

**THIN-WALLED PRESTRESSED CONCRETE PRESSURE VESSEL
FOR HIGH TEMPERATURE REACTORS:
EXPERIMENTAL INVESTIGATIONS ON THREE 1:20
SCALE PCPV MODELS AND DESIGN PHILOSOPHY PROPOSAL**

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SUMMARY

Safety Authorities requirements, asking for an additional external secondary containment for the Reactor Building, have increased the cost of the High Temperature Reactor (HTR) "filière".

The earthquake effects on such massive structures, very important for many countries, and other not less important aspects concerning the wall thickness, compel the designers to better exploit the resources of the materials.

The paper gives the basic results of tests performed on the 1:20 PCPV model test announced at the 1st SMIRT Conference and available information on the result that will be ready at the time of the 2nd SMIRT Conference on two additional 1:20 scale confirmatory models, identical to the previous one but without penetrations.

The triaxial state of stresses in the structure is checked by taking into account the results of triaxial tests carried out by the ENEL Research Laboratory of Niguarda (Milan-Italy) with the method already described at the Rilem Symposium of Cannes (France) Oct. 1972.

The models refer to an advanced design of PCPV devised by the author.

The paper deals with a proposal of "design philosophy" of PCPV and the model test data are presented with the aim to support the suggested design criteria.

The results of a sophisticated finite element calculation of the referenced 1:20 scale models are checked and compared with the results obtained from the tests.

Comments are made regarding the reliability to adopt constant modulus of elasticity and Poisson ratio for elastic calculations (assuming that the model can be considered a design tool in which the actual concrete properties are modelled).

The paper evidences the behaviour of the model in the ultimate conditions. Considerations are made about the possibility of driving the failure by adequately designing the prestressing cable system.

Having the author attended, as an observer, to the tests (during the "commissioning") of an European PCPV, suggestions are made in order to improve the reliability of the full scale tests, on the basis of the experience acquired on small scale models testing technique.

1. INTRODUCTION

The cost for the whole set of a conventional cold PCPV for High Temperature Reactors (including heat exchangers and blowers) varies from 4 to 8 million dollars depending on reference Nuclear Plant unit power.

In examining the possibility of cost reduction we find that the "cable prestressing system" represents one of the most expensive items.

Considering that the existing PCPV have been designed with a $130 + 180$ Kg per cubic meter load of steel for cables and with a wall thickness even three times greater than needed as a biological shield ($2 + 3$ m), it easily comes that a very important cost drop can be achieved in reducing the wall thickness and as a consequence steel quantity, penetration length, Reactor Building dimensions, foundations and relevant antiseismic devices (belt tendons, etc.), air conditioning volume, etc.

From our experimentation on small scale models of a conventional PCPV (thick type) we found that the design method actually adopted tends to lead to structural oversizing. Therefore the research was oriented to design and test, on small scale models, an advanced and economic version obtained with a wall having the thickness required for biological shielding (cylindrical walls). Thus we call "thin" the advanced solution in comparison with the "thick" old one. Comparison between the conventional and thin solution is shown in fig. 1. The economical and technical advantages are self evident.

In the following chapters we will try, within the limits of this paper, to summarize some aspects of our research and to propose for discussion some lines of design philosophy.

We deem advisable to start our exposal by commenting the ultimate conditions in the models.

2. STRUCTURAL COLLAPSE AS SHOWN BY MODELS

The ultimate tests on models are at the same time the least and most significant tests that we performed. Least significant tests because, of course, they do not represent any possible loading condition in the reactor life; but most important ones because, in our opinion, they are the most entitled to direct a conscious design.

Least significant because our models are pressurized by means of water instead of gas, but also significant because we can deduce the crack topology inside the vessel and therefore we are in the best conditions to direct an adequate design of the steel liner: and the liner is the only responsible of the tightness and by consequence of the actual structural collapse limit. Least significant because it is a nonsense to talk about a sort of "rupture day" in the life of the reactor (and therefore to simulate the actual state of stresses on such "day"), but also the most significant means of discussion on safety margins and ultimate behaviour, in order to satisfy in a convincing way, we believe, designers, purchasers

and licensing Authorities. The reason why we start talking about the ultimate conditions is the fact that, in our opinion, the best way of fulfilling the economic commitments and the design rationality is to know the final consequence of our initial decision.

