

**PRESTRESSING AND PROOF PRESSURE TESTING OF THE WYLFA  
PRESTRESSED CONCRETE PRESSURE VESSELS AND COMPARISON OF  
MEASURED DATA WITH PREDICTED PERFORMANCE**

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ABSTRACT

The performance of the two Prestressed Concrete Pressure Vessels built for Wylfa Nuclear Power Station, when subjected to prestress and proof pressure loadings, provided the first comprehensive opportunities to judge both the design assumptions and the analysis employed.

The analytical approaches adopted for the vessel design and the background experimental work undertaken to establish the necessary basic parameters of the concrete and prestress system, are described in this paper. Brief details are also given of the insitu tests which confirmed the design values of the parameters.

Each of the vessels was provided with extensive instrumentation to observe strains, deformations and temperatures, and these were monitored during prestressing and the cold proof pressure tests. This paper examines the results of all this work in comparing the behaviours of both vessels with each other and with the analytical predictions. Sources of error are identified and discussed. General conclusions are drawn on the overall degree of correlation achieved.

## 1. INTRODUCTION

The Nuclear Power Station at Wylfa is the last of the series built in the United Kingdom which employ the Magnox reactors. Its construction was undertaken by the English Electric, Babcock and Wilcox, Taylor Woodrow Construction consortium.

The two prestressed concrete pressure vessels which house the reactors remain the largest to be constructed and are unique in that their internal surfaces are spherical.

Their final design was the outcome of an extensive programme of research and development which encompassed investigations into concrete properties and prestressing components, several model studies and the development of analytical methods. Many aspects of this programme were presented in papers at the 1967 conference on vessels held in London [1].

Construction of these vessels commenced in 1964, the first (PV1) being prestressed between February and June 1968 and the second (PV2) between August and November 1968. The cold proof pressure tests were satisfactorily completed in December 1968 and June 1969 respectively. Because of the unique nature of these structures, each was comprehensively instrumented in a similar manner and this permitted detailed examination of their performance under the two significant early loadings of prestress and proof pressure.

Following a description of the vessels and the relevant development work the paper assesses the results of this examination in relation to the behaviour predicted for the vessels at the time of design.

## 2. DESCRIPTION OF VESSELS

The internal surface of the pressure vessel is formed by a spherical steel liner of 96 ft. (29.26m) internal diameter and 0.75in. (1.9cm) thickness, anchored to the concrete vessel by means of angled stiffeners. The external surface is generally in the form of three concentric vertical cylinders of 71 ft. (21.64m), 102.33 ft. (31.19m) and 118 ft. (35.97m) diameter respectively. For practical convenience the vertical cylindrical faces are formed as a series of flat surfaces between a series of 16 vertical ribs which are symmetrically arranged to carry the external circumferential hoop tendons and anchorages. The ribs are generally 8 ft. (2.44m) wide and project 8 ft. (2.44m) from the face of the vessel.

The vessel shell has to accommodate certain penetrations the most important of which are as follows:-

- i) 397 vertical stand pipes of 19.25in. (49cm) outside diameter and generally on a square grid of 31in. (79cm) pitch.
- ii) Four main circulator penetrations of 10.25 ft. (3.12m) inside diameter, located horizontally and symmetrically about the vessel centre line.

The prestressing system is based on a modified Freyssinet 12/0.6 in. (1.52cm) strand anchorage. Each tendon is formed by 36 stabilized strands which separate into 3 groups at the anchorage. The design anchorage force after transfer is 600 ton (609.6 tonnes) per tendon.

The tendons are distributed in four main groups on each vessel as follows:-

1. An orthogonal small circle system of 218 internal tendons in five layers passing through the stand pipe zone of the top cap. Outside the stand pipe zone the tendons deviate to anchor on vertical surfaces all approximately of uniform spacing around the periphery of the vessel.
2. A similar tendon pattern in the bottom cap of the vessel, with minor deviations about the larger service penetrations.
3. A great circle system of 528 meridional internal tendons in eight layers, generally lying in equally spaced planes with local deviations about circulator penetrations. At the top and bottom of their paths 480 of these tendons cross through the cap small circle systems and anchor on horizontal annular surfaces. The remaining 48 are straight vertical tendons which anchor on the top and bottom surfaces of the main vessel ribs.
4. An external system of 384 hoop tendons, generally in four layers, which apply prestress to the vessel through the 16 vertical ribs and are arranged in 24 bands of 16 tendons. Each tendon passes approximately 180° around the vessel and is paired with the corresponding tendon covering the remaining 180°.

The overall form of the completed vessel is shown in figure 1. The design pressure was stipulated to be 423 lb/in<sup>2</sup> (29.74 kgf/sq.cm) and the internal surface concrete temperature was 35°C. The outside surface temperature varies and whilst the general crossfall is of the order of 15°C, there is a reverse crossfall of 5°C in the pile cap zone.

### 3. VESSEL ANALYSES

#### 3.1 Analysis Developed for Design

The analysis of the vessel was based on flexibility methods commonly employed for statically indeterminate structures. The vessel was divided

into a series of rings and flexibility coefficients calculated for unit forces acting on the face or in the body of each ring. The continuity forces between each ring were derived by equating the deformations developed at the common boundaries. By combining the continuity forces and statically determinate loads to give the direct load, moment, and shear forces at each boundary, the stresses at these sections could then be determined. [2]

The structural calculations took account of the geometrical shape of the vessel, the actual arrangement of prestressing tendons, the temperature regimes, dead loads and construction loads, prestressing losses in the tendons, relative shrinkage, and internal pressure.

Whilst this analysis was basically axisymmetric the ribs could be included by ignoring any contribution to the hoop stiffness but allowing for them as contributing to the longitudinal stiffness. Penetrations which were adequately lined to provide compensation could be ignored. In regions where this was not possible allowance was made for their presence by adjusting the modulus of elasticity and/or applying a factor to achieve the correct hoop stiffness. In the case of the stand pipe area an equivalent modulus is used to achieve the correct stiffness.

### 3.2 Analysis used for Verification - Dynamic Relaxation

The method of dynamic relaxation permits the solution of most elastic problems provided that the structure can be represented by some regularly defined grid. Using this grid the differential equations of motion and stress/strain relationship are expressed in finite difference form. The equations of motion contain a damping term and by iterating through both sets of equations at discrete time intervals it is possible to arrive at the static equilibrium condition. [3] The method was not fully developed until after the contract design was completed and therefore, it was only possible to use this as verification.

In the case of the Wylfa vessel three types of grid were used:-

- i) Polar coordinates - used in plane stress or strain calculations in assessing hoop stress distribution on an equatorial section.
- ii) Axisymmetric cartesian coordinates - this adequately represented the external surface of the vessel but did not conveniently describe the spherical interior. It was most convenient for coping with prestressing effects.
- iii) Axisymmetric polar coordinates - this provided a better representation of the internal surface, with a corresponding reduction in useful definition of the exterior.

A satisfactory description of the vessel behaviour under any loading could be obtained by judicious combination of the results of these three analyses.

#### 4. CONCRETE PROPERTIES

##### 4.1 Properties used for the Vessel Design

Since the concrete in the vessels is structural and integrally related to a complex mechanical system, more precise knowledge was required of its properties and performance for the design than was available at that time. Extensive experimental programmes were, therefore, undertaken to provide a rational selection of a concrete with properties suited to the operational requirement of the vessels [4]. The data obtained on properties was then used in the vessel analysis at the design stage.

The parameters required for correlation purposes in the two loadings considered in this paper were:-

- a) Instantaneous elastic modulus  
( $5.0 \times 10^6$  lb/in<sup>2</sup>). ( $0.35 \times 10^6$  kg/sq.cm.)
- b) Poisson's ratio (0.15).
- c) Specific creep at ambient temperature.

The figures in brackets are those used in the original stress analyses. Specific creep varies with both the age of the concrete at loading and the time under load. The relevant data was therefore extracted from prepared design curves.

##### 4.2 Confirmatory Evidence on Concrete Properties from Vessel Concrete

The values of parameters determined from control specimens require informed interpretation to establish the equivalent values in the mass concrete. However, test data from control samples of the actual concrete placed may usefully be compared with that from laboratory trials providing the sampling procedure is similar.

Concrete specimens were therefore cast regularly from the vessel concrete as it was placed. For verification of elastic modulus and creep, 60 cylindrical specimens were taken from each vessel, distributed uniformly between the barrel section and each of the end caps. Since it was hoped to gain information on the early age behaviour of the mass concrete in the vessels from the embedded strain gauges, control sampling was also restricted to the bays which contained instruments. It was expected that this procedure might improve on the representativity of the control data.

The specimens were subjected to elastic modulus tests at various ages and subsequently held under constant load to determine the creep rate at ambient temperature. Creep tests were continued until the proof pressure

tests on both vessels were completed, and the resultant data generally compared favourably with that obtained from laboratory trials. There was, however, a general tendency for the elastic modulus of the concrete to increase with age, and it was not certain whether this effect would be offset in the larger mass of the vessel concrete by the greater influence of the heat of hydration cycle. The strains predicted for the proof pressure tests were therefore calculated for a range of elastic modulus values, and the final value selected during each test by a study of the strain behaviour in statically determinate regions of the vessels.

