Design and Construction of Nuclear Power Plants

Papers presented to the Seventh FIP Congress, New York, 26 May-1 June

During the Seventh FIP Congress held in New York in May 1974 a Technical Session was organised by Mr J.A. Derrington (UK) at which selected international authors reviewed recent developments in their countries in the light of future requirements. This volume contains the papers presented together with a brief summary of the introduction to the Session and a summary of the papers. FIP is very grateful to the organiser, authors and chairmen of the Session for this outstanding contribution to the state of art at the present time.

Ben C Gerwick, Jr.

President of FIP

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ISBN 0 7210 0991 3

Published by the Fédération Internationale de la Précontrainte,
Wexham Springs, Slough SL3 6PL

Designed and printed by the Cement and Concrete Association,
52 Grosvenor Gardens, London SW1W 0AQ

November 1974

Price £4

Design and Construction of Nuclear Power Plants

Papers presented at the Seventh Congress of the Fédération Internationale de la Précontrainte New York, 26 May - 1 June 1974

Chairman of Technical Session on Nuclear Power Plants
J.A. Derrington

Editor
Paul V. Maxwell-Cook

FEDERATION INTERNATIONALE DE LA PRECONTRAINTE

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Summary of the Nuclear Power Plant Seminar Papers

J.A. Derrington (UK), (Chairman)

The seminar comprised nine papers covering the present state of the art in Europe, Japan and North America and included contributions describing present practice in design and construction, the results of recent research projects, progress in codification in North America, Germany and UK, and future developments in the construction of floating plants.

An appropriate prelude was the lunchtime speech by W.O. Doub of the US Atomic Energy Commission in which he stated the present programme in the US for expanding electricity production from nuclear power plants from 6% of the total in 1974 from 44 units to approximately 44% of the total by the year 2000 from 1000 units. On the same day it was announced that EEC plans were to increase nuclear power generation from the current 1.69% of the total to 17% by 1985.

Mr. Doub stated that nuclear power generation had shown production costs about 80% of that from fossil fuels and spoke of the need for increased standardisation in design to reduce the present lead time for nuclear power plants in US from 10 years to the 5- to 6-year period found elsewhere. These points also featured in the seminar discussions, in all of which it was agreed that prestressed concrete had been chosen as the ideal structural material for both containment in Light Water Reactors and for the pressure vessels for Gas Cooled and Heavy Water Reactors. Prestressed concrete was recognised universally to possess advantages over steel for reasons of economy, reliability and, of increasing importance, safety.

Eriksson (Sweden) in his paper, described the programme for Scandinavia with one plant operating in Sweden, 10 under construction in Sweden and Finland and plans for several further plants to be operational in the 1980s in all four countries. Plants were all BWR or PWR reactors (a majority of them the former), and development was proceeding on plans for a fast reactor. All had prestressed concrete containments for reasons of safety and economy. Tendons were grouted after stressing, and the containment walls were generally slipformed with the steel linear 25mm thick embedded in the concrete. Extensive leakage tests were reported with results all well below specified levels and details of the future construction programme for Scandinavia.

The paper by Bordet and Costaz (France) described the use of prestressed concrete for the construction in France both of pressure vessels for gas cooled reactors and for containment structures for light water reactors of which 12 were already under construction (of 900 MW capacity). Containments were designed for pressures up to 4 atmospheres (60 psi) and steel liners were used. General practice was to grout all prestressing tendons. To provide greater safety against missile damage, containment for a 1200 MW PWR was now being designed as a double-skin structure with the outer shell of reinforced concrete without a metallic lining, the cavity being maintained under negative pressure to control possible vapour leakage. The leakage would be restricted to a figure of 1% per 24 hours. Further developments under consideration involve the use of prestressed concrete pressure vessels for PWRs.

Burrow (UK) outlined the present programme for the construction of 14 gas cooled reactor pressure vessels all of prestressed concrete of which two had been in service for six years. Large tendons with load capacities over 1000 tons had been used and one type using helical stressing for the cylindrical walls gave considerable economy in steel. Another pattern which used wire wrapping for stressing pod-type vessels used 33 layers of 5 mm wire. Cables generally had not been grouted and results of the long term monitoring programme on the tendons showed no significant loss of stress and no deterioration caused by radiation. The future programme in UK for HTRs, PWRs and SGHWRs would rely on prestressed concrete for both containment and pressure vessels and it was planned to increase installed generating capacity from nuclear plants from the current figure of 65 000 MW to 135 000 MW for 1985.

British Standard CP 4975, which was based on limit state principles, has been in use for some years and consideration was now being given to increasing allowable stresses for concrete under a triaxial stress state and to the philosophy of dealing with possible accidental loading conditions.

