Zeitschrift: IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen

Band: 19 (1974)

Artikel: Analysis and behaviour of triaxially prestressed concrete components for prestressed concrete pressure vessels

Autor: Garas, F.K.

DOI: https://doi.org/10.5169/seals-17516

IABSE
AIPC
IVBH

SEMINAR on:
« CONCRETE STRUCTURES SUBJECTED TO TRIAXIAL STRESSES »
17th - 19th MAY, 1974 - ISMES - BERGAMO (ITALY)

Analysis and Behaviour of Triaxially Prestressed Concrete Components for Prestressed Concrete Pressure Vessels

Analyse et comportement de composantes pour caissons en béton précontraint triaxialement

Analyse und Verhalten der dreiachsigen Spannbeton-Komponenten für Spannbeton-Druckbehälte

F.K. GARAS, Ph.D., B.Sc., M.I.C.E.
Head of Structures Research Laboratory,
Taylor Woodrow Construction Limited, Southall, Middlesex, U.K.

1. INTRODUCTION

The two nuclear power stations at present being constructed at Hartlepool and Heysham(1) in the United Kingdom have twin advanced gas-cooled reactors with 1250 MW electric output per station. They are being constructed by British Nuclear Design and Construction for the Central Electricity Generating Board and are the first commercial stations to incorporate the use of the podded boiler concept. In the type of prestressed concrete pressure vessel which has been adopted, the boilers and gas circulators are housed in vertical pods in the walls of the vessel. The advantage of this type of design is that the boilers can later be removed for maintenance, should this ever be required in service.

During the lifetime of the vessel it will be subject to complex mechanically and thermally induced stress conditions. In certain zones of the vessel the stresses may be higher than those normally accepted in conventional structures. Thus there was a need for considerable experimental verification during the development of the pressure vessel structure. The paper deals with only one aspect of the triaxial stresses where the concrete is under the combined action of high shearing forces and biaxial restraint.

2. VESSEL DESCRIPTION

The vessel is cylindrical in form with an external diameter of 25.9 m (85 ft) and overall height of 29.3 m (96 ft) and is vertically and circumferentially prestressed. Circumferential prestress is provided by wire winding the cylindrical surface of the vessel. The end slabs and the walls are 5.5 m (18 ft) and 6.4 m (21 ft) thick respectively. (See Fig. 1.)

The boilers are contained in eight circular cavities of 2.74 m diameter and are housed within the wall of the pressure vessel. Several hundred penetration tubes are also incorporated throughout the concrete mass.

The boiler closure, which forms an integral part of the complete boiler unit, is a prestressed concrete component. In addition to the primary function of closing the upper end of the boiler pod, the closure supports the boiler heat transfer surface by means of a central axial spine tube, and carries penetrations for the steam and feed tail pipes. (See Fig. 2.)

By constructing the closure of prestressed concrete it became possible to apply the same safety philosophy and criteria to the closure as were applied to other parts of the vessel. The closure is prestressed by circumferential wire winding, but additionally, substantially unstressed wire is wound on to provide a passive radial restraint to the concrete plug. The passive restraint ensures that the closure, without prestress, has a load carrying capacity in excess of the design ultimate load.

3. END SLABS AND BOILER CLOSURES

3.1. Design Problems

The end slabs and boiler closures are geometrically deep in relation to their span; the shear span to depth ratios are 2.39 and 1.62 respectively. High operating pressures results in higher shear forces than normally accepted in conventional structures. Their strength is, however, enhanced by the use of hoop prestress.

In the assessment of the ultimate strength of the end slabs and boiler closures, the shear stresses were found to be dominant in determining the mode of collapse. The particular mode of failure is greatly influenced by the behaviour of the concrete under complex states of stress.

Because of the complexity in the behaviour of thick elements under high shear stresses, designers tend to be very conservative in their selection of allowable stress and hence their requirements for a factor of safety. This is due to the lack of knowledge of the distribution of stresses in the compression zone, and of the capability of the concrete to withstand these stresses. The current practice is to adopt methods used for conventional structures which are primarily subjected to flexure. These methods usually ignore the high shear stresses which can be mobilised and the triaxial state of stress which occurs in this type of structure, resulting in an uneconomic design.

3.2. Assessment of Shear Resistance

Because of the low span to depth ratio, flexural failure is unlikely to occur in the end slabs and boiler closures. Shear failure is a characteristic of thick slabs and occurs before the flexural capacity is exhausted. In order to understand the failure mechanism in deep structural members, the following points should be considered:

(i) the shear stresses are more dominant than those due to flexure

(ii) the geometry of the section induces stresses which are essentially three-dimensional

(iii) just prior to failure the strength of the member is dependent on the resistance of an element of concrete subject to triaxial compression with shear - a state of stress which as far as the author is aware has not fully been studied.

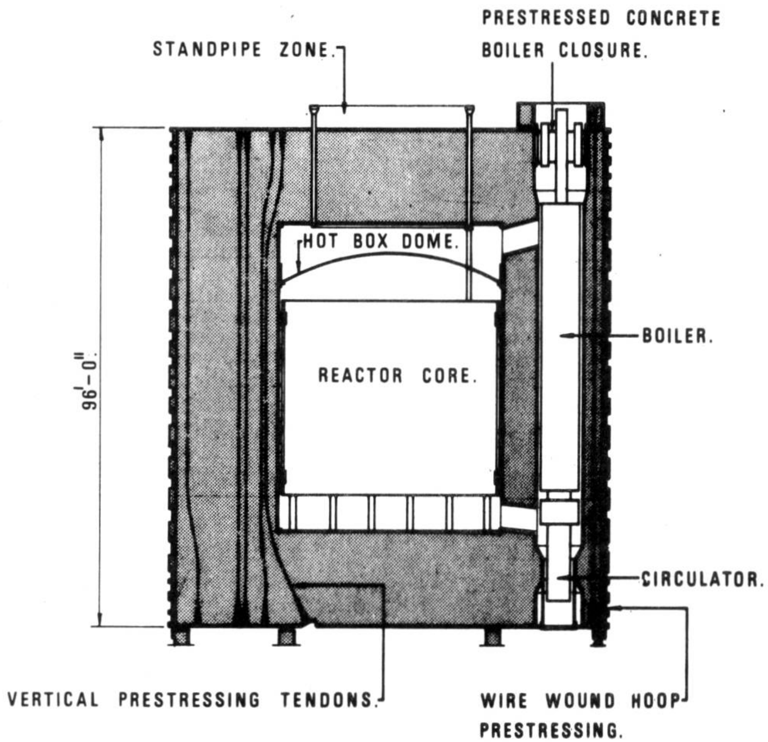


FIG. 1

SECTION THROUGH HARTLEPOOL / HEYSHAM PRESSURE VESSEL.