The first decision is the choice on the shaping of the PCPV; we refer for comments to the geometry of our models that are aligned with the recent developed geometry, consisting of prestressed cylinders with plugs on both ends, which we consider at present the best compromise between design and layout requirements. The model tests show that:

- a) the structural collapse is strictly related to the collapse of the cable steel (for the tested geometries, no punching effect was experienced in the cap slabs). For the thick model, the collapse was due to the breakage of the vertical cable system and consequential separation of the unit in the two single components: top cap and cylindrical walls. For the first two tiny-walled models, breakage of the hooping cable system occurred: at the height of the blowers' main penetration (weakest section) for model no. 1, and at the equator for model no. 2, without wall penetrations (fig. 2);
- b) thick and thin solutions have put in evidence localized cracks in the gusset areas confirming the natural tendency, for these geometries, to separate the structure thus shaped into two fundamental components (caps and cylindrical walls). No visible crack was detected in the cap slab portion corresponding to the internal diameter;
- c) from tests on cap slabs, we have experienced that when pressure is rising, they show a gradual transformation into an "inverse dome" through successive structural partializations (cfr. "Techniques for Rupture Testing of PCPV Models" - British Nuclear Energy Society, London, 1969. Conference of 10-11 July 1969. Paper no. 4);
- d) the most important cracks are originated and propagate from specific regions of the structure like gussets, penetrations, sudden variations of geometry;
- e) from our general research on complete models and their portions (cap slabs and wall ring) we have found that the pressure value, representing the tendons collapse limits of the PCPV structure, is about the 70% of the pressure value that leads to the tendons' collapse of single elements. The thick model collapsed at a pressure of 192 Kg/cm^2 . whereas single elements collapsed at higher values: the ring alone at 262 Kg/cm^2 and the cap slab at 275 Kg/cm^2 (confirming the good balancing of the design).

The above statements give us some suggestions for the designing phase:

3.1. Geometry

As concerns the cap slabs, we realize that the inverted dome shape is the most rational, which gives in addition the economic benefit to reduce the concrete quantity and the penetration lengths. The lack of cracks and secant prestressing cables in the cap slabs

suggests to concentrate the main penetrations on such structures.

Penetrations in cylindrical walls are certainly feasible but the lack of circumferential cables at their levels suggests, especially for "thin" solutions, to take care of local effects and by consequence, to provide adequate local reinforcements.

Generally speaking, a geometry as axialsymmetrical as possible should be chosen, in order both to minimize the design problems (related to the approach of the mathematical model to the physical one) and at the same time to avoid the sudden variation in the geometry so as to reduce the critical points of initial cracking.

3.2. Prestressing cable system

The models are prestressed with "post-tensioned" cables, embedded in the concrete; anchorages BBRV system. The hooping cable pattern, designed by the Author (London Conference 13-17 March 1967, Paper no. 27) tries to minimize the ovalizing effects of the anchoring heads and utilizes these reactions to provide additional centripetal forces (fig. 3). This type of cable pattern will certainly drive the rupture modality when the two fundamental systems (circumferential and vertical) are balanced to counteract the same pressure load under the ultimate conditions (economical input of a well balanced design). In fact, the rupture of the vertical or circumferential system depends on the route the cracks follow in their propagation across the structure up to determining the big deformations able to elongate the wire to their breaking limits (many tens of centimeters) and in turn the crack propagation depends on cables layout. We believe that cables embedded in concrete represent a very good solution because of the linking effect on the external concrete bark. In the model, we are forced, for scale reasons, to reduce the number of cables, that is to improve their unit power. Therefore the equivalent unit power of the cables referred to the full scale prototype will be of either 1600 tons or 2200 tons.

Such heavy unit cables behave excellently in the model and we are strongly convinced that this aspect is worth to be taken into consideration for the design.

3.2.1. Prestressing

The prestressing operations (prestressing sequence) are carried out adequately in order to maintain always a compressive state in the concrete structure.