Bremer (FDR) gave details of a German prestressed concrete pressure vessel for a 300 MW helium cooled reactor which was now under construction. This was designed for a pressure of 40 atmospheres (600 psi) with gas temperatures of 250°C to 770°C High strength concrete (90-day strength 70 N/mm^2) was used and special measures to limit shrinkage cracking were required. Prestressing both vertical and tangential was obtained with large tendons of over 900 tonne capacity using BBRV cables containing 151 wires. Development work was also proceeeding on gas cooled HTRs and LWRs in all of which safety considerations dictated the use of prestressed concrete.

The paper by Perry and Burdette (USA) described the construction of the Bellefonte Nuclear Plant for TVA which comprises two 1300 MW PWRs. The containment vessels for these are approximately 130 ft diameter, 270 ft high, designed for a pressure of 50 psi and represent the latest development in US practice in that they use prestressing tendons with an ultimate capacity of over 500 tons, use only four buttresses for the hoop prestressing and prestressed grouted rock anchors to resist uplift forces instead of a thick concrete raft foundation. As a result of the safety guidelines established by the US Atomic Energy Commission, prestressed concrete containments with thin steel liners (¼ in. to ½ in.) have evolved as the best method of containing postulated accidental release of high energy radioactive steam from the reactor coolant system. They state the need for consistent design criteria for containment vessels particularly to cover tangential shear impact effect of tornado-borne missiles, earthquake and thermal loading effects on liner and concrete.

The paper by Northup (USA) discussed the proposed Standard prepared by a joint ACI/ASME Technical Committee (359) on Concrete Pressure Components for Nuclear Service, which constitutes the requirements for the design, construction and use of concrete reactor vessels and concrete containment structures for nuclear power plants. The Standard was approved for publication in April 1973 and is now in use on a trial-for-use and comment basis for a period of about one year. It includes a section on the requirements for a quality assurance programme, the performance of inspection and qualifications of inspection personnel.

Two papers on research and testing were presented, one by Goffin (FDR) and another by Takemoto and others (Japan). The German paper stated that with a planned nuclear power production reaching 40 000 to 50 000 MW by 1985 several basic research programmes were necessary to develop advanced reactor types. These were designed to produce prestressed concrete reactor vessels for LWRs and HTRs and covered some 25 research projects. These included:

(1) Studies of the strength deformation and fracture properties under multi-axial load transfer, especially in the high temperature range (20°–150°C).

(2) The influence of reinforcement on multi-axial strength of concrete.

(3) The influence of changes of stress and temperature on compressive strength.

(4) Studies of the increase in tensile strength and elasticity of concrete by steel fibre reinforcement.

(5) The influence of moisture content and maturity on thermal conductivity of concrete.

(6) Creep properties of concrete under multi-axial load.

(7) Influence of irradiation on the material properties of concrete.

(8) Determination of the thermal expansion of concrete.

(9) A study of the protection of prestressing steel against corrosion by galvanising.

(10) Theoretical and parametric studies of calculation methods for reactor vessels.

(11) Development of vessel instrumentation.

(12) Studies of three types of liners.

For the next four years a research budget of over DM4 million has been planned.

The Japanese paper gave details of the tests on two 1/20 scale models of a new type of prestressed concrete reactor vessel for a multi-cavity high temperature gas-cooled reactor. The research programme now under way in the Ohbayashi-gunic Limited Technical Research Institute includes pressure tests of models, vibration tests of support structures and developments in methods of analysis, The results of tests and comparisons between calculated and measured values are reported in this paper.

The fianl paper by Orr and Bennett (USA) described the facility now under construction in Jacksonville, Florida, to manufacture floating nuclear power plants. This comprises a totally integrated nuclear station – a conventional PWR of 1150 MW – mounted on a floating platform. The pressurised water reactor, nuclear steam supply system, turbine generator, auxiliary systems and safeguard systems are based on proven land based technology and concrete structures are provided in those areas where biological shielding is necessary such as containment structures and auxiliary areas housing potentially radioactive equipment. The entire structure is supported upon a steel raft 400 ft long, 378 ft wide and 44 ft deep with a total displacement of about 150 00 t. The use of steel for the foundation raft caused some surprise in the discussion, and prestressed concrete was recommended for reasons of economy and availability.

The discussion period covered the whole of the nine papers presented and showed that prestressed concrete was the preferred material for construction due to reasons of economy, reliability and safety. The plans for development of nucelar energy showed generally an increase in the size of containment structures, increased pressures and temperatures for pressure vessels and as a result increases in the size of individual tendons up to and beyond the 1000 t figure. The trend to standardising designs was marked and discussion also ranged over the development of design codes and standards covering not only the material and structural performance but also the philosophy of safety measures. In all the discussions the work of FIP in providing an international forum for discussion was appreciated and particularly the need for the new FIP Commission on Pressure Vessels.