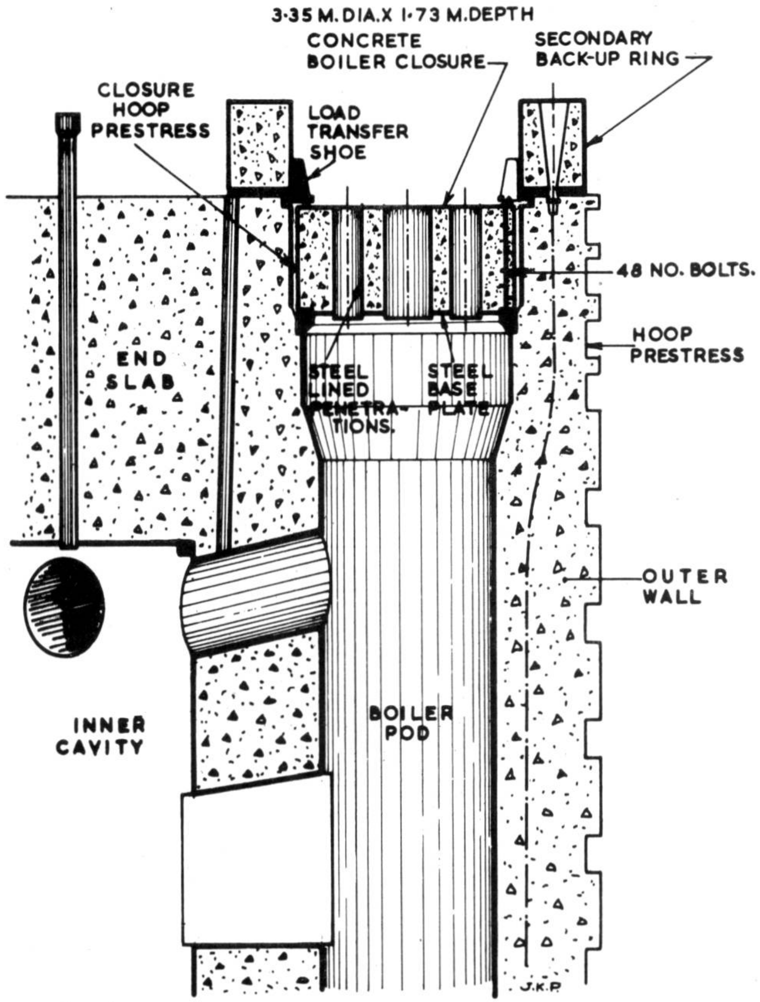


FIG. 2

CONCRETE BOILER CLOSURE IN THE HARTLEPOOL PRESTRESSED CONCRETE PRESSURE VESSEL

3.3. Shear Components

As an aid to the understanding of shear mechanism the components of the internal shear resistance which balance the total shear force generated by the applied forces at the point of failure are identified. In restrained slabs the total shearing force 'F' is resisted by the component forces shown in Fig. 3.

3.3.1. Shear Resistance of the Compression Zone 'Fc'

From the author's experimental work(2) it was found that at the point of failure the depth of the compression zone was reduced to about 15% of the original depth of the slab, see Fig. 4.

The contribution of this component may be assessed using the relationship between the biaxial compression and the shear stresses based on the parametric study which was carried out by the author(3). Using this information and determining the depth of the compression zone, the shear resistance of the compression zone was found to be 45% of the total.

3.3.2. Shear Transfer by the Vertical Components of the Aggregate Interlock within the Tension Zone 'Fg'

This action develops because of a shear displacement parallel to the direction of the shear planes. When a crack forms in the section the shear displacement causes the larger aggregate particle to act as dowels. Smaller crack widths increase the significance of aggregate interlock, and since this is a function of the aggregate itself the properties of the aggregates are important.

From the behaviour of end slabs examined by the author(2) the calculated crack width prior to failure was about 0.003 in (0.08 mm). By extrapolation using Fenwick's work(4) on the contribution of the aggregate interlock in resisting shear forces, this value was found to be in the range of 50% of the total shear force.

3.3.3. Shear Transfer by Dowel Action of Bonded Reinforcement 'Fd'

At present there is no adequate method of assessing the magnitude of this component but because of the low steel percentage its contribution to the shear resistance was found to be very small(2).

3.4. Method of Analysis

To develop general design rules for deep restrained sections, two approaches were possible:

(i) to develop a mathematical approximation to the particular mode of failure observed from the testing of elements under certain boundary conditions, i.e. find a solution for each type of shear failure.

(ii) to undertake a parametric experimental study and relate all results non-dimensionally; this could lead to an empirical formula based on a statistical analysis of the data.

The second alternative was more realistic although the amount of available data on the ultimate load behaviour of restrained and unrestrained deep elements is still too limited to enable comprehensive design rules for shear to be developed.

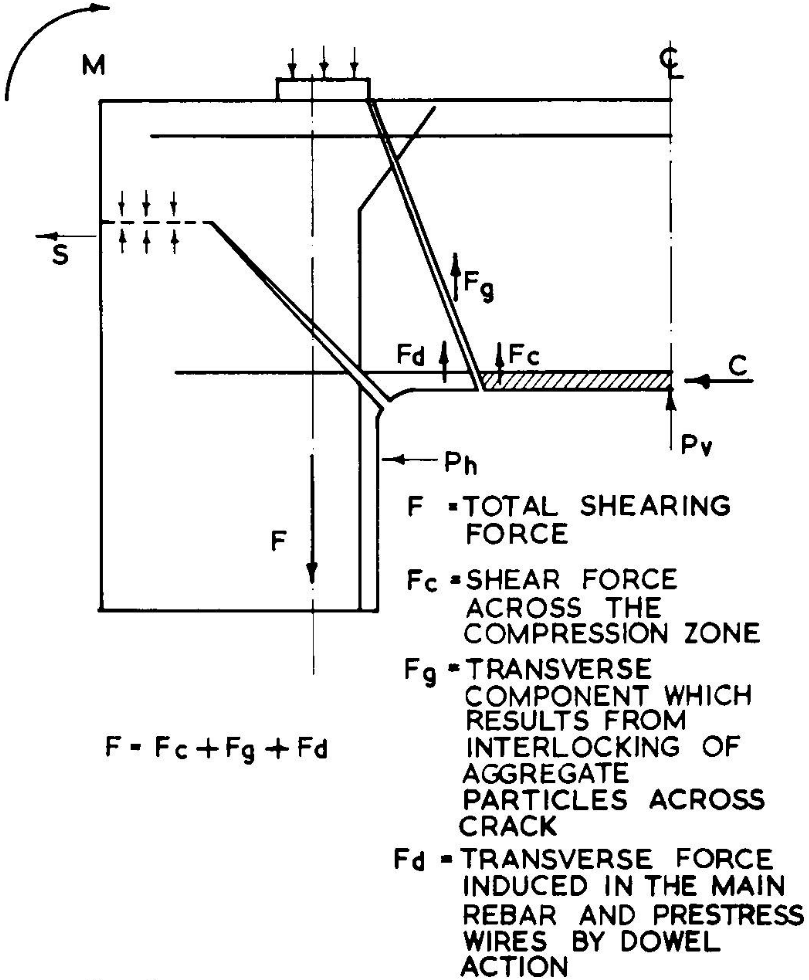


FIG. 3

MECHANISM OF SHEAR FAILURE

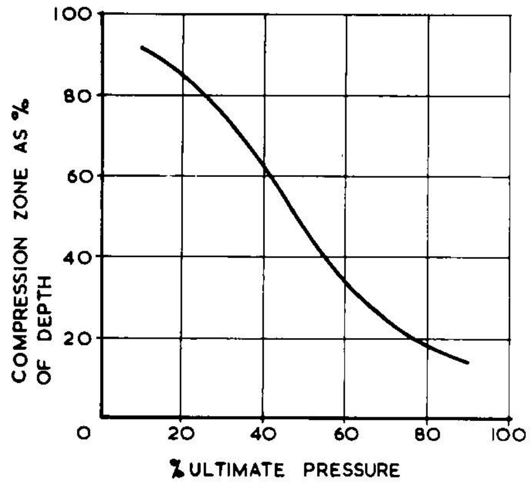


FIG. 4

REDUCTION OF DEPTH OF COMPRESSION ZONE WITH PRESSURE

4. EXPERIMENTAL INVESTIGATION OF END SLABS

4.1. General

During the last eight years an extensive study was carried out into the behaviour of end slabs in cylindrical prestressed concrete pressure vessels. To date, 23 end slab models have been constructed and tested to failure. The test programme, which was described in an earlier paper(2) was designed to examine the effect of the variation of the major parameters on the ultimate strength of failure of end slabs, e.g. hoop prestress level, penetrations lined or unlined and pressurised penetrations, boundary conditions and sustained elevated temperature. Fig. 5 shows a cross section of the pile cap model clamped to the test base. A rubber chamber was used to produce water pressure and to act as a liner between the pile cap and the base slab.

From these studies, and assessing most of the published work on end slab behaviour, it became apparent that the possible mode of failure is the classical shear plug failure. Shear or sliding planes are developed such that a central plug is forced outwards and isolated from the main annulus of concrete. The position of the failure plane is governed, on the outer face, by the location of the vertical prestress, the inclination of the plane by the hoop prestress and span to depth ratio.