Tiny-walled no. 1 model (with penetrations) was provided with conventionally adopted prestressing steel. Tiny-walled no. 2 model (without penetrations) was provided with stabilized prestressing steel. This choice was made in order to ascertain the effect of the plastic reserve in the steel wires on the "ultimate" of the structure. The losses are in the ratio of about 6% for the stabilized steel and 12% for the normal steel (at 200 hours). For these two "thin" models it was decided to apply a very high initial prestress in order to test a design limit condition and then to minimize the quantity of steel

and the corresponding collapse limits. The ultimate tests have shown that apparently there is no difference between using stabilized steel or conventional steel. The former offers the economic advantage of reducing relaxation losses and steel quantity, but leaves smaller plastic reserve due to the stabilization process. In fact both models collapsed at about 120 Kg/cm^2 . In our opinion, this behaviour suggests to select stabilized steel for design, and choose, for initial prestress, the highest pull value allowed by regulations in such condition. Our model cables are stretched at 80% of U.T.S.G. steel.

Fig. 4 shows how the cable systems of the model (circumferential and vertical) withstand pressure. The first and most remarkable aspect, that clearly appears from the examination of the figure, lies in the fact that the theoretical pressure at which the cable system should break is fairly inferior to the one that actually results from the ultimate tests. More precisely:

$$\text{thick model: } \frac{179}{192} = 0.93 \text{ (vertical system), thin models: } \frac{84}{118} = 0.71 \text{ (horizontal system)}$$

It derives that:

- a) the theoretical ultimate of the cable do not correspond to the ultimate of the actual cable collapse in the structure;
- b) the ratio z between the pressure corresponding to the ultimate of the cables and the working pressure is always higher than 2; namely:

$$\text{thick model } z = \frac{179}{40} = 4.5, \text{ thin models } z = \frac{84}{40} = 2.1$$

- c) for reference models, the safety factors related to the actual collapse of the cable systems are:

$$\text{thick model } k = \frac{P_c}{P_w} = \frac{192}{40} = 4.8, \text{ thin models } k = \frac{P_c}{P_w} = \frac{120}{40} = 3$$

As a consequence, the ultimate of a PCPV cannot be guessed on the only basis of theoretical cable resistance. The additional margin of resistance should be attributed to mild steel reinforcement, mild steel cable ducts, combined effects of cable systems, and, last but not least, concrete resistance to tension. This aspect suggests, for the design, to make use of the mild steel cable ducts tensile resistance, not at all a negligible contribution, that is to select a steel pipe cable duct instead of the conventional flexible light one, which does not offer any tensional resistance.

Another interesting aspect that shows in the said fig. 4, lies in the fact that it is impossible to predict which system will be most likely to break; in fact we read the following pressures, which correspond to the breakage of the predicted system:

	thick model	thin models
- circumferential system:	135 Kg/cm ²	84 Kg/cm ²
- vertical system:	180 Kg/cm ²	97 Kg/cm ²
- broken system:	the vertical one	the circumferential one

The real ultimate of the structure probably can be reached at a value, in the reference case, inferior to the structural collapse limit, by stopping the pressure at that level and waiting for the progression of the collapse. This limit corresponds to the yield of a significant number of cables. Let us deepen the point: from the readings of loads cells on cable anchoring heads in model tests, it is possible to define exactly the pressure which corresponds to the first conventional yield in the cable systems' steel (0,2%). This limit is very important because it can be considered the boundary between respectively the small deformations and the upstart of big ones in the concrete structure.

Below this pressure limit, down to the one which corresponds to the first visible cracking of the structure, the cracks developed in the concrete structure can be considered nonsignificant for the liner integrity because of the elastic response of the cable systems. We call this pressure range the "small cracks range". Above this pressure limit, we have the "big cracks range". In this range we have to expect the development of the important cracks, able to disconnect the structure which in turn allows the main elongations of the tendons up to their breaking limits.

The maximum pressure level we reach in our tests is strictly related to the speed rate of pressurization we use (3 Kg/cm² per minute from 115 Kg/cm² upwards).