Introduction

M.J. Holley, Jr. (USA), (Co-Chairman)

One observes a growing perception that there are, indeed, limits to growth – in human members and in the scale of certain human activities which can be supported by the space and resources our planet affords us. As an example relevant to this seminar, it can be persuasively argued that an absolute limit on man's rate of energy production is associated with its effects on the natural climate. This upper limit is, of course, a substantial multiple of the current rate of energy production. However, even if one makes conservative allowances for our present imprecise knowledge of this limit, current growth rates would, if sustained, bring us to it in the course of shockingly few additional human generations.

In the light of these recently perceived limits, their rude conflict with an almost universally accepted growth ethic, and the lengthy time required to implement any substantial change in direction, there is an urgent need for society to raise its planning horizons by a few decades. One would hope for long range energy planning reflecting not only present demands but the desirability of a smooth transition, through future reduced growth rates toward some safe target limit. At the present time our global society is without any such plans, nor can we reasonably hope for their development and general acceptance for another decade or two. This has implications for all who are engaged in the engineering of energy production facilities, particularly nuclear power plants, over the interim period.

In the absence of comprehensive planning for future growth, indeed in the light of our incomplete understanding of the relationships of energy availability to world-wide economics, there may be cause to fear any unplanned, substantial slackening in growth. Any *abrupt* reduction in growth in energy production may have even more serious economic consequences. In the area of nuclear power these two concerns underlie much of the technological effort which our speakers described in their papers.

Costs of nuclear power plants (particularly those costs associated with lengthened construction times) continue to rise. These growing costs threaten an unplanned turndown in growth, deriving from limits to capital resources. As nuclear power assumes a greater share of total energy production, any interruption of the growth in nuclear plant facilities may have increasingly severe economic consequences. Thus, quite apart from the direct, potentially injurious, effects of a serious nuclear accident, one must recognise potentially serious secondary economic effects. Indeed, it is not necessary to postulate an accident; discovery of a sufficiently serious, generic, flaw, could interrupt the service lives and new constructions of a large subset of nuclear power plants. The safety record, to date, is excellent, and it must continue so into the indefinite future.

Reliability, improved construction and operating economies, and, *always*, safety, are the motives underlying all of the technological progress discussed in this seminar. The validity of these motivations cannot be over-emphasised.

The use of prestressed concrete in nuclear plant construction in Scandinavia

Kurt Eriksson (Sweden)

GENERATION OF NUCLEAR POWER

The generation of electric power in Scandinavia is shared between the state and private and municipal utilities. In Sweden the electric power generating capacity is divided as follows:

The State Power Board	45%
Private utilities	42%
Municipal power companies	13%

In Finland and Norway the picture is very much the same while in Denmark the power generation is highly decentralized (no state agency exists). The different utilities (public and private) collaborate in the operation of the national grids (in Sweden through the Agency CDL), which are also interconnected at different points. CDL is also responsible for the forecasting of power consumption and generation.

Consumption of electric power is high in the Scandinavian countries (Figure 1), especially in Norway, where the cheap hydro power available has encouraged an extensive use of electricity for heating purposes and for the establishing of industries such as aluminium mills with a very high demand of electric energy.

Despite a considerable generation of electric power, the total energy balance for Sweden (Figure 2) shows a dependence on oil supply which is greater than for any other country. As the sources of hydro power are quite insufficient for the increasing demand of power, development of a programme for a large installation of nuclear power has become a necessity.

Power production in Sweden (1973) consists of:

Hydro power	72%
Thermal power	19%
Nuclear power	2%
Imports	7%

Figure 3 shows the recorded and anticipated increase of the consumption of electric power for different purposes in the period 1960-1990 and Figure 4 illustrates how the demand is going to be met. From today's 2% the nuclear power is expected to cover 50% of the power production in 1985.

Mainly for environmental reasons the general policy for Swedish licensing authorities has been to concentrate the expansion of nuclear power plants on the four sites, where construction has already started, namely Oskarshamn and Barsebäck for the private–municipal sector and Ringhals and Forsmark for the State Power Board. Later another two sites, Södermanland and Brodalen, will be opened for exploitation. The remaining site Haninge, is specially planned for the installation of a nuclear plant for combined generation of power and for district heating in the Stockholm area. All plants use sea water for condenser cooling.