4.2. Behaviour of End Slabs Prior to Failure

The general behaviour of an end slab is described in Fig. 6 where the central deflection, as a percentage of the value of the ultimate is plotted against percentage of maximum pressure. The flexural behaviour of the slab may be divided into three stages. The first stage, elastic, is defined by the pressure at which the first radial cracks formed on the external surface. This pressure is almost twice the design pressure whilst the central deflection was equivalent to about 5% of the maximum (1/450 of the depth). Furthermore, tensile strains of the order of 3000 microstrain have been measured at the outer face of the slab before any visible cracking was observed.

The second stage, elastoplastic, was characterised by the development of external flexure cracks. The crack started at the centre of the slab and spread towards the edges of the model; on average they were fully developed at about three times the design pressure (74% of the ultimate strength). Further pressures caused secondary cracks to appear between the major radial cracks and accompanied by a rapid increase of deflection due to yielding of the tensile reinforcement in the central region.

In the final stage, yielding of the reinforcement was fully developed along all the radial cracks and the deflections increased considerably with a small increase in pressure; the maximum deflection varied between 10 and 12.5 mm (0.4 and 0.5 inch).

Although the flexural plastic stage was well advanced, it did not result in failure as another mode interceded, i.e. a central plug of concrete was extruded through the circular slab. (See Fig. 7.)

4.3. Summary of Results

The following are general comments obtained from the experimental investigation:

(a) An increase in the thickness of the slab in relation to its span has a dominant effect on the ultimate strength.

(b) Provision of hoop prestress or lateral restraint increases the ultimate strength of the slab but the rate of increase of strength progressively falls. At the magnitude of prestress required for producing acceptable stress levels under