As noted before, an intermediate pressure must exist, which corresponds to the yield of a significant number of steel cables and must be the one producing the progression of the structural deformations up to the collapse. In any case, in the range of the big cracks, the liner integrity will be lost; therefore on the safety point of view, this pressure range should be disregarded and consequently the real PCPV safety factor should be considered under this respect. From our tests we have found the following pressure ranges that were guessed for the thick model:

	thick model	thin models
- small crack range:	90 ÷ 160/180 Kg/cm ²	70/82.5 ÷ 110/90 Kg/cm ²
- big crack range:	160 ÷ 192 Kg/cm ²	110/90 ÷ 120 Kg/cm ²

Therefore the actual safety factor K for the two models (liner integrity limit) can be considered to be respectively:

$$\text{thick model } K = \frac{160}{40} = 4, \quad \text{thin models } K = \frac{90}{40} = 2.25; \quad \frac{110}{40} = 2.75$$

Since the small deformation limit represents the structure's real safety limit, we

are capable of driving the design ultimate conditions, by increasing the steel quantity, which corresponds to an increase of safety margins. This additional quantity of steel can be assigned to one or both prestressing systems in order to hypothesize structural collapse of one type or another. This choice, which implies higher costs, should be matter of discussion with safety people and, in our opinion, this kind of discussions can be adequately supported and evidenced only by means of models.

3. ELASTIC BEHAVIOUR AS SHOWN BY MODELS

In order to show in a synthetic way the model behaviour in the elastic field, we plotted in fig. 5 a comparison between the aequatorial (Δr) and vertical (Δz) deflections either for thick and thin models. This figure puts in evidence the two ways, expressed by the structure, of withstanding the pressure which corresponds to the two slopes expressed by deformations in the small deformation field. In turn, such two slopes represent the response of the two fundamental components of the structure: concrete and prestressing cable steel. Both models do not only fulfill the commitment to withstand elastically to the incidental conditions (48 Kg/cm^2), but show additional margins and a quasi-linear behaviour up to and beyond $60 \text{ Kg/cm}^2 = 1.5 P_w$. In fig. 5 there are also listed the reversibility pressure limits, that is the pressure limits reached during the tests, from which, by reversing to zero pressure, it was still possible to obtain, in the operating pressure range 0 to 40 Kg/cm^2 , exactly the same quasi-elastic behaviour experienced before trespassing the design conditions (40 Kg/cm^2).

It is worth to be noted that despite the great reduction in thickness achieved for the thin solutions, the deflections listed in fig. 5, remain comparably very small and within the acceptable limits for operating purposes. A comparison between the deflections, both calculated by considering the PCPV as an axial symmetric isotropic body and read on the models, show quite a good agreement in the results. The average modulus of elasticity that better matches with the model deflections is higher than the one derived from a mono-axial test on reference model concrete samples. This is probably due to the confining effect of the triaxial state in the structure.

I am firmly convinced that for the loading conditions generally adopted for gas reactors (incidental condition $p_i = 1.1 p_d$, p_d = design pressure) and for the said geometry, it is possible to design the PCPV in a satisfactory way by limiting the analysis to the so called elastic fields, that is, to perform elastic calculations. Once we respect the conditions of having the structure fully compressed at 40 Kg/cm^2 and adequate margins in the prestressing steel, as for instance suggested in the chapter 3, automatically the ultimate conditions will be within the acceptable limits. About the acceptability of the state of stresses in the structure for the thick and thin model no. 1, we base our check

control on the French Mohr-Caquot method to define the safety margins of a triaxial state of stresses in such structures. In fig. 6 the "contours" of safety factors N for thin models are given for values of N up to 5 ($N = 1$ is the breaking condition).

As an example, the safety factors for some typical points are respectively for the thick and the thin models:

5.41 - 3.44 5.92 - 4.83 5.85 - 2.38 230.95 - 17.23 and so on

As a matter of fact, the Caquot method as checked on models can be defined satisfactory for practical purposes, but does not satisfy our needs as concerns interpretation of the physical aspects of the phenomena. ENEL research laboratory of Niguarda (Milan, Italy) is working at this matter, in a general perspective. First results of their research were presented at the RILEM Symposium of Cannes (France) in October 1972.