## 5. INSTRUMENTATION OF VESSELS

Because of the unique nature of these structures it was decided that both should contain a similar number and disposition of sensors to record strain, movement and temperature for monitoring the performance of the vessels during construction, proof testing and commissioning.

### 5.1 Vibrating Wire Gauges

A total of 426 vibrating wire gauges were embedded in the concrete of each vessel during the course of construction. The majority of these were in pairs having the same location and orientation; one gauge acting as either a check or, in the event of failure, a replacement for the other. All gauges incorporated electrical resistance thermometers. The disposition is best described in relation to the two groupings into which they were divided for correlation purposes, and by reference to fig. 2.

- a) A primary group of 78 pairs spaced to the internal and external surfaces and arranged to measure circumferential and meridional strains.
- b) A secondary group of 270 gauges whose main purposes were to provide information on strain around penetrations and also to check the symmetry of the behaviour around the equator.

The gauges were manufactured to a rigorous specification which required that their life expectancy should be at least that of the vessel itself, and that there should be a low tolerance on the gauge factor. The actual gauge factor could only be determined experimentally and tests suggested that the variation could be  $\pm 10\%$ . [5]

### 5.2 Vessel Movements

Measurements of vertical and horizontal movements of the vessel with respect to the Reactor Hall basement were made at plates fixed to the vessel external surface. Fixed datum points were located in the basement of the Reactor Hall and plates were provided on the outside face of 11 of the 16 main ribs at 7 levels, and also on 15 of the 16 lower small ribs at 5 levels.

The vertical movement of a plate was measured by means of an invar tape. The lower end of the tape was attached to a socket in the vessel floor and the upper end passed over a pulley and was weighted. The tape was read against a removable pointer fixed into the plate on the vessel rib.

In order to measure both radial and tangential movements targets were placed in the fixed datum plates and in the plates on the vessel and the movement with respect to vertical sight lines measured using an Autoplumb.

Vertical movements of the top surface were measured by precise level at the centre point and on the circumference relative to a datum on the Reactor Hall wall at pile cap level.

### 5.3 Liner Gauges

Strains in the steel liners were measured by foil resistance gauges bonded to both surfaces in rosettes. Those on the outside of the liner were of 1in. gauge length and were pre-encapsulated in resin before being installed immediately prior to casting of the enclosing concrete. Each assembly was protected from wet concrete by an additional 5in. diameter covering of silicone rubber cast insitu. The internal gauges of 0.5in. gauge length, could only be installed immediately prior to each proof pressure test, and were located where possible in positions corresponding to the external gauges. A total of 692 gauges were installed on each vessel liner in 194 rosettes. These were generally disposed in similar planes to the internal vibrating wire gauges with additional gauges situated to monitor one of each of the service penetrations.

## 6. PRESTRESSING OF VESSELS

### 6.1 Measurement of Prestress Tendon Properties for Design Analysis

The majority of the prestressing tendons were totally enclosed in helical steel tubing and incorporated both a lubricant and a corrosion protection medium. In general, stressing was undertaken at both ends of the tendon simultaneously which resulted in a force profile along the tendon of the form shown in figure 3a.

Initially, the stressing jacks draw the tendons up to the full transfer load at the anchorages. The slope of the line ABC then represents the force distribution along the length of the tendon and the force at any point is dependent upon -

- a) The pressure in the jacks
- b) Friction losses in the jacks
- c) Friction losses through the anchorage and the cone shaped end to the tendon duct (trumpet).
- d) Distance from the anchorage and angle turned through.

- e) The coefficient of friction between the tendon and its duct. ( $\mu$ )
- f) The degree of loss caused by "wobble" or general tolerance misalignment of the ducting. ( $k$ )

After transfer in the Freyssinet system, there is an immediate loss of load at the anchorage generated from the amount of extension lost as the male and female anchorage seat together. The final anchorage load  $F^t$  and the position of the maximum load B are dependent, not only upon  $\mu$  and  $k$ , but also the male cone draw-in and the stiffness of the tendon ( $EA$ ). The latter is a function of the elastic modulus  $E$  and cross sectional area  $A$  of the steel in the tendon.

In order to perform a comprehensive analysis of the stress distribution incurred by prestress it was necessary to examine these characteristics of the prestressing tendon system. Values for the parameters  $\mu$ ,  $k$ , and  $EA$  and estimates of the friction losses in jacks and anchorage assemblies were therefore established in a comprehensive series of full scale trials in a specially constructed test rig. [6]

### 6.2 Calculation of Forces for Analysis

With knowledge of the geometry of the tendon and values for the prestress system parameters it was possible to calculate the force distribution along the tendon, as represented by ABC in fig. 3a. In this particular form of stressing system a loss of force occurs when the male cone finally seats on the female cone represented by A'B and the area  $ABA'$  divided by  $EA$  equals the amount of draw-in.

Because of the large number of different tendon geometries in the Wylfa vessels, the tedious procedure of calculating forces was expedited by a suitable computer program. The program also generated basic geometric data for setting out the tendons during construction. Additional programs were employed to apply the calculated forces to either the ring element or dynamic relaxation analyses.

### 6.3 Prestressing Tendon Parameters Measured Insitu

In order to provide verification of the values of the parameters adopted in the design, an assessment of insitu values was attempted by conducting tests on actual vessel tendons.

Initially 11 tendons in each of the vessels were stressed to full load at one end only, and both the total extension and the load at the unstressed (dead) end recorded. From this data, and an assumed range of values for  $k$ , it was then possible to estimate the likely insitu values of  $\mu$  and  $EA$ .

Whilst there were no significant differences in the values of  $\mu$  obtained, the observed values of  $EA$  were up to 20% lower than the design



values, which had been calculated as the product of the modulus and cross sectional area of individual strands. Lower EA values were thought to result from the elastic distortion of the tendon cross section as the strands "spread" around curves in the ducting. The reduction appears to relate to the total angular rotation within internal ducting and, in fact, the value of EA obtained from the straight tendon was almost equal to the design value. There was not expected to be any deviation from the assumed tendon force distribution because of the above effect.

As a further confirmation, the parameters deduced from the insitu trials were used to calculate anticipated extensions for 68 other tendons in the first vessel. These were then shown to compare favourably with the measured values, the average difference being generally within 1%.

#### 6.4 Measurement of Actual Prestress Forces

It has been explained how the tendon parameters were evaluated and subsequently confirmed insitu. To establish the force profile after transfer shown in figure 3a, it was assumed in design that as the load decreases due to anchorage draw-in, a "reverse friction" effect occurs for which the values of  $\mu$  and  $k$  are equally valid. It was, therefore, considered necessary to confirm this assumption by insitu tests in order to complete the understanding of the tendon system.

A test procedure was devised whereby a tendon which had already been stressed could be retensioned to recover the load lost at the anchor during transfer. In these tests potentiometric transducers mounted on the jack assemblies recorded the movement of the combined male and female cones relative to the anchor block as the load was increased. Typical data obtained from such a test (figure 5b) shows that the load at which the cones begin to lift-off from the anchor block is reasonably well defined. Apart from minor influences due to the elasticity of the anchorage components this lift-off load must be equal to the residual anchorage load after transfer. It is also reasonable to assume that the total extension  $X$  developed in recovering the load is equivalent to the draw-in since the original pretransfer force profile has been re-established.

This test procedure was employed on 36 tendons in the second pressure vessel covering the complete range of tendon profiles. Each test was undertaken immediately after the initial prestressing of the tendon, in order that the results should not be influenced by changes in load conditions due to creep etc. A high degree of correlation was obtained, the observed values of anchorage load being only 4.5% greater on average than those predicted for the design. Closer correlation may have been possible if the predicted values had been derived using the tendon parameters observed insitu. It was noticeable that the difference was greater for tendons which were more influenced by variation in the values of parameters,

e.g. the cap tendons were 8.6% high on average whereas the hoop and straight rib vertical tendons were almost in exact agreement. However the scatter of differences was small, the standard error being only 1.23% of the mean predicted anchorage load.

## 6.5 Analysis of Prestress Loading

### a) Ring Element

Both of the analytical methods described earlier were employed to determine the stress and strain distributions under prestress loading. In the ring-element analysis undertaken for the design, creep of the concrete was anticipated by the introduction of an effective modulus of  $4 \times 10^6$  lb/in<sup>2</sup>. Values of strain were determined by solutions of the usual three dimensional stress/strain equations in which both Poissons ratio and the magnitude of the radial stresses had to be estimated.

### b) Dynamic Relaxation

The Dynamic Relaxation Method, developed after the contract design, was employed in a series of confirmatory analyses in which the actual timing and numbers of tendons appropriate to six significant stages in the stressing of the first vessel were used. Since this analysis gave the strains within the concrete, the accuracy of prediction was expected to be higher.