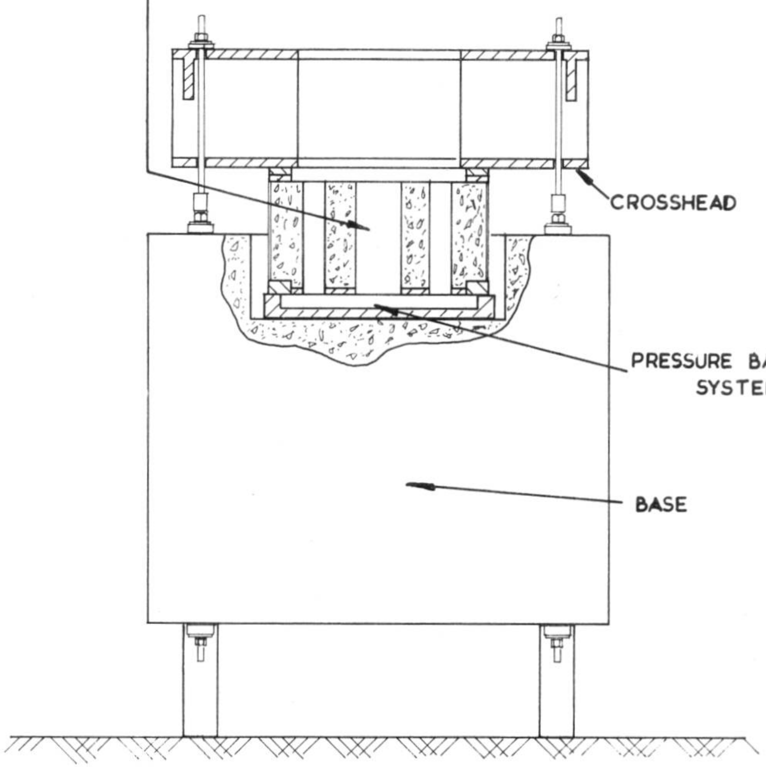


FIG. 11

TEST ARRANGEMENT FOR $\frac{1}{3}$RD SCALE MODEL BOILER

CLOSURE

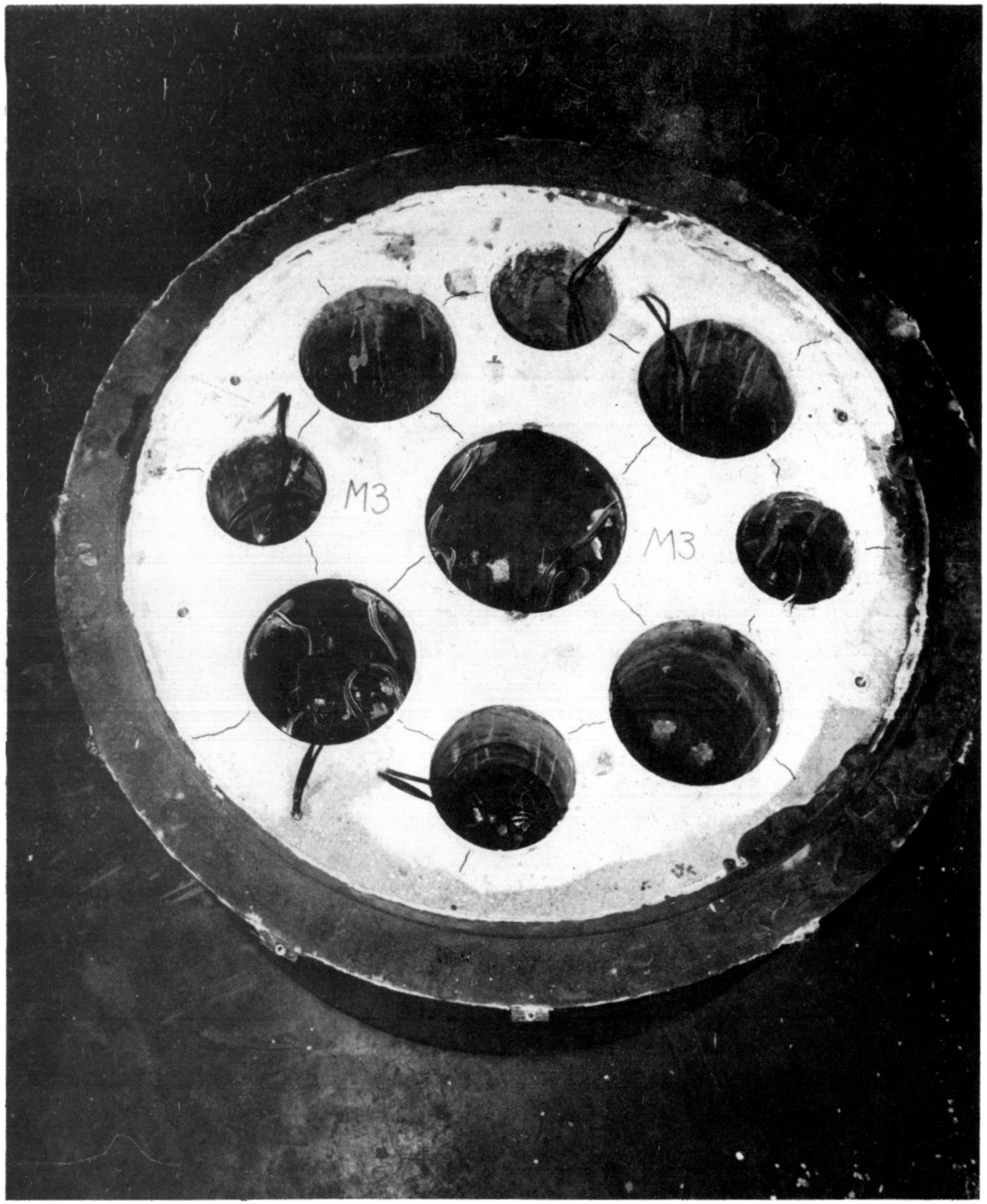


FIG. 12 1/3rd Scale Model Boiler Closure Crack Patterns

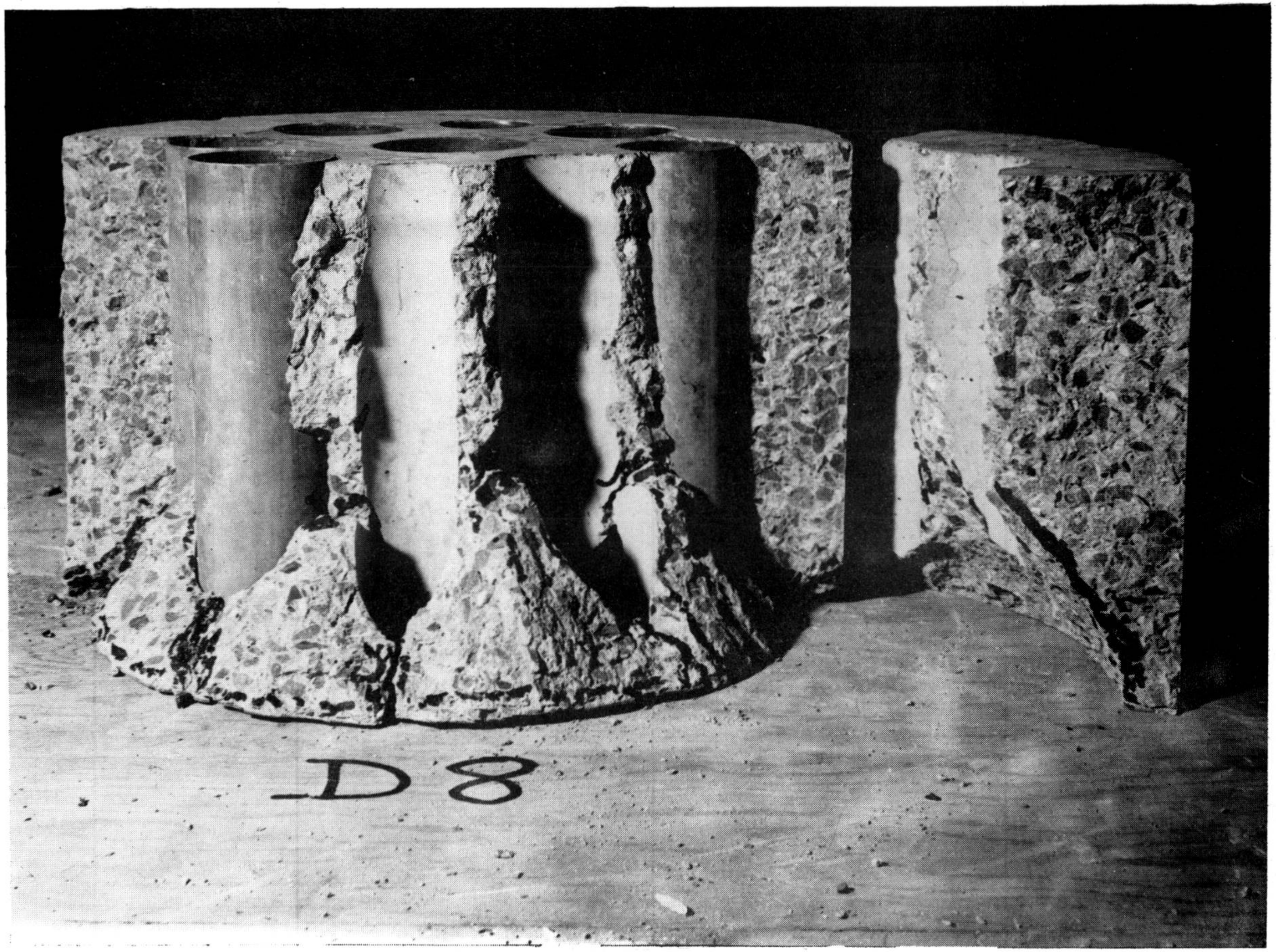


FIG. 10 1/10th Scale Model Boiler Closure Showing Failure Planes reactor working conditions the ultimate strength is not significantly affected by variation of the hoop prestress.

(c) When penetrations are incorporated in the slab, the liners partly compensate, by dowel action, for the loss of the area of concrete acting in shear. The loss of strength due to the introduction of a standpipe array is therefore not severe (17.5% loss in cross sectional area resulted in about 10% reduction in the ultimate shear strength).

(d) Surface reinforcement equivalent to 0.44% of the cross sectional area of the slab only increased the ultimate shearing strength by 3% presumably because of the dowel forces.

(e) By reducing the level of hoop prestress at transfer from 4.1 to 2.5 N/mm^2 (600 to 360 lb/sq.in.) whilst maintaining the same area of steel, the difference between the ultimate shear strength was only 10%.

(f) Sustained temperature has no effect on the ultimate shearing strength of concrete.

(g) For similar shapes of models the ultimate shear strength has been found to be approximately proportional to the square root of the cylinder strength of the concrete.

4.4. Ultimate Shear Strength Equation

An attempt was made to obtain an analytical expression which would give better correlation with the available published data on end slabs, including the author's work, and which takes into consideration the effect of the lateral prestress, depth of slab, diameter of pressurised area, as well as the strength of the concrete.

The following expression was found to fit the test data of 36 end slab models reasonably well:

$$P = 77\frac{d}{D}\sqrt{f_c} + 10\left(1 - \frac{D}{\phi}\right)^{\frac{1}{2}}\sqrt{f_h}$$

where P = ultimate pressure in lb/sq.in.
f_h = lateral prestress at transfer in lb/sq.in.
d = depth of slab
D = internal diameter of pressure vessel
f_c = cylinder compressive strength in lb/sq.in.
ϕ = outer diameter of pressure vessel

This equation is based on a small number of tests and can only be considered to be applicable to structural members similar to the test results and where lateral prestress is applied. With additional data a more generally applicable expression similar to the above could be developed.

5. DESIGN DEVELOPMENT OF BOILER CLOSURE

5.1. Analysis and Design of Closure

In developing the structural design of the closure, model analysis was preferred to a purely theoretical approach. The closure contains several closely spaced steel lined penetrations and other steel components. The combined effect of operating pressure and the active prestress produces a complex triaxial state of stress within the structure. An ultimate load analysis incorporating cracking and plasticity of the concrete is very difficult due to the limitation of establishing applicable criteria of failure for concrete. The only alternative was therefore to undertake model studies.

Initial examination of the design was based on a simplified axi-symmetric Dynamic Relaxation analysis under operating pressure and various extreme boundary conditions. This was supported by a rapid but extensive series of tests of 1/10th scale models to determine the likely effects of various parameters on the functional and ultimate behaviour of the closure. The intensity of prestress, quantity of circumferential steel, the effect of penetration liners, the system of support and other relevant factors were all examined. The guidance obtained from these models enabled a final design to be chosen.

As final confirmation, four fully representative 1/3rd scale models were tested to establish the load factor of the closures and then to investigate the effects on operational and overload behaviour of both cycled and sustained pressure and temperature.

5.2. 1/10th Scale Models

5.2.1. Test Programme and Procedure

The experimental programme described in Table 1 was phased to examine the following main parameters which are likely to affect the behaviour of the closure under working and overload conditions. The models were simplifications of prototype designs but were still representative for obtaining general conclusions. Sixty models were tested using the experimental arrangement shown in Fig. 8.

TABLE 1 1/10th Scale Secondary Closure Models - Test Programme

Shear Span (mm)	Prestress (N/mm2) None	Passive 0.5	Passive 1.0	Active 6.2	Steel Casing
229	*	*	*	*	*
290			*	*	*

* Two models tested with this combination of parameters.

To reduce the frictional restraint at the bearing ring, and achieve uniformity of bearing, a resin/polytetrafluorethylene interface was included on certain specimens.

The geometry of the specimens, penetrations and liners were scaled from an early prototype design in which the span to depth ratio was 1.27. Where the introduction of a reduced bearing area was incorporated into the tests, this ratio was increased to 1.42. The prototype closure design was finally fixed at 1.61. The penetration geometry has remained substantially unchanged.

Prestress was applied by a wire winding machine and consisted of close pitched windings of 0.94 mm (0.036 inch) diameter wire.

For active restraint, prestress was applied to give a mean stress on the cross section of 6.2 N/mm^2. This was achieved by one layer of wire approximately 165 turns wound at 65% of the ultimate tensile strength of the wire (U.T.S.) and anchored by pins set into the concrete.

