from this "instantaneous" prestress test are shown in Fig.12. It was possible to relate these results directly to the elastic analysis without consideration of creep or thermal corrections.

In general, the observed hoop and longitudinal strain profiles are similar to, but marginally greater than those predicted. The distribution of longitudinal strain is very similar for sections through and between the radial centre lines of boilers. Local effects
adjacent to the points of application of prestress were predicted, but there were insufficient gauges positioned to record them in detail.

7.2 Behaviour under Pressure 0–1000 lb./sq.in. (6.9 N/mm²)

The model was sensibly linear and elastic up to a pressure of 1000 lb./sq.in., all strain and deflections showing complete recovery on each depressurisation. There was no visible external cracking below 1000 lb./sq.in., and no significant changes in vertical tendon anchorages forces were recorded.

The design pressure of 644 lb./sq.in. (4.4 N/mm²) was achieved 13 times before loading beyond the "elastic" limit of the model. Strain profile data obtained from these tests has been averaged and compared with theory in Figs. 13 and 14. The only significant variation from predicted behaviour occurs in the external measured hoop strains near the bottom cap because of the assumed symmetry of the vessel in the analysis. The deflected profile of the model shown at design pressure in Fig.15 provides an indication of the overall stiffness of the structure compared with that predicted.

The profiles of the hoop and radial strains recorded by the embedded gauges near the upper surface of the top cap in relation to theory are shown in Fig.16. Differences arise because the analysis uses an effective modulus for the standpipe zone and makes no allowance for the stress concentrations set up by the standpipes. For the measured radial deflections the gauge length was sufficiently long for the deflections to be largely unaffected by stress concentrations. In Fig.17, theoretical values are shown for effective moduli equal to 55, 70 and 85% of the concrete modulus. The experimental points all lie within the two outer limits and it can be seen that the 70% value of effective modulus adopted represents a reasonable mean for the overall vessel analysis.

Strains recorded by vibrating wire gauges situated at the inner face haunches are given in Fig.18. These indicate that stress concentration cracks developed in these regions above design pressure, initiating at the haunch corners nearest to the barrel. At 880 lb/sq.in. (6.1 N/mm²) (1.37 times design), these cracks had a maximum estimated width of only 0.001 in. (0.025 mm).

The development of equator and top cap deflections with pressure, given in Figs. 19 and 20 respectively, illustrate that the haunch cracks had no significant effect upon the linear response below a pressure of 1.56 times design.

8. ELASTO-PLASTIC BEHAVIOUR – CORRELATION OF RESULTS

The following discussion relates to the correlation between the overload behaviour of the 1/10th scale concrete model described in section 6 and the ultimate load analysis developed in section 5.

8.1 Model behaviour at pressures in excess of 1000 lb./sq.in.

There were no major divergencies from the general linear behaviour of the model until the pressure exceeded 1200 lb./sq.in. (8.3 N/mm²). The progressive development of the internal haunch cracks with increasing pressure resulted in a reduction in the magnitude of the rotation of the corner of the top cap towards the centre. Between 1200 and 1610 lb./sq.in. (2.5 times design), the edge of the cap developed a significant outward rotation about a point in the barrel 12 in. (305 mm) below the upper surface of the cap. (fig.21).
The complete deflected profile at 1610 lb./sq.in. (11.1 N/mm²) is compared with that at design pressure in Fig. 22; the predominance of the flexure at the top cap is becoming apparent. Peak deflections at both the equator and centre top cap trebled in value over this pressure range.

First horizontal flexural cracking was observed at 1400 lb./sq.in. (9.7 N/mm²) in the barrel immediately above the equator. At 1610 lb./sq.in. longitudinal cracks had developed on the centre lines of all boiler pods. These cracks were visible on both surfaces of the barrel, extending internally to within 4 in. (100 mm) of the gas duct, and to a lesser extent externally. The full extent of the barrel cracks at 1610 lb./sq.in. is shown in Fig. 23; they are slightly non-symmetrical about the equator due to the higher stiffness of the bottom cap. Adjacent to the equator, these cracks were of sufficient magnitude to cause local yielding of the reinforcement. Typical strains measured on the hoop steel are shown in Fig. 24 compared with those observed in the adjacent concrete. All the prestress remained well below yield and the residual cracks on depressurisation were not of measurable width.

Internally, the strains measured at the haunches indicated a maximum crack width of 0.008 – 0.010 (0.2 – 0.25 mm) at 1610 lb./sq.in. There were no visible cracks in either surface of the top cap, but small radial cracks were visible on the outer surface of the bottom cap extending from the manhole to the inner ring of vertical prestress.

1610 lb./sq.in. was achieved six times before progressing higher, and, since this cycling advanced the flexural cracking, the overall deflections of the model increased. Subsequent pressurisation to 1932 lb./sq.in. (13.5 N/mm²) further advanced the observed mode of failure, the crack pattern developed being substantially that assumed in the ultimate load analysis. First flexural cracking occurred in the pile cap at 1800 lb./sq.in. (12.4 N/mm²).