This analysis was axisymmetric, using a 36in. (91.4cm) square cartesian grid. The vessel ribs were represented by "smearing" them to become axisymmetric, reducing their modulus to provide the correct longitudinal stiffness, and completely removing their hoop stiffness. The influence of creep of the concrete on the strains developed with time could be represented by the introduction of an effective modulus. It could then be assumed that the cumulative effect of individual loads applied at different times was the sum of separate analyses undertaken for each load with its relevant effective modulus.

For instance, supposing load 1 was applied at time  $t_1$ , and load 2 at time  $t_2$  and it was required to know the strain at time  $t_3$ . This could be achieved by analysing a vessel subject to load 1 with an effective modulus which included for the effect of creep between  $t_1$  and  $t_3$ , and another vessel subject to load 2 with an effective modulus which included for the effect of creep between time  $t_2$  and  $t_3$ , and then summing the two results. Since it was proposed to monitor prestress at 6 stages, the above approach was time consuming and another more convenient method was considered. In this, the vessel has a basic modulus and loads 1 and 2 are factored by the ratio of the basic modulus to the respective creep effective modulus. The loads are then applied simultaneously. The strains and deflections would thus be of the correct magnitude but the stresses would be higher. The correct stresses are obtained by application of the unfactored loads to the vessel with the basic concrete modulus. The above method was used in

the D.R. study, and check values were taken at all six prestressing stages, stage six being that of full prestress. Two analyses were, therefore, conducted for each stage one to give correct strains and deflections and the other to provide the correct stresses.

The technique was very easy to apply. A set of data was created which gave the loadings on the vessel for every layer of tendons and it was only required to supply the effective modulus ratios at any stage. For tendons not stressed, the ratio was obviously zero and for the remainder, it was derived from the specific creep strain value relevant to the time for which the particular layer had been applying stress.

#### 6.6 Vessel Monitoring During Prestressing

A complete set of strain gauge and movement readings was taken just before the commencement of the main prestressing of the vessels and again immediately after its completion. In addition, intermediate readings of selected gauges and movement reference points were taken at convenient stages during the prestressing operation.

All gauge readings had to be taken by comparators which were carried to appropriate junction boxes in order to connect in the gauges.

It was not possible to take all the movement measurements because of the difficulty of getting to some of the plates to install the pointers or the targets and in some cases sight lines were obscured by temporary works.

### 7. COMPARISON OF OBSERVED AND PREDICTED BEHAVIOURS UNDER PRESTRESS

#### 7.1 General

Comparisons can be affected by the following factors:-

- i) Tendon forces
- ii) The assumptions made in the transference of tendon forces to loading on the vessel.
- iii) The method of analysis
- iv) The choice of concrete effective modulus
- v) Inaccuracies in strain gauge readings.

From the insitu tests on the tendons and the lift-off tests it would seem that the theoretical tendon forces would be within 5% of the actual tendon forces. The rest of the effects can only be assessed by comparing predictions with measurement.

#### 7.2 Performance of Vibrating Gauges

The consistency of gauge results can be judged by comparing the primary and duplicate gauge readings, and on PVI of the 116 pairs of gauges which could be compared, only 33% were within 5% of their average value and 53%

within 10%, on PV2 48% were within 5% and 74% within 10%.

Figures 4 and 5 show the results obtained at two positions on the vessel during the prestressing sequence compared with prediction. These diagrams include all gauges which could be reasonably expected to yield similar results, and they do therefore illustrate the amount of scatter which was recorded. Closer examination of the strains recorded by an individual gauge, as shown by the dashed lines on these diagrams, reveal that whilst the final strain appears to be grossly in error the strain profile is very similar to that which was predicted and there is in fact a uniform shift of the profile indicating that the final error was simply due to a miss reading at zero prestress. These results illustrate the difficulty of reading gauges over a long period. Errors can occur because the gauge has not been activated for a long time, they can also occur because of reconnections being made and they may also occur because of miss reading of the comparator or wrong transcription of the reading or errors in the temperature correction. If meaningful information is to be obtained from strain gauges it is, therefore, essential that readings are carefully monitored and if necessary readings retaken to obtain correct values.

### 7.3 Comparisons with Predictions

Figures 6 and 7 shows comparisons at final prestress of meridional and hoop strains on the inside face which broadly show reasonable agreement except in the following areas:-

- i) Top Cap
- ii) Around  $55^{\circ}$  degrees below the equator in the hoop direction.
- iii) Around  $45^{\circ}$  degrees above the equator in the meridional direction.

The strains in the bottom cap are in reasonable agreement with those predicted thus confirming the adequacy both of the assumptions as to the distribution of prestress load, and the analytical method. Since the tendon arrangement in the top cap is very similar to that in the bottom cap there are three possible reasons why there is diversity of results in the top cap:-

- i) Strain gauge errors.
- ii) The choice of reduced modulus in the top cap, to simulate the perforated area, is not correct.
- iii) There are strain concentrations.

Results were obtained from four gauge reading and no result was more than 20% from the mean, which is considerably less than the errors between predicted and measured. Thus strain gauge errors must be discounted.

Measurement made on the deflection of the top cap, relative to the Reactor Hall, produced 0.48 inches on PV1, and 0.395 inches on PV2 compared with 0.47 inches predicted. This suggests that the overall stiffness of the perforated area is essentially as anticipated.

It is therefore likely that these discrepancies are due to strain concentrations caused by the penetrations. A tendency for higher strains to develop in this region under proof pressure loading was also noted.

Around 55° below the equator the analysis suggested low strains in the hoop direction. The gauges indicated lower strains than expected and there is no reason to suspect the gauge readings. Since the hoop strains at this point are almost entirely dependant upon the hoop prestress then it must be concluded that the analysis did not correctly represent the true behaviour in this area. The dynamic relaxation method went some way towards the correct representation compared with the multi-ring analysis used in the design.

In the area at 45° above the equator the meridional strains appear much higher than expected. This is a highly complex area with regard to strain distribution caused by the effect of the relatively soft top cap and it must be concluded that the analysis did perhaps over complicate the strain distribution in this area although the dip in the curve at 50° appears in both the dynamic analysis and the multi-ring analysis.

Figure 8 shows the final deflections up the external ribs which show reasonable agreement with predicted and were in fact consistent with the measured hoop strains on the external surface.

#### 7.4 Performance of Liner Strain Gauges

In the first vessel, comparisons between corresponding concrete and concrete face liner gauges in unpenetrated areas of the liner could be made at 13 positions as a guide to the general liner behaviour. At these positions the liner strain followed closely the concrete strain behaviour pattern and in many cases kept well within 50 microstrain of the anticipated value. A typical example of the correlation is shown on Figure 9.

### 8. PROOF PRESSURE TEST

#### 8.1 Analysis for Pressure Loading

Two axisymmetric dynamic relaxation analyses were used to calculate strains and deflections for the vessel. One analysis was based on cartesian coordinates which best described the geometry of the external surface and the other used spherical coordinates which perfectly represented the inner surface. Strain contour diagrams were prepared by combining both sets of results and individual strain values obtained from these contour diagrams. The values for deflection of the external surface were derived

directly from the cartesian coordinate analysis.

### 8.2 Test Procedure

A substantial amount of preparatory work was necessary before each pressure test could be undertaken and strict safety measures were adopted for personnel during the tests. Temporary cabling was brought down from all instruments to one central area situated in the basement of Reactor Hall. Access for the purpose of vessel surface inspections and the precise levelling was only permitted at pressure holds.

The proof pressure for these vessels was 488 lb/sq.in., this is 1.15 x design pressure which itself is 1.10 x normal vessel working pressure of 385 lb/sq.in. Full instrument readings were taken at proof and working pressures and at three intermediate stages of  $\frac{1}{4}$ ,  $\frac{1}{2}$  and  $\frac{3}{4}$  working pressure. As an extra safety precaution during intermediate rising increments, each hold pressure was exceeded by 10 lb/sq.in. and then held for 30 minutes before blowing down to the desired level for instrument readings and inspection.

With approximately 1 hour holds at each pressure level the overall time to reach proof pressure was  $5\frac{1}{4}$  hours for each vessel. Blow down to atmospheric pressure using the vessel relief valves was achieved in less than half a day. Full data scans were taken at each pressure hold during blow down but these were not processed during the course of the tests.

### 8.3 Test Instrumentation

#### a) Strain Gauges

During the proof pressure tests one gauge from each pair in both primary and secondary groups of vibrating wire gauges and all liner strain gauges were monitored by data logger and the resulting data processed. At each increment of pressure the data from the primary groups were plotted as strain profiles and pressure versus strain graphs for comparison with predicted values and assessment by the controlling engineers. Data from the secondary groups was not plotted but visual assessments and linearity checks were carried out at each pressure level.

#### b) Deflection Measurement

Overall deformations of the external surface of the vessels under pressure were monitored by a combination of temporary measurement systems which were introduced specifically for these tests.