The quantity of passive restraint required was estimated from the author's previous work on shear(3). In general, two layers of 165 turns were applied, with a limited examination on specimens with one layer. A nominal tension only was applied to maintain uniform wire lay; this was equivalent to about 4% of the U.T.S. of the wire.

Load to simulate the gas pressure was applied to the specimen via. heavy steel circular plattens of the diameters indicated in Fig. 8, the largest being equivalent to the total area which would be pressurised in the prototype. The reaction was taken by annular bearing rings.

An hydraulic system was also used with some of the models and consisted of a rubber platten operated by a pressurised oil filled diaphragm such that a uniform pressure was applied to the gas face of the model closure.

5.2.2. General Behaviour

The following are general comments on the behaviour of the models:

(a) The specimens initially behaved elastically in flexure until sufficient over pressure was applied to overcome the prestress compression and/or tensile strength of the concrete at the free face. (Fig. 9.)

(b) Radial dilations indicated that there was a substantial reserve of strain capacity in either active or passive prestress when shear failure occurred.

(c) Flexural cracks developed at the free face and the disposition was governed by the major penetrations which act as stress raisers. The elements so formed were held in equilibrium by the elastic external restraint and the high compressive stress state developed in the uncracked concrete adjacent to the gas face.

(d) As the closure continued to flex elastoplastically, the vertical shear stresses within the concrete increased. At the same time the shear capacity of the uncracked concrete at the gas face was significantly increased by the high lateral compressive stresses developed in this region.

(e) The shear strength was finally exceeded in the ligament between adjacent penetrations where this pitch-point has significantly concentrated the shear stresses.

(f) In the majority of the tests the mode of failure was by vertical shearing of the concrete along a plane formed by the pitch circle of the outer ring of penetrations. (Fig. 10.) Initial development of this plane, indicated by a rapid increase in the rate of vertical deflection did not occur in passively prestressed specimens until pressures above three times design was attained. In specimens having active prestress, this pressure was in excess of six times the design value.

(g) The use of polytetrafluorethylene in some of the tests only reduced the stresses by about 7% in the case of prestressed specimens.

(h) In the case of unlined penetrations the strength was reduced by about 23%

5.3. 1/3rd Scale Models

5.3.1. Test Programme and Procedure

Four 1/3rd scale models were tested using the arrangement shown in Fig. 11. The models simulated the prototype design as closely as was

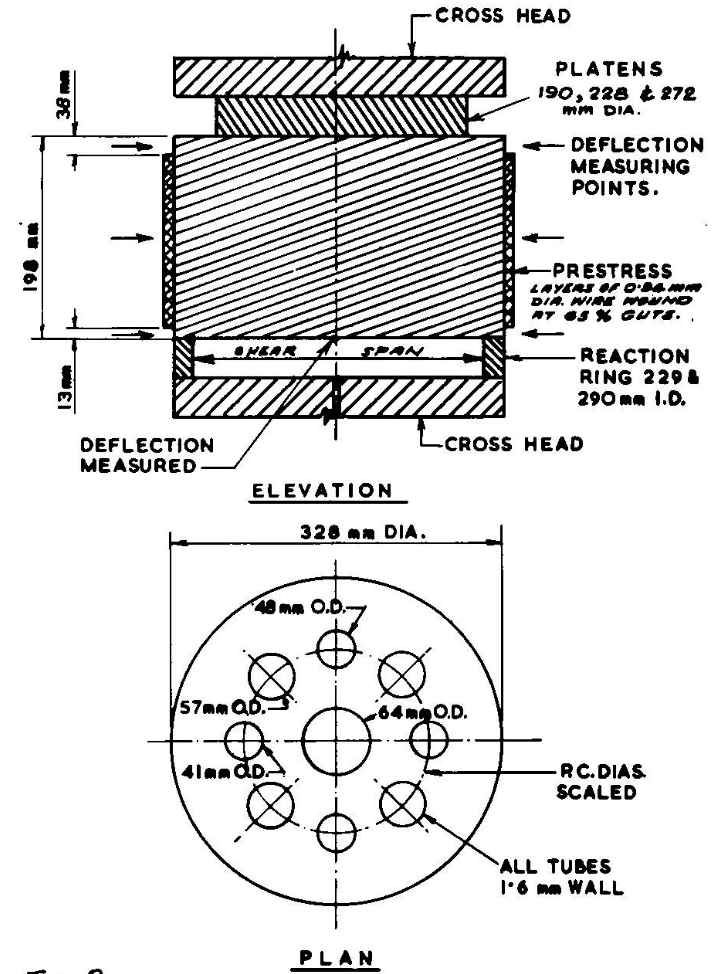


FIG. 8

TEST ARRANGEMENT FOR 1/10TH SCALE BOILER CLOSURE.