8.2 Correlation

The ultimate load analysis gives the following information, which is discussed in relation to the model behaviour:

8.2.1. Hinge position at barrel cap connection: The relationship between the hinge position and pressure for different mean strain values at the equator is shown in Fig. 6. For pressures in excess of 1200 lb./sq.in., a series of deflection gauges were mounted on the model at close centre along the length of the barrel where the hinge was expected. Analysis of the observed deflections showed a minimum value to occur 12 in. below the top corner of the vessel. This was within 0.5 in. (12 mm) of the predicted value.

8.2.2. Deflections at the pole and equator: The theoretical deflections are shown in Figs. 19 and 20 in relation to the experimental values. At the maximum observed deflection, the theoretical and experimental pressures agreed within 4%.

8.2.3. Mean tendon strain: These are illustrated in Fig. 24.

8.2.4. Crack sizes: The crack width to stress relationship of the bonded steel was assumed to be as shown in Fig. 8. The theoretical crack sizes are plotted against pressure in Fig. 25. It may be seen that the equatorial crack on the outside of the vessel was predicted to be 0.033 in. (0.84 mm) at 1610 lb./sq.in. pressure. The measured crack sizes were 0.010 in. (0.25 mm) at the haunch for this pressure, but it was found that the vertical
bonded steel distributed the cracks and that, on average, there were three roughly parallel cracks. In the prototype, it is not expected that this distribution will be effective after substantial yielding of the steel. Although the reinforcement in the model was the same type as that in the prototype subsequent tests showed that, for the size of bars used in the model, there was no precise yield point. This may explain why the distribution of cracks occurred in the model after the yield stress was reached.

Experimentally, it was not possible to determine the crack width stress relationship, as the 10 mm gauges averaged the peak strain in the bar. Also the gauges were not generally coincident with the cracks formed in the concrete.

8.2.5. **Barrel shear**: The net shear force at the hinge position was determined from the total forces acting on the barrel element. Any out-of-balance was assumed to be resisted by shear across the compression zone. The maximum shear resistance has been assumed as that which would produce a principal tensile stress of 400 lb./sq.in. in the compression block under an average vertical stress of 4200 lb./sq.in. (29 N/mm²). This calculated shear resistance was exceeded at pressures greater than 1600 lb./sq.in. On increasing the pressure in the model to 1932 lb./sq.in., at which time the lining membrane failed, horizontal jets of water were observed at the lower edge of the second band of prestress. This indicated the existence of a second horizontal crack and that the shear criterion adopted was conservative.

The ultimate load program was also used to study the contribution of the hoop prestress, hoop reinforcement and the moments at the equator and haunch in resisting the internal pressure. The percentage contribution offered by these four components at four different pressures is shown in the table below.

<table>
<thead>
<tr>
<th>Pressure (lb./in²)</th>
<th>1090</th>
<th>1378</th>
<th>1621</th>
<th>2015</th>
</tr>
</thead>
<tbody>
<tr>
<td>N/mm²</td>
<td>7.5</td>
<td>9.5</td>
<td>11.2</td>
<td>14.0</td>
</tr>
<tr>
<td>Hoop prestress</td>
<td>84</td>
<td>68</td>
<td>62</td>
<td>60</td>
</tr>
<tr>
<td>Hoop reinforcement</td>
<td>3</td>
<td>17</td>
<td>23</td>
<td>25</td>
</tr>
<tr>
<td>Equator moment</td>
<td>6</td>
<td>9</td>
<td>9</td>
<td>9</td>
</tr>
<tr>
<td>Haunch moment</td>
<td>7</td>
<td>6</td>
<td>6</td>
<td>6</td>
</tr>
</tbody>
</table>

8.3. **Examination of Assumptions**

The assumption that a single hinge would form near the equator was examined by altering the program to analyse the barrel as a four hinge mechanism, with the two central hinges at a distance Z either side of the equator. For all vessels analysed, it has been found that the mechanism with least potential energy has the value of Z = 0. This has also been confirmed by test results on cylindrical pressure vessels.

The second assumption of rigid plastic deflection implies that, after cracking, all rotation occurs at the neutral axes of the plastic hinges, and that elastic flexing of the concrete is small. Examination of the deflected profiles at pressures in the range 2½ to 3 times design pressure showed that the barrel deformed essentially as rigid elements with a sharp change of deflection slightly above the equator and approximately 12 in. from the outer corners of both end caps. The sensitivity of the analysis to the assumed value of compressive stress in the hinge was examined. Variation of the stress from 1250 to 5000 lb/sq.in. (8.6 to 34.5 N/mm²) only changed the ultimate pressure by 1%. 
9. CONCLUSIONS

9.1 The 1/10 scale model of the Hartlepool vessel was shown to behave elastically up to a pressure of about 1.5 times design and to have an ultimate load factor greater than three.