Linear potentiometric transducers were symmetrically disposed at locations on two diametrically opposed vessel ribs as shown in Figure 2 and denoted by LPT. Those at the equator locations were duplicated on the pair of ribs normal to the main line as a check on symmetry. Each transducer was supported by braced scaffold frames bolted to the surface of the main Reactor Hall columns which were founded independently to the vessels.

All transducers were read by a digital voltmeter. The resolution of each transducer was 0.0017 ins. but this accuracy was reduced since the majority were sited in an exposed environment. Small temperature fluctuations, therefore, influenced their supporting frames to an indeterminate extent.

Limited space between the upper surface of the vessel and the charge floor precluded the use of transducers to measure vertical pile cap deflections. Consequently this area was monitored by a precise level at the locations L on Figure 2. Staffs were installed at each position and their movements observed by a precise level situated on the periphery of the floor. In addition, overall lateral movements of the pile cap along two normal diameters were taken by mechanical dial gauges which were operated by tensioned piano wires. It was not expected that the accuracy of either of these systems was better than  $\pm 0.005$  ins.

Data from all these systems was again reduced and plotted as profiles for assessment at each pressure increment.

## 9. COMPARISON OF OBSERVED AND PREDICTED BEHAVIOURS UNDER PROOF PRESSURE

### 9.1 Evaluation of Insitu Concrete Modulus

As explained earlier, any comparison between the performance predicted and that observed is dependent primarily upon the value assumed for instantaneous elastic modulus for the concrete (E). The mean value obtained from controls prior to the first proof pressure test was  $5.46 \times 10^6$  lb/in<sup>2</sup> and a modulus of  $5.0 \times 10^6$  has been adopted in the analysis. However, the majority of the tests had been conducted when the concrete age was less than six months and a small sample of specimens tested at the time of proof pressure gave a mean value near to  $6.0 \times 10^6$  lb/in<sup>2</sup>.

The component deflections observed from the potentiometric transducers and precise levelling in each test are compared with predicted values calculated for an E of  $6.0 \times 10^6$  lb/in<sup>2</sup> in Figure 10. Since the accuracy of this system of measurement was considerably less than that of the strain gauges, the plotted deflections have been derived from the line of best fit from those observations taken during decreasing pressure. These results were selected since the pressure decrease operation was short in duration consequently the potentiometers were less influenced by temperature or time dependent effects. The comparison appears to uphold the values of concrete modulus selected, the peak values of deflection being substantially as predicted.

Since the response of the vessels was elastic it was also possible to calculate the values for the insitu concrete modulus by equating the mean strain development in the group of secondary embedded gauges at the equatorial section to the vertical pressure force. For this it was

necessary to accept a value for the gauge factor for the vibrating wire gauges and this was set at  $3.0 \times 10^{-3}$ ; this value was derived from the small sample of controls tested at the time of the first proof pressure test. Using the value of 0.15 for Poissons ratio this calculation gave a modulus of  $6.15 \times 10^6$  lb/sq.in. for PV1 and  $5.85 \times 10^6$  for PV2.

It was concluded that these two pieces of evidence provided sufficient justification for the adoption of overall values of  $6.0 \times 10^6$  lb/sq.in for the concrete modulus and  $3.0 \times 10^{-3}$  for the gauge factor of the gauges.

## 9.2 Correlation of Vibrating Wire Gauge Data

Examples of the strain behaviour observed in the tests on the two vessels are shown in Figures 11 and 12. In each case profiles of the strain developed near the concrete surfaces at proof pressure are plotted against the angle of the gauge location from the equator. In general the accuracy of the predictions is reasonable and most minor divergencies can be attributed to the influence of simplifying assumptions made within the analyses. These may be summarised as -

- a) The strain gradient on the contour diagram. Steep gradients occurred theoretically at re-entrant corners and adjacent to penetrations; both effected the accuracy to which the true strain could be isolated. In practice, it was also anticipated that rapid stress redistribution might inhibit the full development of high localised strains in the concrete. There is evidence of this behaviour from the external strains shown in Figure 11.
- b) The use of an equivalent stiffness for the standpipe zone. In the multi-perforated standpipe array the theory only predicted field strains assuming the cap to be unperforated but of a lower stiffness. Local concentration effects were therefore not indicated by analysis. All gauges in locations above an angle  $55^\circ$  above the equator lie within the standpipe zone and, whilst it is evident that correlation is reduced, few gauges indicated significant strain concentrations.
- c) Secondary bending effects. Where the pitch between the ribs is greatest, i.e. the equator of the vessels, it was to be expected that local bending of the shell might occur in the hoop direction. This effect would not be represented by the axisymmetric analysis which evened out the effect of the ribs. This is illustrated by the low hoop strains developed at the interface between the ribs and the shell which are complimented by values slightly higher than predicted on the inner surface (Figure 12).



### 9.3 General Performance of Vibrating Wire Gauges

The gauges showed that the vessel concrete was behaving in a substantially linear and elastic manner. The observed differences between the rising and the falling strain values at each pressure level and the mean residual error on returning to zero were less than  $\pm 5$  microstrain for all gauges, which was well within the expected accuracy of the method of recording.

In attempt to quantify the degree of correlation achieved at proof pressure, linear regression analyses were carried out on the vibrating wire gauge data to determine the standard errors of estimate. In a normal distribution of discrepancies, 68% of observed results will lie within one standard error and 99% within two standard errors of the predicted value. This analysis gave the following results:-

	<u>STANDARD ESTIMATE OF ERROR (MICROSTRAIN)</u>		
	<u>PV1 v prediction</u>	<u>PV2 v prediction</u>	<u>PV1 v PV2</u>
Primary groups	26	26	22
Secondary "	40	36	22

It is noticeable that significantly better correlation was achieved within the primary groups since these gauges lay in regions less influenced by the simplifying assumptions introduced into the analysis. As might be expected the standard error is similar for either group when comparing the observed values from each of the vessels. The results suggest that the error attributable to factors outside the scope of analysis, such as resolution of the strain indicator, variation of gauge factor and local variation of modulus of elasticity accounted for some 22 microstrain of the standard error. Limitations of the stress analysis contributed only a further  $\frac{1}{4}$  microstrain in the case of the primary gauges which represents an extremely adequate description of the overall behaviour of the structure. The standard error in predicting local effects was higher, but not unsatisfactory, the majority of the larger discrepancies having arisen from over-estimates of the predicted strains from the theoretical contours. This is best illustrated by the imbalance shown in the histogram in Figure 13.

### 9.4 Correlation of Liner Strains

The gauges monitored exhibited linearly elastic behaviour during pressure rise. In many cases non-linearity and residual strains occurred on depressurisation. This hysteresis effect is to be expected when resistance gauges have not been repeatedly strain cycled before use. Typical results obtained at an equatorial location are shown in Figure 14.

Where gauges were at coincident positions on the inner and outer liner faces, the observed strains were not equal, indicating a small outward bending equivalent to 100 - 125 microstrain. It was not unreasonable to believe that this order of bending resulted from the reduced liner support

offered by the silicone rubber protection to the concrete face gauges.

Because of gauge losses and the liner bending effect, there were few gauge positions affording a direct comparison of liner and concrete behaviour. Exact correlation was also inhibited by the strain gradient through the vessel at any section. The average differences between comparable readings were 37 microstrain and 30 microstrain respectively for PV1 and PV2. Applicable results are plotted with the concrete strains in Figures 11 and 12.

There was considerable scatter of individual results between the two vessels but, at the 18 locations where liner strains were directly comparable the average of the meridional strains on PV1 was within 10 microstrain ( 8% ) of that predicted, and 20 microstrain (16%) on PV2.

#### CONCLUDING OBSERVATIONS

The correlation of the behaviour of these vessels with that predicted by analysis is dependent upon:-

- a) The accuracy of the measurement techniques available to determine the actual vessel behaviour.
- b) The limitations of the analysis in describing both the geometry of the structure and the imposed loadings.
- c) The extent to which the insitu behaviours of the component materials and systems are understood and can be represented in the analysis.

Any difference between observed and predicted values is a combination of all these influences, and it is not possible to quantitatively apportion the error between them.

Proof pressure was a well conditioned test for correlation purposes as the loading was simple to represent analytically and the duration sufficiently short to ignore time dependent effects. Automatic logging also considerably improved the performance of the vessel instruments. However, the major portion (80%) of the standard error existing between measured and predicted strain data arose from unquantifiable effects - namely the heterogeneity of the vessel concrete and variation in instrument calibration factors.

For the prestress loading case, the added influence of time on the interpretation of the concrete properties for the analysis and on the performance of the vessel instruments considerably reduced the degree of correlation achieved. Further difficulties arose from the inability to accurately know and represent prestress loading in the analysis; by virtue

of, not only variations in tendon parameters, but also because the load transfer to the concrete was not necessarily continuous along the tendon path, as was assumed in the analysis. Nevertheless the major source of error was still believed to be associated with the strain gauge readings which were only read intermittently over a relatively long period. It was evident, however, from the insitu trials on the prestress system that its efficiency, and consequently the development of the required forces, was very close to that anticipated in the design.