This research is directed not only to define, for a given concrete, the regions of different behaviour (physical state) and the corresponding triaxial state of stresses (like discontinuity limits, stable or self propagating cracks regions) but also by means of suitable instrumentation to search the governing laws. As a first approach, the input coming from this research on model concrete specimens will be inserted in the computer program and results will be compared with model experimental data. Special efforts will be devoted to take into consideration the route followed by a typical structural element to reach its final state of stresses (Historical loading sequence).

This stressing route can in fact interest different regions, therefore the knowledge of the relevant governing laws is essential, should we try to interpret the physical reality by means of computer tools.

We are working in this direction and our perspective is to find the appropriate techniques and methods for a rational design, based on suitable calculation tools and supported by adequate information on reference material and checked for the elastic behaviour, then integrated for the rupture analysis by small scale models.

We are working, to give preliminary information on such research, for IABSE Seminar on Concrete Structures subjected to Triaxial Stresses to be held in Bergamo (Italy) next May 1974.

4. CONCLUSIONS

The model represents, in my opinion, the best calculation tool we can have at present, capable of covering all the fields from the elastic range to the ultimate one.

The best way to control model reliability should be to compare the model behaviour with a prototype. We do not have PCPV prototype at present in our country, but we are prepared to examine a collaboration in this field.

Our research was made not only to investigate in the field of PCPV, but in general

to deepen our knowledge both in the field of structural behaviour in triaxial state of stresses, and in testing and design techniques.

The fundamental aim is to reduce our ignorance coefficients, thus to improve our design and cut down our costs.

Within the next year the detailed results of our research will be condensed and collected for publication.

AKNOWLEDGEMENTS

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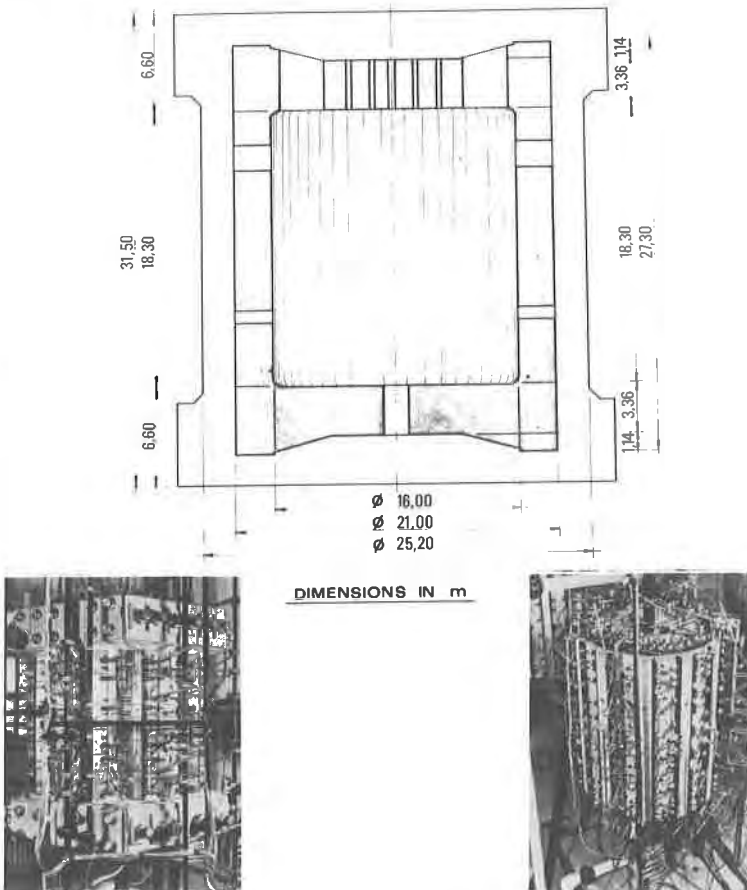


FIG. 1 COMPARISON BETWEEN THE THICK AND THE THIN SOLUTIONS

PC.PV. FOR H.T.R.