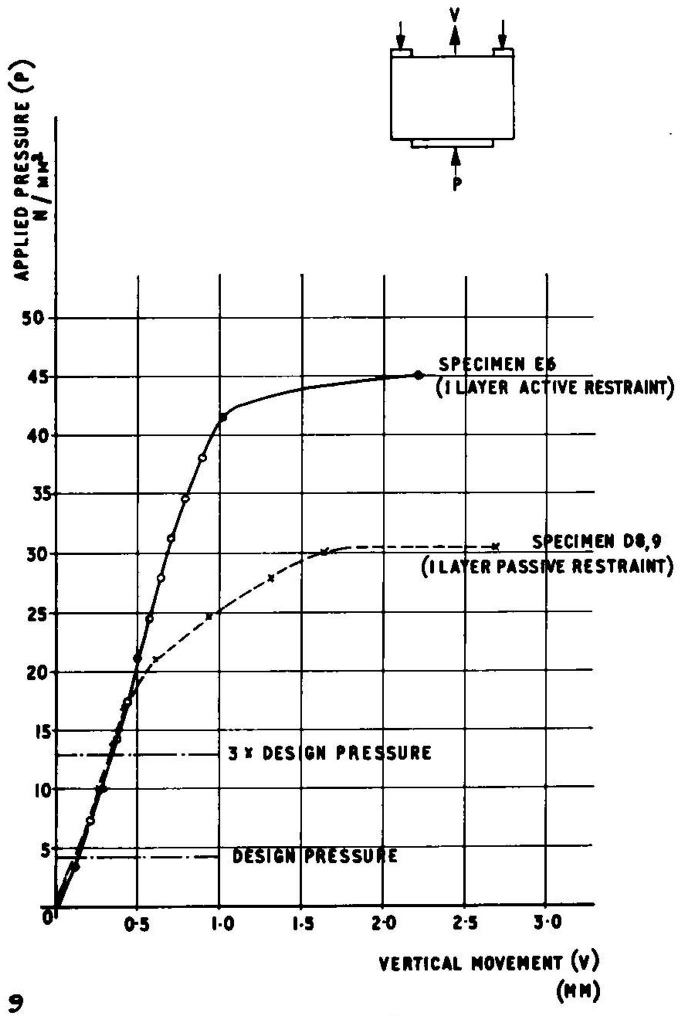


FIG. 9

SHEAR DEFORMATION CURVES FOR $\frac{1}{10}$TH SCALE MODEL SECONDARY CLOSURES

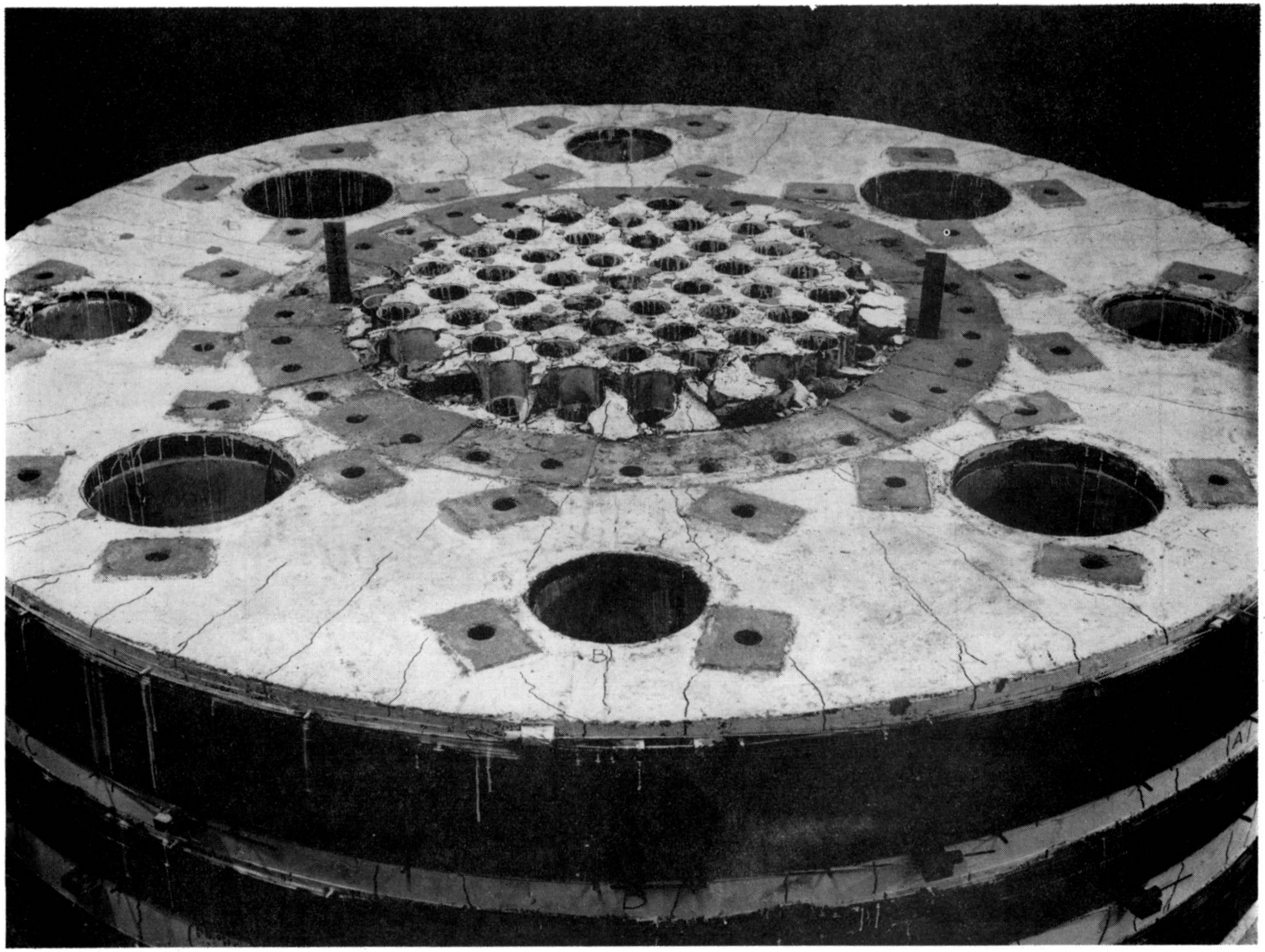


FIG. 7 Failure of Perforated Cap (Model M3)

FIG. 5

CROSS SECTION OF END SLAB MODELS
SHOWING TEST ARRANGEMENT

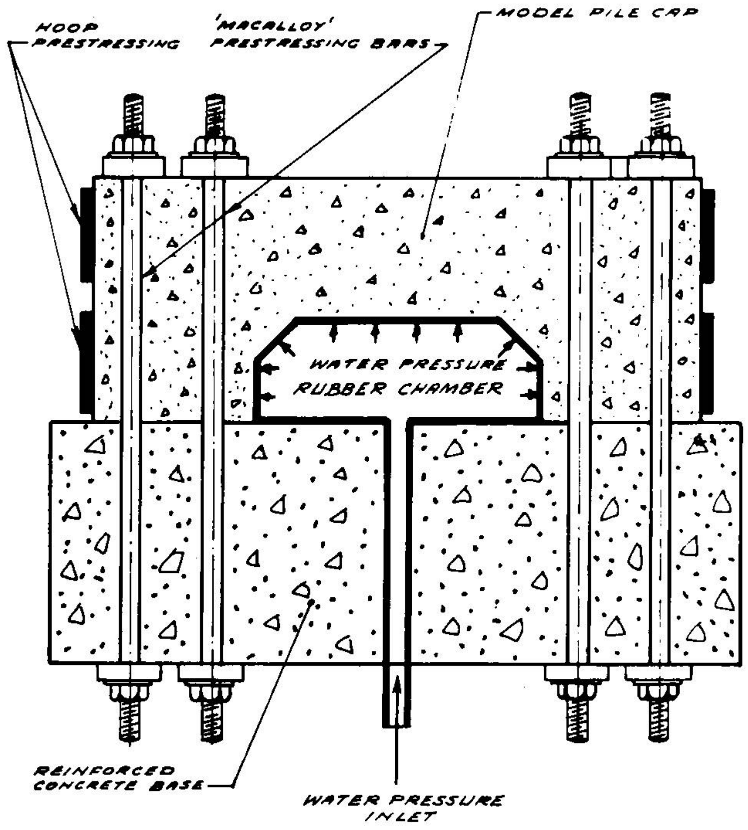
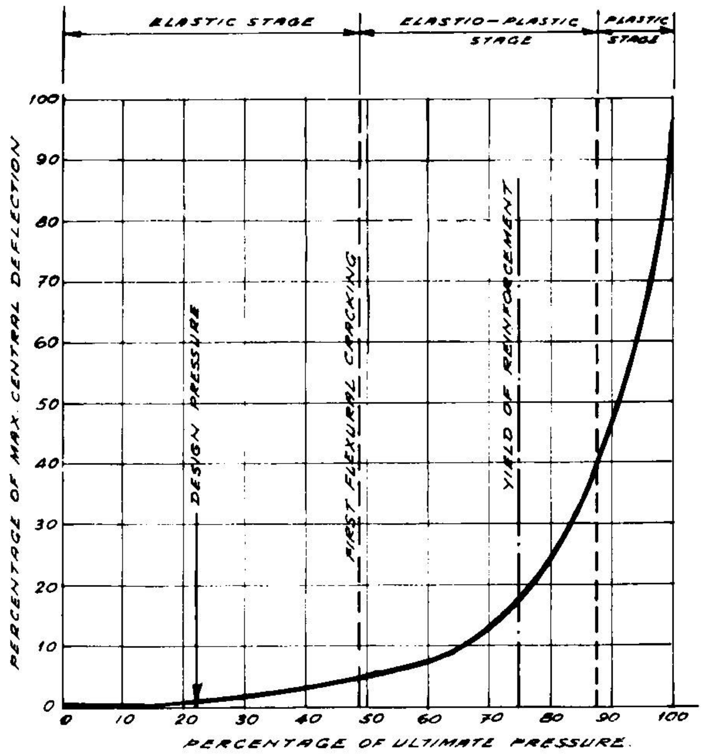


FIG. 6

FLEXURAL BEHAVIOUR
OF END SLAB



experimentally practicable. The test programme described in Table 2 was designed to examine the short and long term behaviour of the closure.

TABLE 2 1/3rd Scale Models - Test Programme

Model	Prestress	Loading
M1	Passive	Short term up to 6 times design pressure at ambient temperature.
M2	Active	Short term up to 6.5 times design pressure at ambient temperature.
M3	Active & Passive	(i) Sustained 50°C and 7 N/mm² pressure for a period of 9 months. (ii) Overload test.
M4	Active &	(i) 54 temperature cycle between 24°C and 50°C. (ii) 10 temperature cycle combined with pressure cycle between 0 and 7 N/mm². (iii) 10 pressure cycle and ambient temperature. (iv) Overload test.

The prestress consisted of close pitched windings of 0.94 mm diameter wire, one wire representing almost exactly the scaled area of one prototype 2.6 mm (0.104 inch) diameter wire. Each layer of wire was individually anchored.