As stated above it is known that the designed prestressing loads have been applied to the structure, and it is also true that simple statical checks, ultimate load predictions and various model tests [7] have confirmed the ability of the structure to safely carry the designed pressure loading. In general, the levels of strain indicated by the gauges compare reasonably well with predictions for the two loadings considered and hence the structures are behaving essentially as expected. There are significant differences between the observed and predicted strains at individual locations but there are also variations between one vessel and the other and between gauges in similar locations which cannot be explained quantitatively and further work is in progress in an endeavour to obtain more meaningful results.

For the future it must be recognized that the installation of gauges into a vessel should do more than just provide information at proof pressure testing. It is evident that analytical techniques are now adequate to describe the general behaviour of P.C.P.V.'s and efforts must be directed to obtaining the operational behaviour of the vessel, particularly with respect to movements and strains which effect the reactor plant. The behaviour of the vessel is of course dependent upon the physical properties of the concrete and the prestressing tendons.

Measurements will only be useful if the relevant parameters can be quantified and the performance of vessels in service could produce valuable information on this subject. This information can be obtained from, not only strain and movement measurements in both determinate and more complex zones, but also from continuous re-evaluation of the prestress force distribution. Above all it must be realized that instrumentation will only produce genuine values as a result of careful monitoring and scrutiny of the indicated strains.

#### ACKNOWLEDGEMENTS

The Authors are grateful to Taylor Woodrow Construction Limited and British Nuclear Design and Construction for permission to publish this Paper. The Authors are also indebted to their colleagues who contributed to the many aspects of development, design and construction.

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- ① CORE
- ② SHIELDING
- ③ BOILERS
- ④ STANDPIPES
- ⑤ CIRCULATORS
- ⑥ RIBS
- ⑦ TENDONS
- ⑧ EQUIPMENT BUILDING

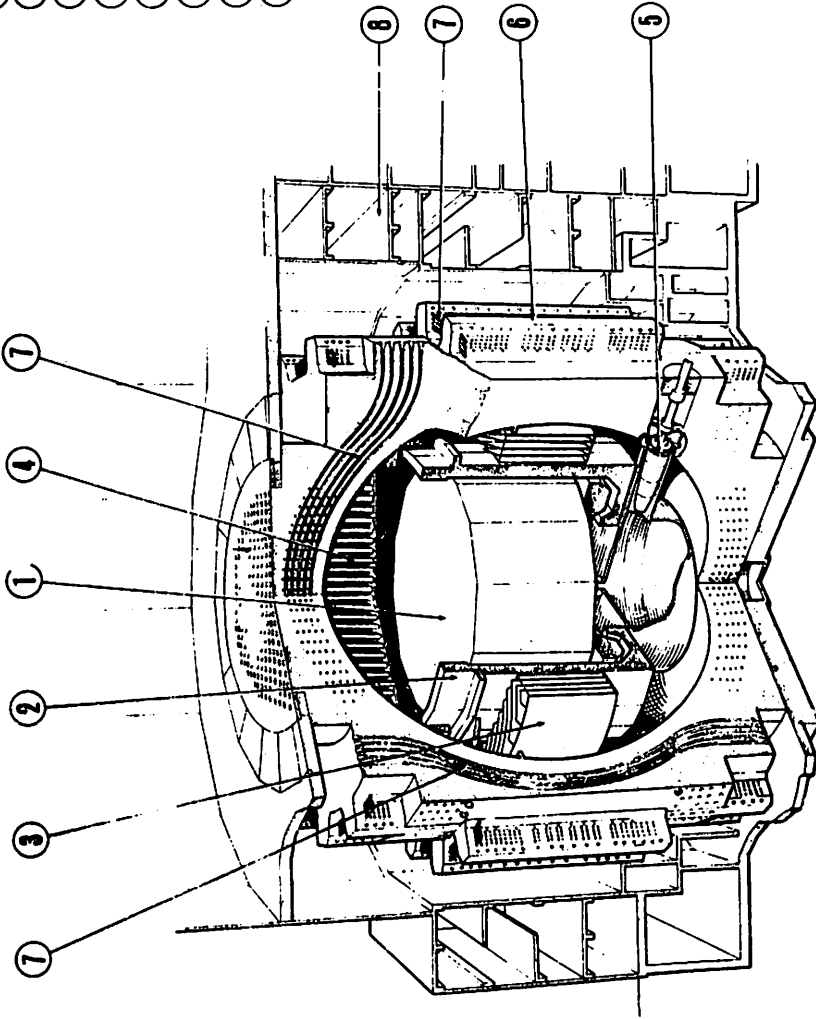


Figure I. General Arrangement of Reactor Pressure Vessel.

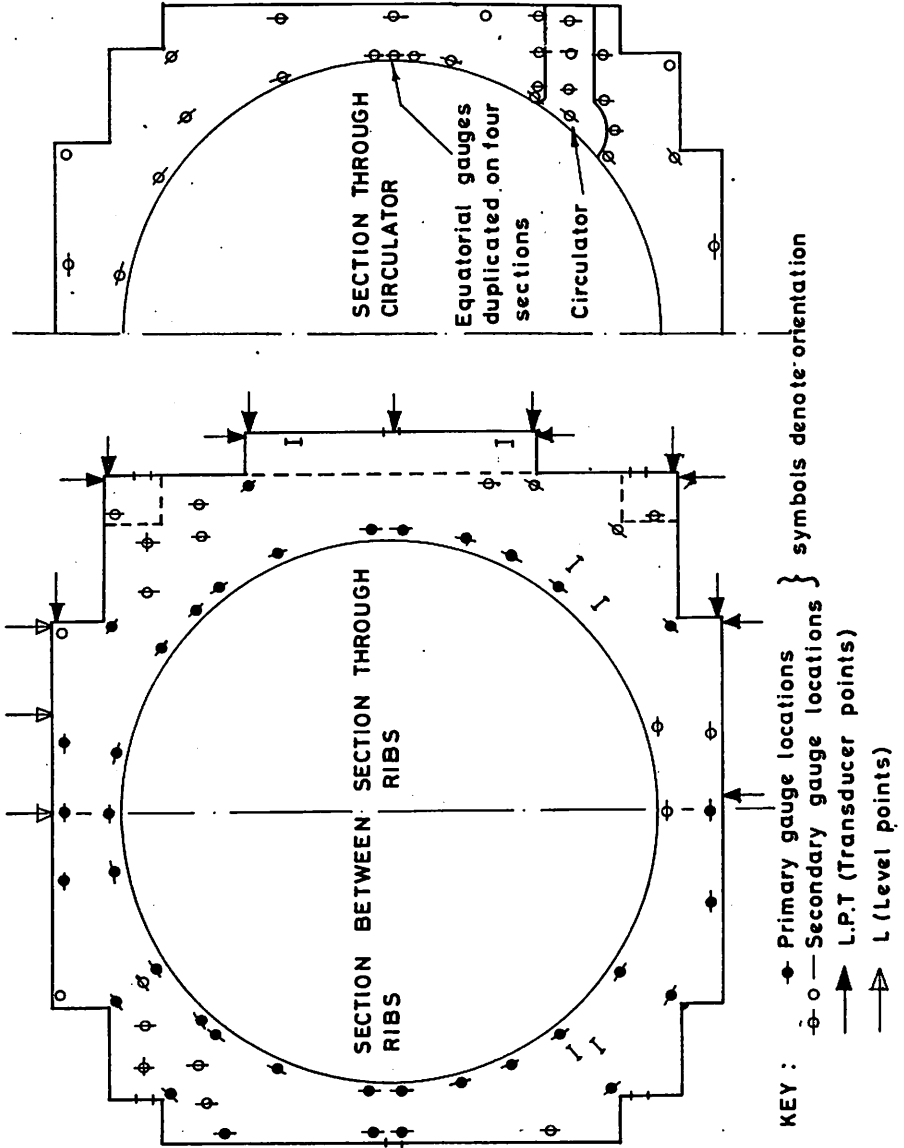


Figure 2. Location of strain and movement sensors.