THIN MODEL N.1 (WITH PENETRATIONS) THICK MODEL THIN MODEL N.2 (WITHOUT PENETRATIONS)

FIG 2 PC.PV. THICK AND THIN MODELS AFTER COLLAPSE

PC.PV. FOR H.T.R.

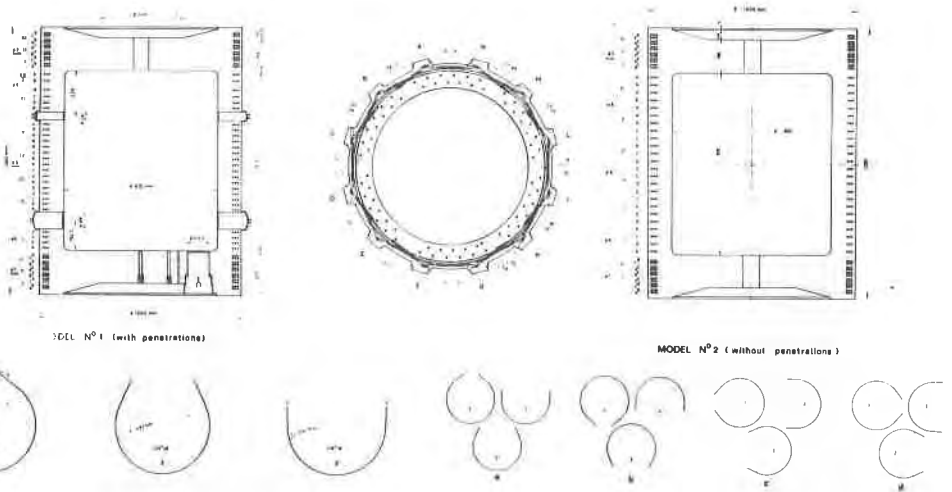
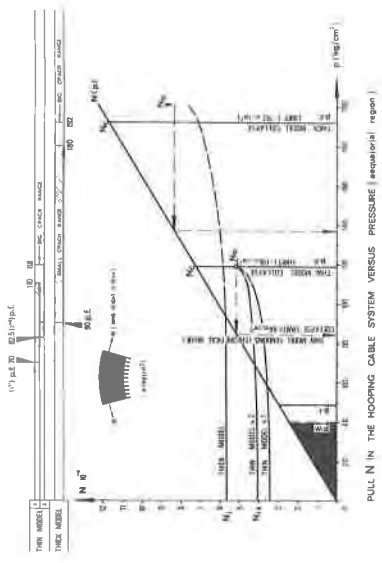


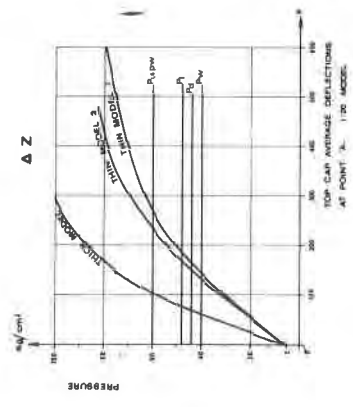
FIG. 3 PC.PV. MODELS N°1 AND N°2 (THIN SOLUTIONS)

PC.P.V. FOR H.I.R.



PULL N IN THE HOOPING CABLE SYSTEM VERSUS PRESSURE (equatorial region)

PC.P.V. FOR H.I.R.



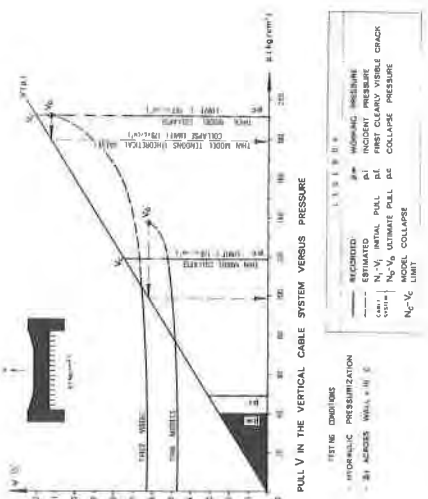
EQUATORIAL AVERAGE DEFLECTIONS AT POINT Z IN THICK MODEL

EQUATORIAL AVERAGE DEFLECTIONS IN P AT WORKING PRESSURE (kg/cm²)

DESCRIPTION	THICK MODEL	THICK MODEL 2	THICK MODEL 3
ΔH at P	422	2332	2260
Δ Z	820	2280	2190

- P₀ - working pressure 40 kg/cm²
- P₁ - design pressure 44 kg/cm²
- P₂ - initial internal pressure at 1100mm (radial)

PC.P.V. FOR H.I.R.