The passive restraint consisted of six layers and, when combined with active prestress, it was wound at a nominal tension designed to compensate for creep, shrinkage and elastic shortening. In the test in which passive prestress was examined in isolation, windings were at a tension of approximately 5% of the wire U.T.S. to facilitate uniformity of lag.

The gas pressure was simulated by water pressure applied at the lower surface of the model closure through a contained reinforced rubber pressure chamber. Pressure was applied over an area equivalent to that lying within the inner sealing ring of the prototype.

The reaction was taken via. the bolt system to a structural steel crosshead restrained by 42 No. 32 mm Macalloy bars.

In the prototype, the main source of heat occurs at the gas face, and this was simulated in the models by resistance elements cast into the concrete 50 mm from the pressurised face. Each element was duplicated.

The rate of temperature rise to 50°C did not exceed 5°C per hour to avoid thermal shock. Under temperature cycling, the models were allowed to cool naturally along the diameter and the adjacent sides. Strains were measured in the concrete and on a number of selected penetrations.

5.3.2. Results and Behaviour of Models

The following are very brief comments obtained from the test results:

(a) Models M1 and M2 withstood maximum pressures of 26 and 29.5 N/mm^2 without failure which gave a load factor of 5.90 and 6.55 respectively.

(b) At the maximum pressure, both M1 and M2 were tending towards the anticipated shear plug failure.

(c) At pressures varied between 5.5 and 9.5 N/mm^2 fine radial cracks were observed in three of the four ligaments between the main and superheater penetrations of models M1 and M2. These cracks (Fig. 12) were fully developed, but with a maximum crack width of 0.25 mm, at pressures of 13 and 16 N/mm^2 for models M1 and M2 respectively. All cracks, however, appeared to exhibit complete recovery on depressurisation.

(d) At five times the design pressure, the central deflections for models M1 and M2 were only 0.036% and 0.18% respectively of the depth of the models. (Fig. 13.)

(e) Both M3 and M4 were shown to be structurally stable after sustaining the severe overload conditions for the tested period. Up to four times the design pressure the central deflections of these models were very similar to those of M2. (Fig. 14.)

(f) The rate of creep development for models M3 and M4 was shown to be substantially that predicted from control specimens(5). (Fig. 15.)

(g) In the final tests, both M3 and M4 withstood pressures of six times the design pressure without failure. Subsequent sectioning revealed negligible fracture, and both models were considered capable of withstanding higher overload pressures.

(h) Model behaviour was compared with that predicted by the axi-symmetric analysis. Under the simplest loading conditions, e.g. hoop prestress only, measured hoop strains in zones of mass concrete were within 4% of the predicted values.

5.4. Comparison Between 1/3rd and 1/10th Scale Model Behaviour

Small scale models were used to optimize the main parameters for the final design of the boiler closure. These parameters were then incorporated in the 1/3rd scale models. Direct comparison between the behaviour of the two scales of models is limited by variations in some of the key factors, e.g. depth, loading and boundary conditions and material properties. However, supplementary tests on the 1/10th scale models showed that none of these variations influenced the ultimate strength by more than 15%. Thus, if these factors are taken into consideration, a realistic comparison is possible.

6. CONCLUSIONS

6.1. The paper describes the behaviour of structural components in which stresses are utilised, to greatly enhance the shear strength of deep concrete sections.

6.2. Two examples are given of the use of model analysis as a design tool in determining the long term and ultimate load behaviour of complex structural components. This technique will continue to be a basis for designing such structures until our fundamental knowledge of concrete behaviour under multiaxial stress states is more rigorously defined.

FIG. 13 - COMPARISON OF CENTRAL VERTICAL DEFLECTIONS FOR 1/10 & 1/3 SCALE CLOSURE MODELS.

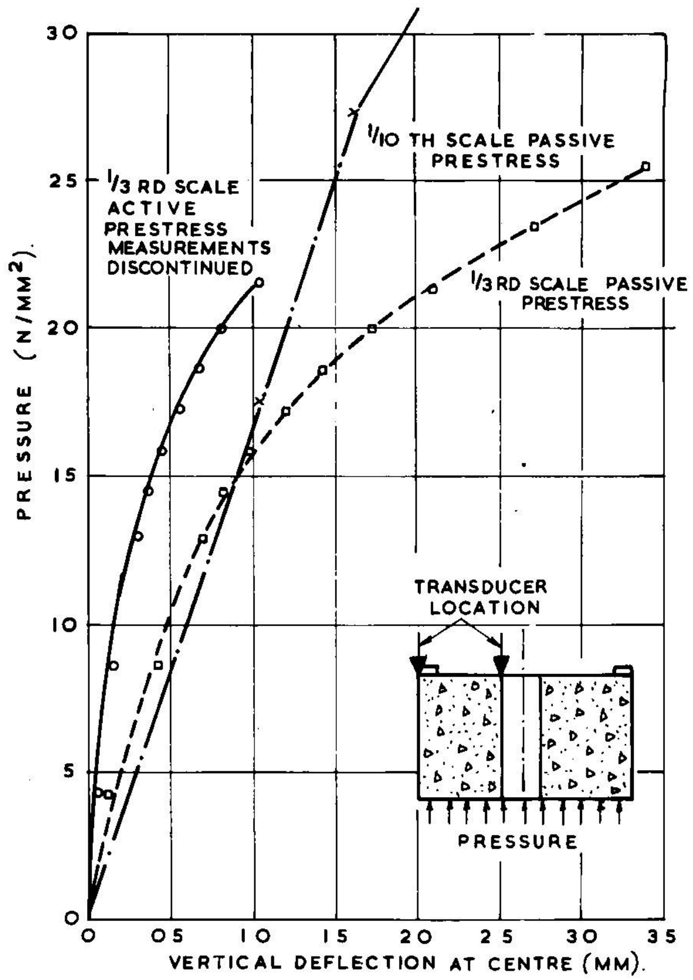
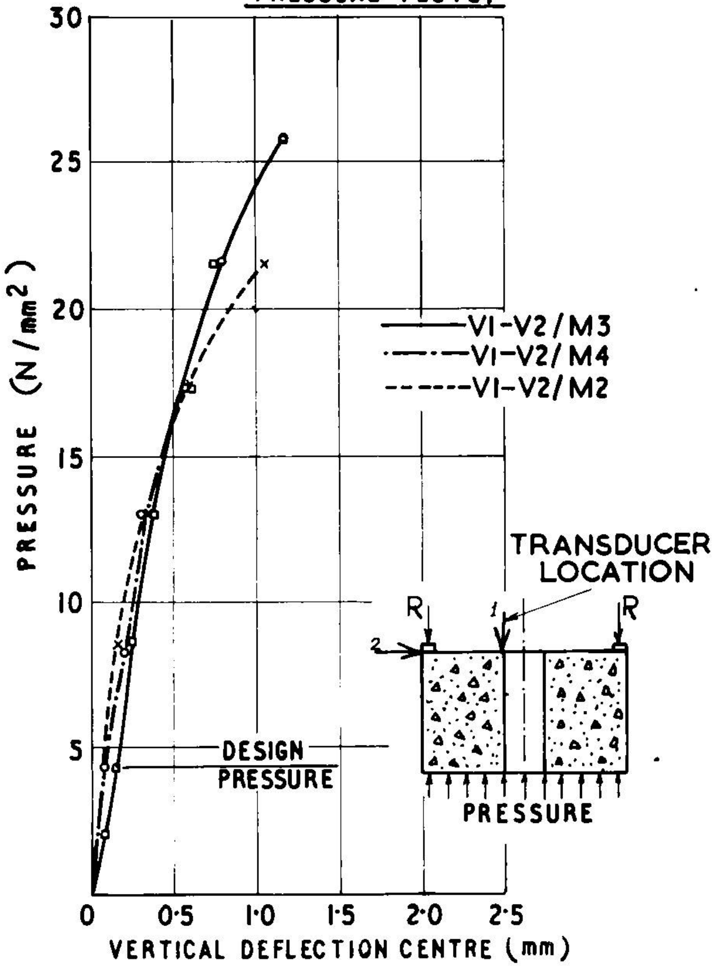


FIG. 14 - ONE THIRD SCALE CLOSURE MODELS M2, M3, M4. CENTRAL VERTICAL DEFLECTION IN FINAL PRESSURE TESTS.