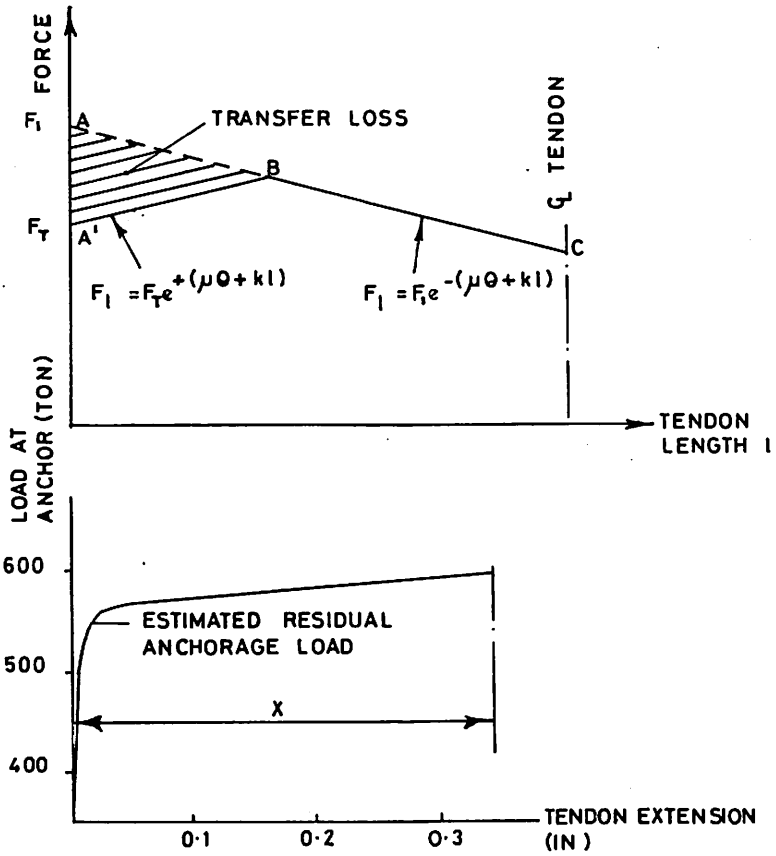


Figure 3. a) Simplified Tendon Force Distribution.  
b) Typical lift-off load extension curve.

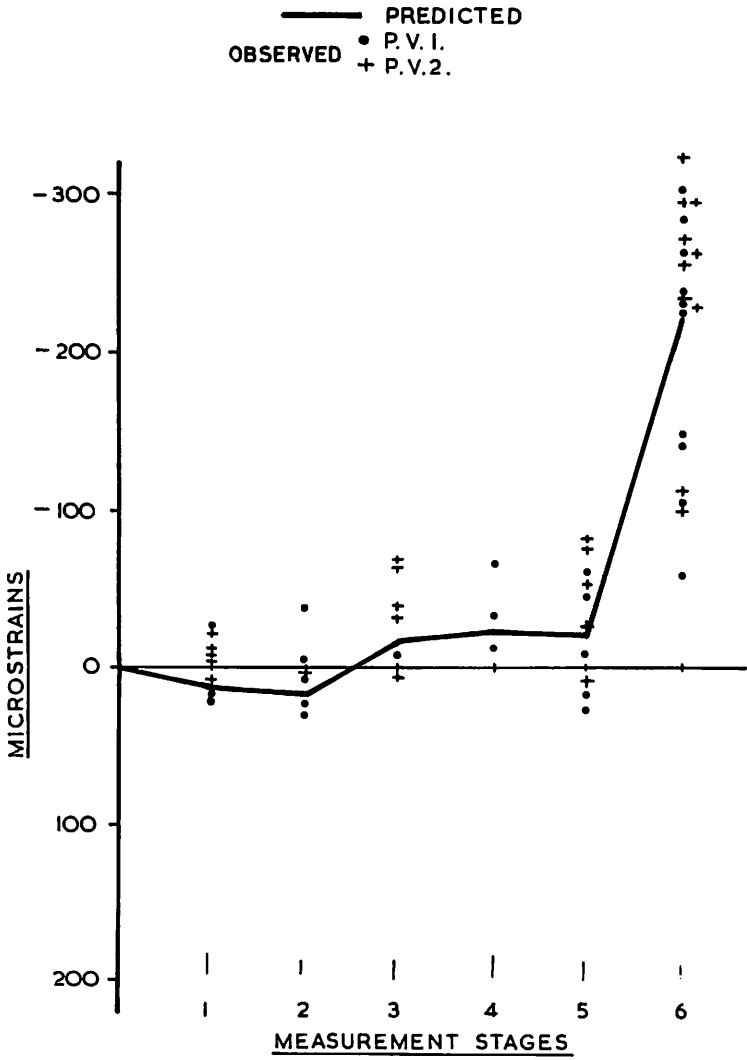


Figure 4. External Equatorial Hoop Strains During Prestressing.



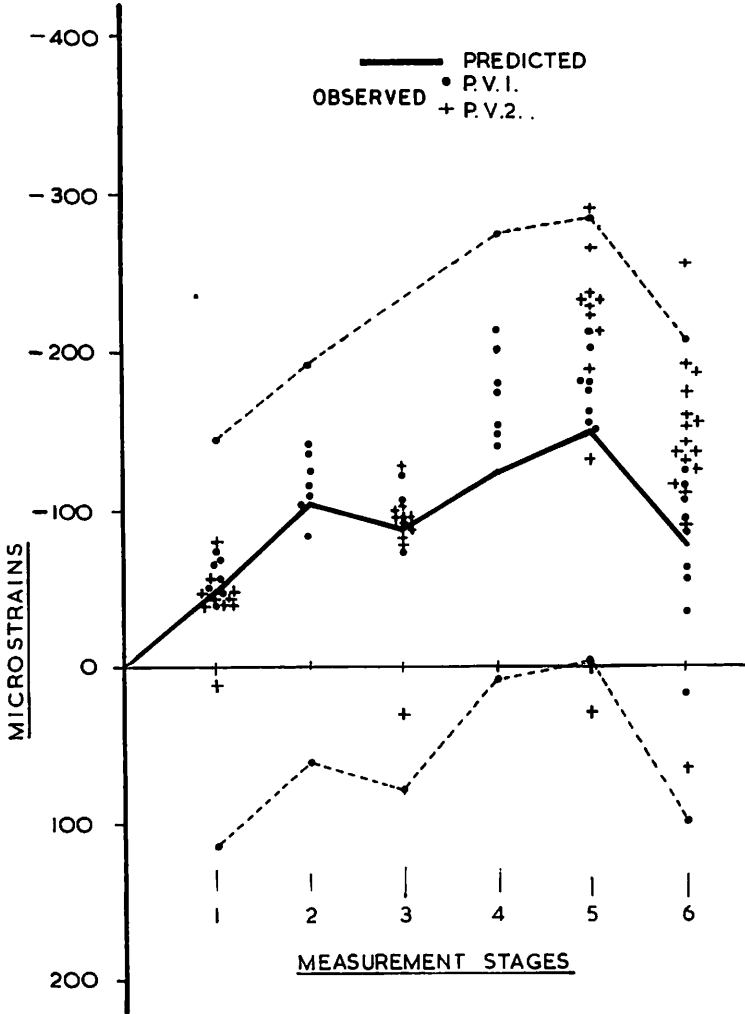


Figure 5. Internal Equatorial Meridional Strain during Prestressing.

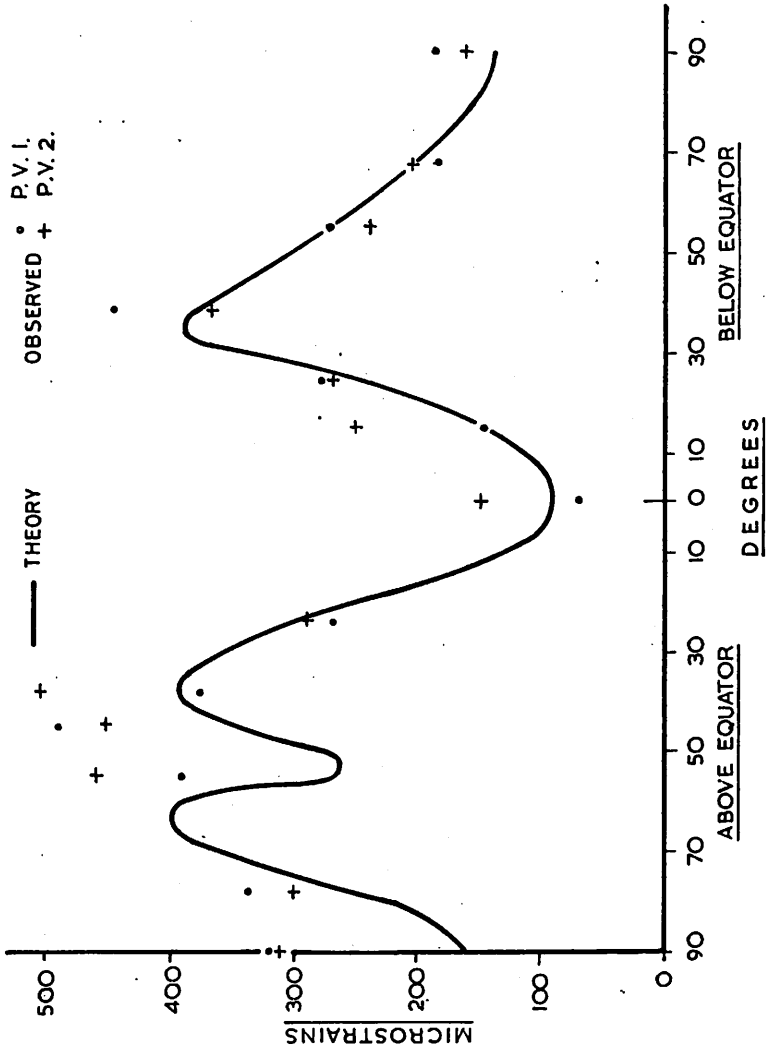


Figure 6. Internal Meridional Strains Due to Full Prestress.