PULL V IN THE VERTICAL CABLE SYSTEM VERSUS PRESSURE

- TITRE ORIGINE
- RECORD
 - HOOPING BEARING
 - N₁-V₁ INITIAL PULL
 - N₂-V₂ INITIAL PULL AT FIRST GLASSY VISCO CRACK
 - N₃-V₃ ULTIMATE PULL AT COLLAPSE PRESSURE
 - N₄-V₄ MODEL COLLAPSE LIMIT

FIG 4 BEHAVIOUR OF TENDONS UP TO COLLAPSE

- P₀ - free liquid - static cyclic pressure
- THICK MODEL 1 - 70 kg/cm² (with parameters)
- THICK MODEL 2 - 62.5 - duration - 3
- THICK MODEL 30 - "

P₁ - testing equivalent static pressure

(see also kg/cm²)

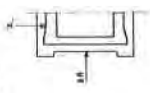


FIG.5 Deflection comparison between the three 1:20 scale models

P.C.P.V. FOR H.T.R.

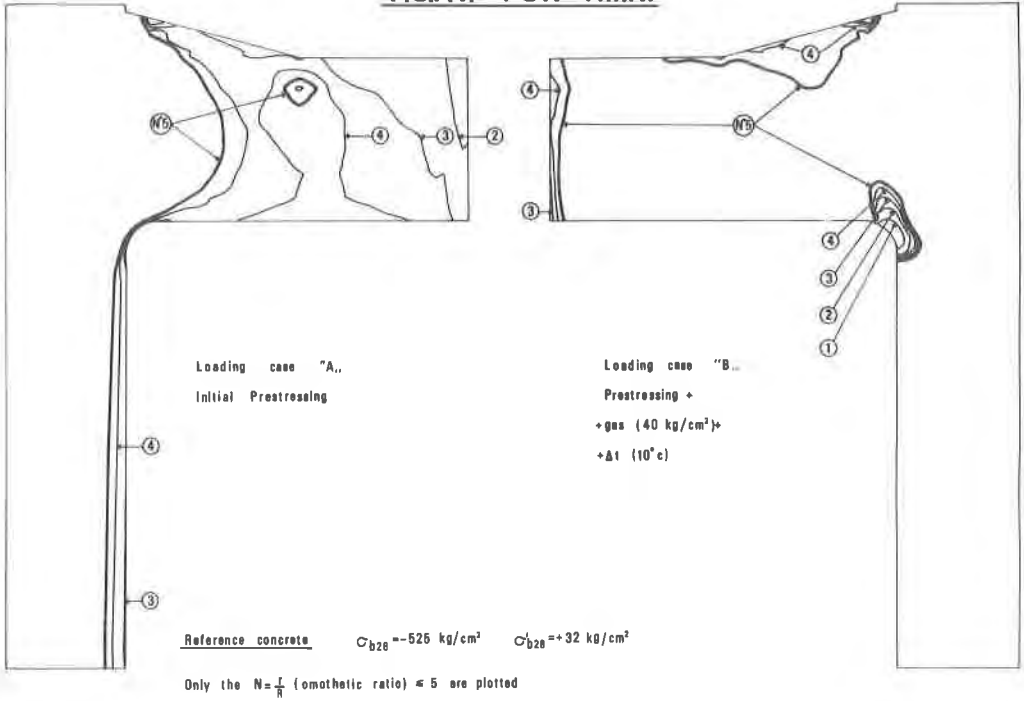


FIG 6 TYPICAL THIN MODEL MOHR - COULOMB SAFETY FACTORS N CONTOURS OUTPUTS