9.2 The general mode of collapse derived from the ultimate load analysis agreed with that developing in the model vessel. The deviation of the crack pattern from the equator is not considered to have a significant effect upon the failing pressure.

9.3 The degree of correlation between model and analysis both in the elastic and overload stages was such that the methods developed could confidently be applied to the design of the full-scale podded boiler pressure vessels.

REFERENCES


ACKNOWLEDGMENTS

The authors wish to thank Taylor Woodrow Construction Limited for the permission to produce this paper. They are also grateful for the help and encouragement received from their colleagues in the Research Laboratory, Pressure Vessel Section and the Computer Application Group.
1. ANCHORAGES
2. GAS DUCT
3. BOILER POD
4. GAS DUCT
5. SHIELD WALLS
6. HOOP PRESTRESSING CHANNELS
7. TENDONS
8. SUPPORT WALLS

1. Hartlepool N.P.3. Pressure Vessel
At time $t$ the variables are:

- $A$: $\overline{rr}_{ijkt}$, $\overline{θθ}_{ijkt}$, $\overline{zz}_{ijkt}$
- $B$: $\overline{rθ}_{ijkt}$
- $C$: $\overline{rz}_{ijkt}$
- $D$: $\overline{zθ}_{ijkt}$
- $E$: $\dot{u}_{ijkt}$, $\nu_{ijkt}$, $p_{ijk}$
- $F$: $\dot{v}_{ijkt}$, $\nu_{ijkt}$, $q_{ijk}$
- $G$: $\dot{w}_{ijkt}$, $\nu_{ijkt}$, $s_{ijk}$

2. Dynamic Relaxation Grid Element

CONCRETE MODULUS $E_c (x \, idf)$

- $E_c$ = Young's Modulus of Concrete
- $E'$ = Effective Modulus
- $v'$ = Effective Poisson's
- $K$ is derived from formula $\frac{E'}{1-v'} = K = \frac{K E_c}{1-v}$

3. Effective Properties of Standpipe Region
4. Three Dimensional Grid

5. Mode of Cracking
Figure showing the position of a crack in a model, with a graph illustrating the relationship between pressure (lb/in²) and hinge position for a given equatorial strain. The graph includes a scale model labeled as a 1/10th scale model. The text at the bottom reads: "Height of Hinge v. Pressure."
Flow Diagram for Method of Solution
8. Assumed Crack Width v. Stress for H.T. bar
9. Forces on hinge

10. G.A. of model
11. Histogram of Strain Results

12. Hoop strains due to prestress

E = 5.38 x 10^6 LB/SQ.IN
13. Hoop strains due to 640 lb/sq.in. pressure

14. Longitudinal strains due to 640 lb/sq.in. pressure
15. External deflections at 640 lb/sq.in pressure

16. Strains in Top Cap at 640 lb/sq.in pressure
17. Radial Deflections across Top Cap

18. Observed Haunch strains v. pressure
19. Equator Deflections v. pressure

20. Top Cap Deflections v. pressure
III. Observed Top Corner rotation v. pressure

22. Deflections at 1610 lb/sq.in. pressure
Band widths not to scale

23. Crack pattern at 1610 lb/sq.in. pressure
24. Strains in Hoop Steel

25. Theory Crack Width v. pressure
DISCUSSION

Q  P. BINDSEIL, Germany

1. Comment on what Mr. Langan said on mode of failure. For THTR we assumed the mode discussed by Mr. Schwiers. The crack pattern is given by the two levels of horizontal tubes in the cylinder of the THTR-vessel. The pattern has recently been proven by a high-pressure test.

2. Do you mean by elastic limit (1.5 po) the limit of linear behaviour and when yes, did you investigate the behaviour of deflections and strains during unloading after a maximum pressure higher than this linear-limit pressure?

I agree to the guess of Mr. Langan, that the overall elastic limit is in a region of 2.0 po.

A  D. LANGAN, U. K.

1. I would agree that the presence of two rows of major penetrations must influence the position of the crack pattern in the barrel.

2. The elastic limit (by which we mean point of non-linearity) is difficult to establish, but could be examined at two levels, i.e.:

   a) local cracking which produces non-linear strain,
   b) deflections and strains throughout the general body of the vessel.

In this case pressure cycling up to 1.5 design pressure does not produce significant plastic set or residual deformation.

Q  K. MEERWALD, Germany

I would like to have some definition of "linearity" and "limit of elasticity". What is your definition related to your measurement values of the models?

A  D. LANGAN, U. K.

I believe the question was answered by my response to the previous speaker. Briefly, local cracking is irreversible and strains at the cracks become non-linear. The general behaviour, however, is unaffected by local cracking and therefore non-linearity only takes place when the cracking is so extensive that edge flexibility conditions for the barrel and cap change significantly.