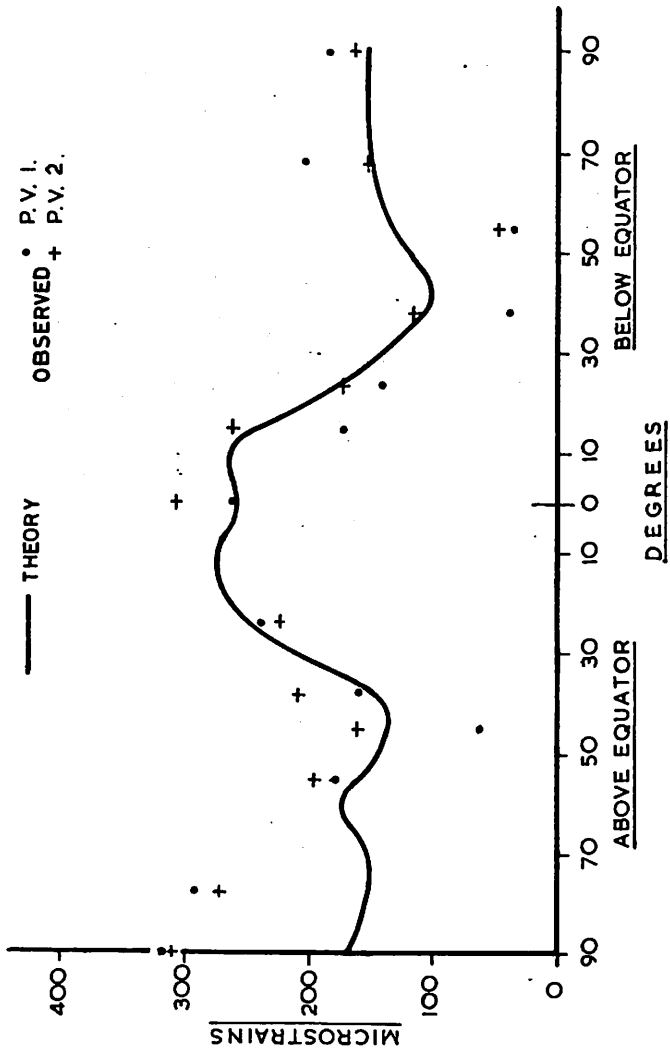


Figure 7. Internal Hoop Strains Due to Full Prestress.

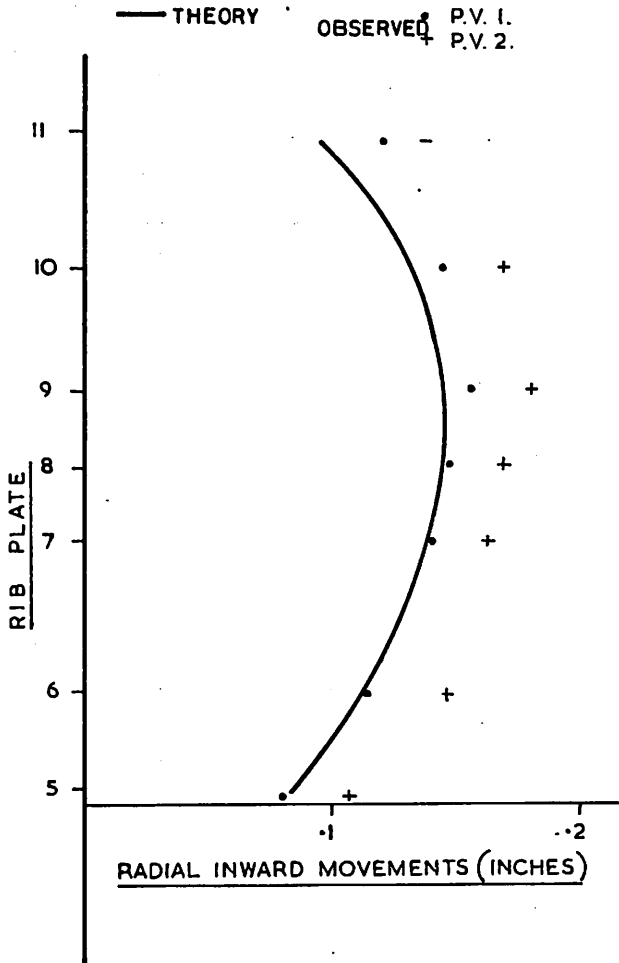


Figure 8. External Movements on Vessel Ribs Due to Full Prestress.

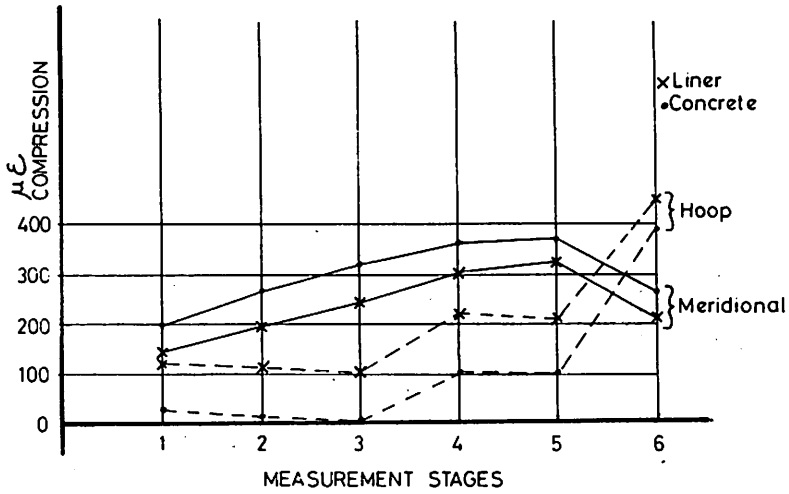


Figure 9. Comparison of Liner and Concrete Strains during Prestressing.

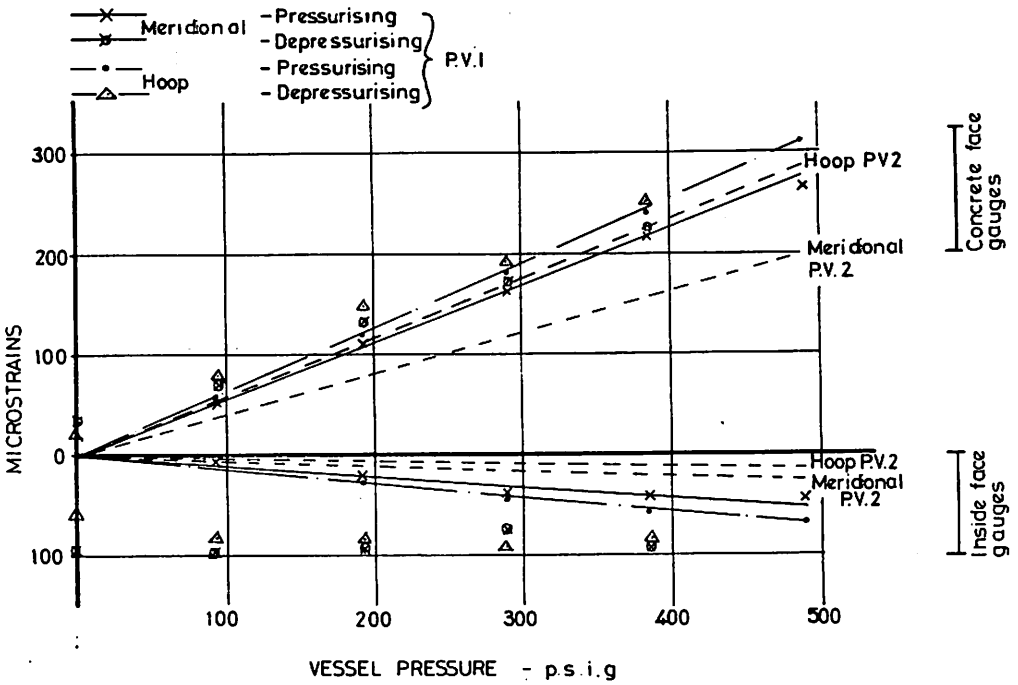


Figure 14. Performance of Typical Liner Strain Gauges during Proof Pressure Test.