MODEL M3 GAS FACE

THEORETICAL & MEASURED HOOP STRAINS

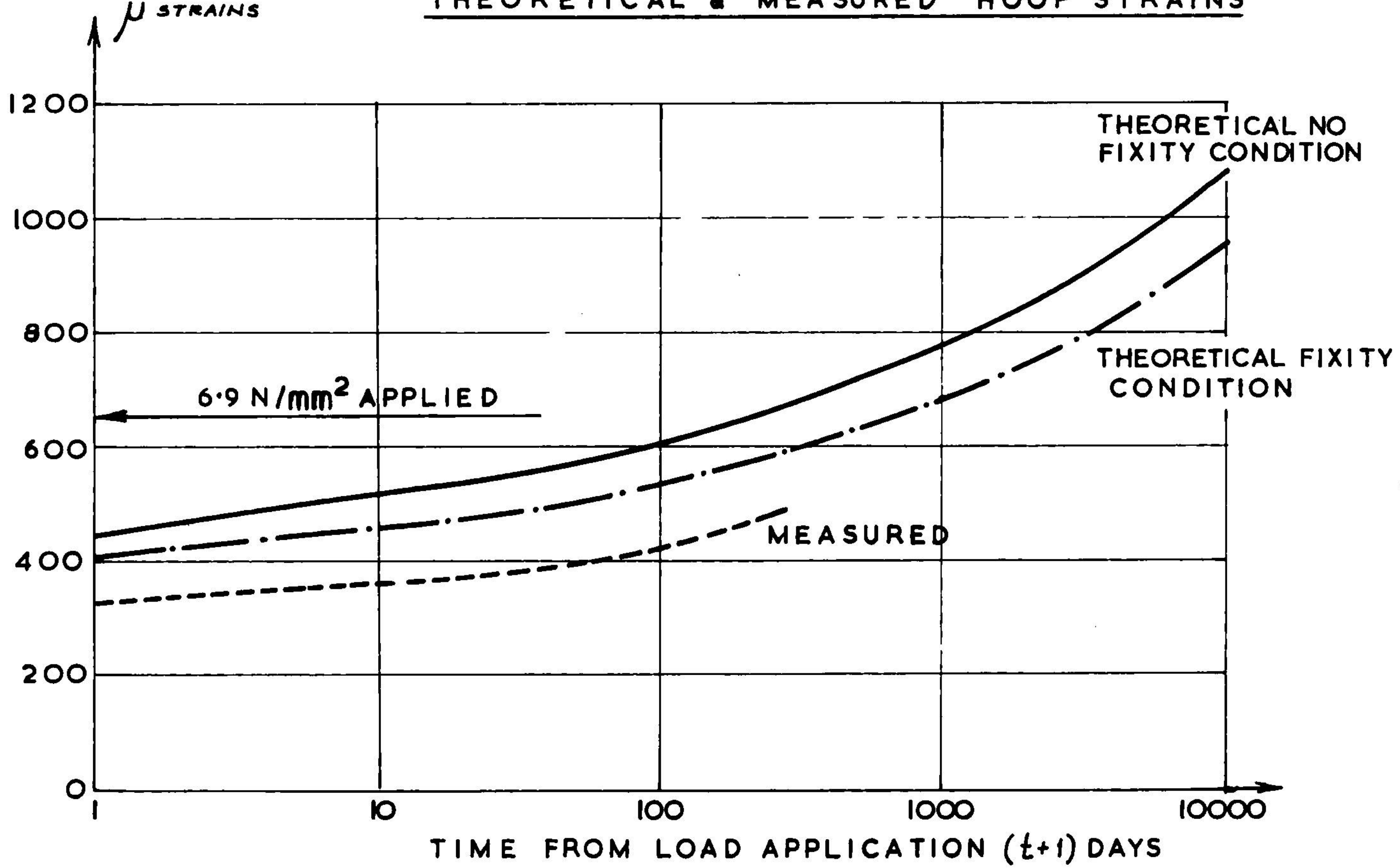


FIG. 15

7. REFERENCES

(1) Burrow, R.E.D. and Williams, A.J. "Hartlepool AGR Reactor Pressure Vessel" Nuclear Engineering International, Nov. 1969.

(2) Langan, D. and Garas, F.K. "Behaviour of End Slabs in Cylindrical Prestressed Concrete Pressure Vessels" Proceedings of the First International Conference on Structural Mechanics in Reactor Technology, Berlin, 1972.

(3) Langan, D. and Garas, F.K. "The Failure of Concrete Under the Combined Action of High Shearing Forces and Biaxial Restraint" International Conference on Structures, Solid Mechanics and Engineering Design in Civil Engineering Materials, Southampton, April 1969.

(4) Fenwick, R.C. and Paulay Thomas, "Mechanism of Shear Resistance of Concrete Beams" Journal of the Structural Division, Proceedings of the American Society of Civil Engineers, Oct. 1968.

(5) Browne, R.D. and Blundell, R. "The Behaviour of Concrete in Prestressed Concrete Pressure Vessels" Proceedings of the First International Conference on Structural Mechanics in Reactor Technology, Berlin 1972.

ACKNOWLEDGEMENT

The author wishes to thank Taylor Woodrow Construction Limited for permission to publish this paper.

SUMMARY

The paper describes the mechanism of the shear resistance in restrained concrete elements. Details are given of two experimental investigations where shear stresses were dominant. In the first, the general behaviour of 23 end slab models is summarised and an empirical formula is introduced which was found to fit most of the available data on this subject. Secondly, the paper describes model tests undertaken to aid the design of a boiler closure. Sixty 1/10 scale models were tested to give a quick assessment of the influence of main structural parameters. These were followed by short and long term studies on four realistic 1/3 scale models to confirm the final design. Details of the investigation are given together with the general behaviour of the models during various stages of testing.

RESUME

Le rapport décrit le méchanisme de résistance au cisaillement des éléments contenus en béton. On donne des détails sur deux études expérimentales dans lesquelles les efforts de cisaillement étaient dominants. D' abord on résume le comportement général de 23 maquettes de dalles de fermeture et on présente une relation empirique qui s' est démontrée en accordance avec la plus part des résultats expérimentaux à disposition au sujet.

Ensuite le rapport décrit des essais sur maquettes réalisés afin d' aider le projet de fermeture de chaudière. On a essayer 60 maquettes en échelle 1 : 10 afin d' obtenir une évaluation rapide de l' influence des paramètres structuraux plus importants. Ensuite on a réalisé des études de longue et courte durée sur 4 maquettes réalistiques en échelle 1 : 3 pour confirmer le projet final. On donne des détails sur les études réalisées et sur le comportement des maquettes durant les différentes phases d' essai.

ZUSAMMENFASSUNG

Der Artikel beschreibt den Mechanismus des Schnittwiderstandes in Spannbeton-Komponenten und schildert die Einzelheiten von zwei Versuchsstudien, in denen Schnittbelastungen vorherrschend waren. Zuerst wird in Kürze das Verhalten von 23 Betonsohlen-Modellen beschrieben und eine empirische Formel eingeführt, die auf die meisten, für diesen Fall zur Verfügung stehenden Resultate passt.

Es folgt eine Beschreibung der Tests an Modellen, die eigens dafür angefertigt wurden, um einen Boilerverschluss zu entwerfen. Sechzig Modelle im Mass-Stab 1 : 60 wurden getestet, um schnell den Einfluss der wichtigsten Struktur-Parameter bewerten zu können. Darauf folgen kurz- und langfristige Studien an vier realistischen Modellen im Mass-Stab 1 : 3, um die Gültigkeit des endgültigen Modells zu bestätigen. Die durchgeführten Studien und das allgemeine Verhalten der Modelle während der verschiedenen Test-Phasen werden ausführlich beschrieben.