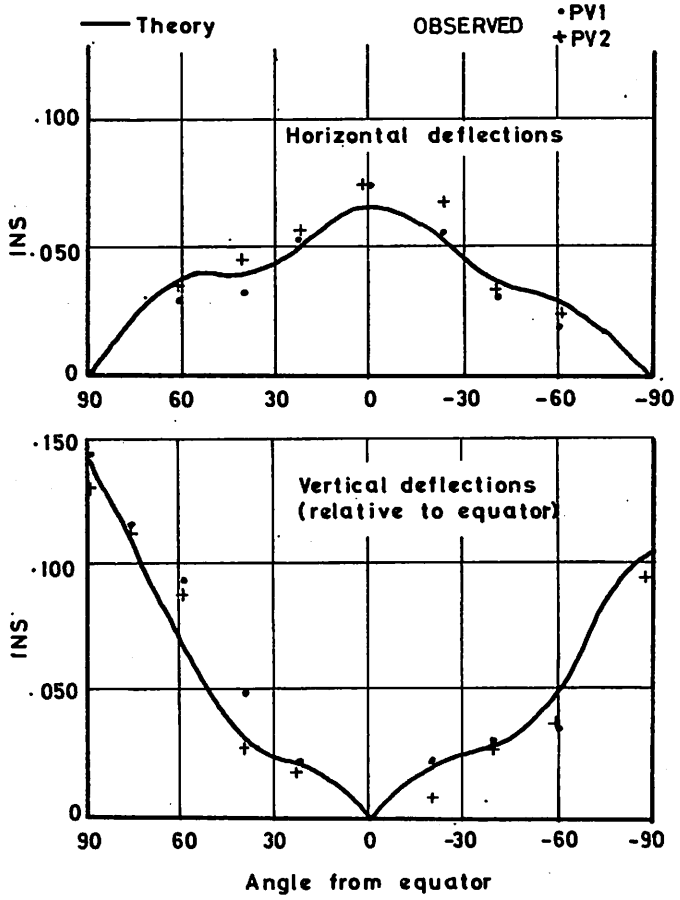


Figure 10. External Deflections Due to Proof Pressure

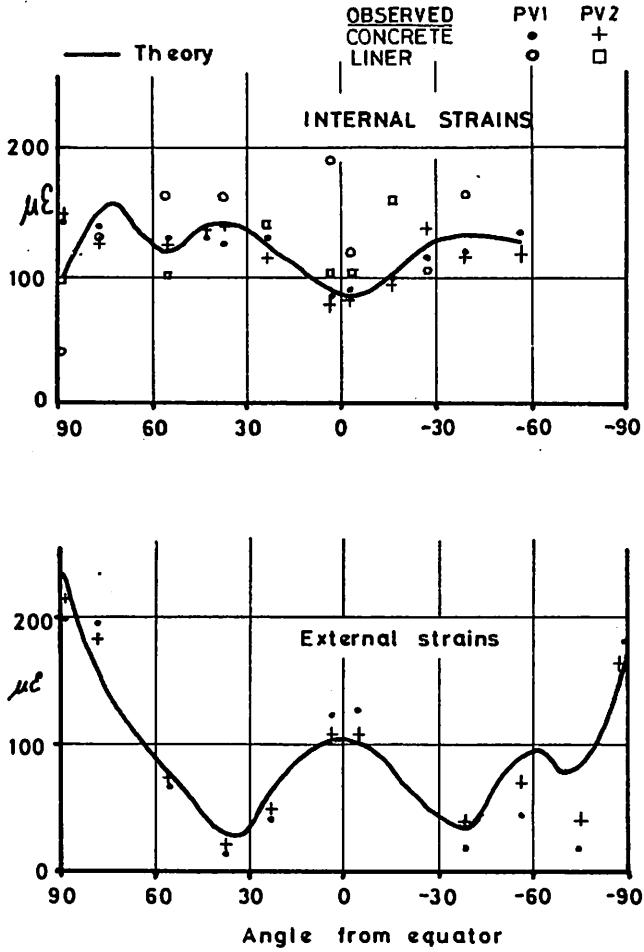


Figure 11. Meridional Strains at Section between Ribs due to Proof Pressure.

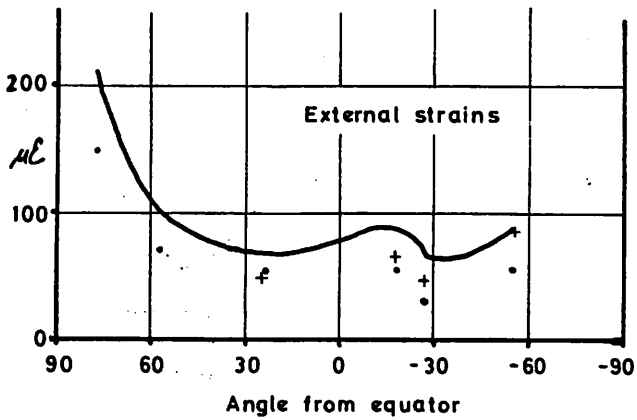
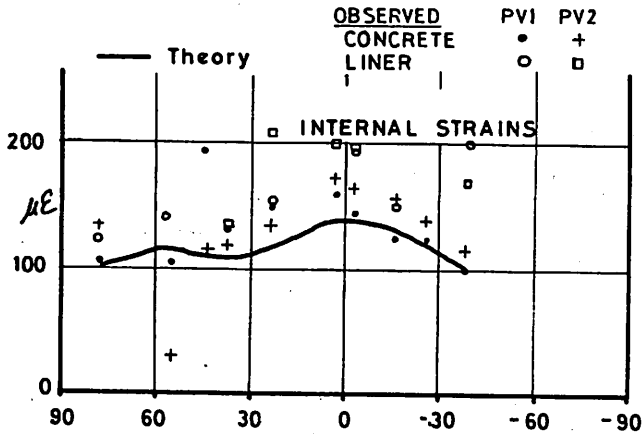


Figure 12. Hoop Strains at Section through Ribs due to Proof Pressure



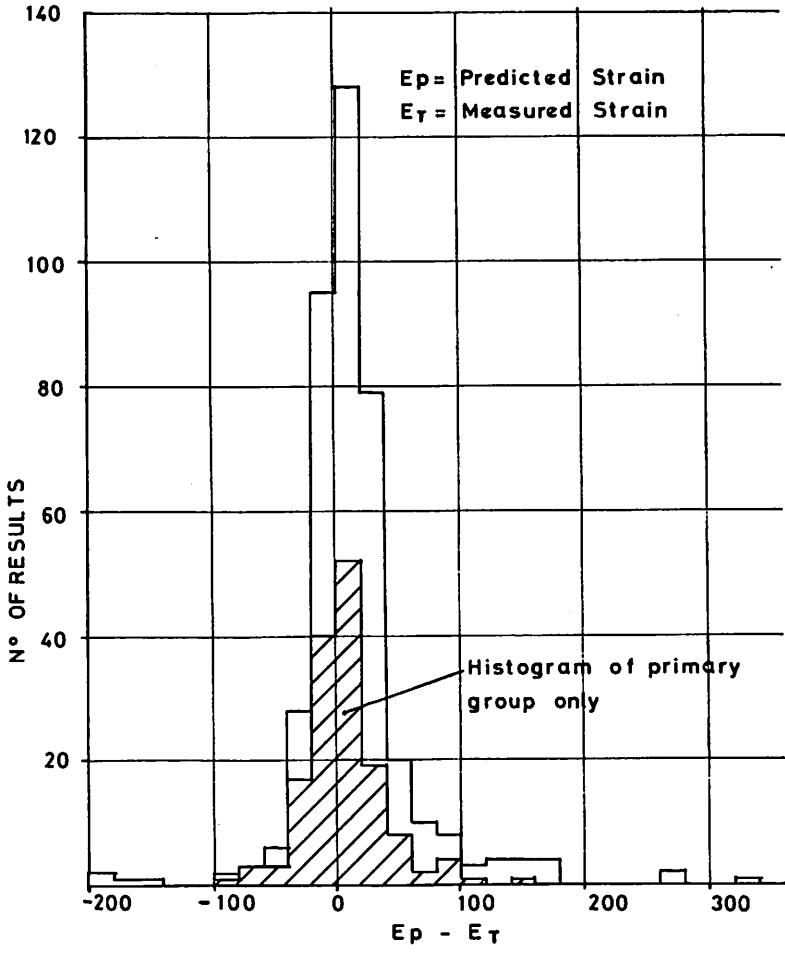


Figure 15. Comparison between Predicted and Measured Concrete Strains for Both Vessels at Proof Pressure.

DISCUSSION

F. SCOTTO, Italy

Q

The leading philosophy in your country was, as far as I know, for "not to grout" the tendons.

1. Which is your personal opinion in this respect on the basis of your experience ?
2. Did you schedule the control of tendons during the reactor life, if yes which kind of control is foreseen ?

R. E. D. BURROW, U. K.

A

British specifications require the prestressing tendons of nuclear vessels to be left ungrouted so that it is possible to verify the loads in the tendons and, if necessary, restress or replace these. Thus, the safety of the vessel can at all times be demonstrated to the inspection authorities. Surveillance measures for the Wylfa vessel are now under final discussion with the U. K. Central Electricity Generating Board. The most important check will be a periodic measurement of the residual load in typical prestressing tendons. Initially this check will be made annually, but with satisfactory experience, the interval will be progressively increased. Each surveillance will also include a check of selected tendons for corrosion, and an inspection of the anchorages and external surface of the vessel for any signs of damage or malfunction.

The initial surveillance of Wylfa, recently completed, has given the two vessels a completely satisfactory report. There are no signs of deterioration and the prestressing loads are within one or two percent of the values predicted.