Swedish commercial nuclear plants in operation or under construction are:

Plant	Reactor	In operation	Rated capacity MW_e
Oskarshamn 1	BWR Asea-Atom	1972	440
Oskarshamn 2	BWR Asea-Atom	1974	580
Ringhals 1	BWR Asea-Atom	1974	760
Ringhals 2	PWR Westinghouse	1974	820
Barsebäck 1	BWR Asea-Atom	1975	580
Barsebäck 2	BWR Asea-Atom	1977	580
Ringhals 3	PWR Westinghouse	1977	900
Forsmark 1	BWR Asea-Atom	1978	900
Ringhals 4	PWR Westinghouse	1979	900
Forsmark 2	BWR Asea-Atom	1980	900

The further construction schedule planned according to Figure 5 indicates an installed capacity of 16 000 MW by 1985 and 24 000 MW by 1990. This is a very ambitious programme as is evident from Figure 6, in which the installed capacity per capita in 1985 is represented, showing Sweden with the top figure.

In the other Scandinavian countries planning for nuclear power has not yet advanced as far as it has in Sweden. Finland is the most advanced in this respect, where three plants are under construction:

Plant	Reactor	In operation	Rated capacity MW_e
Lovisa 1	PWR V/O Techno-promexport	1976	440
Lovisa 2	PWR V/C Techno-promexport	1978	440
TVO Olkiluoto 1	BWR Asea-Atom	1978	660

Lovisa 1 and 2 are of the Novovoronesh type and ordered from USSR by the state utility Imatran Voima OY. The buildings, including the steel containment are supplied by the owner, as well as the ice condenser built on a Westinghouse licence. TVO is a private utility, owned by a group of industries.

In Denmark tender documents for the first nuclear power plant have been issued recently. Purchasing will probably be on a turnkey basis. In Norway the need for nuclear power is less than in the other countries, due to the presence of cheap hydro power. The first nuclear plant is planned to be in operation in 1982 or 1983.

All Swedish and Finnish nuclear power plants, except Lovisa 1 and 2, are provided with reactor containments in prestressed concrete.

BOILING WATER REACTORS (BWR)

All boiling water reactors in Scandinavia are of Asea-Atom design. Containment is based on the pressure suppression (PS) principle, i.e. at a major pipe rupture the steam is led from the upper part, dry-well, through a number of pipes to the lower part of the containment, wet-well, with a water pool, where condensation takes place. Thanks to this principle the volume of the containment can be kept small, about one fifth of that of a dry containment with the same design pressure. (1,2)

The layout of the containment varies from plant to plant as can be seen in Figures 7-10. The typical bottle-shaped shell structure for Oskarshamn 2 (Figure 7, is the same design applied for Barsebäck 1 and 2) is due to the location of the main circulation pumps in an annular space outside the reactor vessel and its biological shield. The fuel element pools, together with the pool for the storage of reactor internals, rest on the containment proper, which is statically independent from the pool structure and from the surrounding outer reactor building which houses the reactor auxiliary systems. For Ringhals 1 (Figures 8 and 9) the containment vessel has been built more integrally with the fuel element pools. In later designs (Figure 10 is in principle applicable not only to TVO Olkiluoto 1, but also to Forsmark 1 and 2) the external circulation pumps have been replaced by internal recirculation pumps, where the motor housing forms an integral part of the reactor vessel. Consequently, the layout of the containment can be simplified to a cylinder with a solid partition slab between the dry-well and wet-well compartments.(3)

The PS containments are all prestressed horizontally and vertically, the latter for internal gas pressure minus the overlying weight. The design pressures are between 460 and 550 kN/m^2 (absolute pressure). In most cases the fuel element pools, which are partly cantilevered, are provided with prestressing tendons in the side walls. The top of the containment over the reactor vessel has to be removable for the refuelling operation and is designed as a steel dome.

A further development of the layout of the containment and adjoining structural units is seen in Figure 11, showing the Asea-Atom current design (temporarily denoted as BWR 75). Consideration has also been taken to earthquake and airplane crash loading. The structural connection between the fuel pools and the containment proper has been further simplified.

All BWR plants, except Barsebäck 1 and 2, are founded on rock. In Barsebäck 1 and 2 the bottom of the containment consists of a thick slab on hard moraine-clay.

PRESSURIZED WATER REACTORS (PWR)

PWR nuclear plants, Ringhals 2, 3 and 4 (Figure 12) are all standard Westinghouse design, except for the arrangement of the liner (see later). They are provided with a dry containment in the shape of a cylinder with a dome roof, both in prestressed concrete. The design pressure is 510 kN/m^2 (absolute).

DESIGN DETAILS

A special feature in the design of the Scandinavian reactor containments is that the steel liner is usually embedded in the concrete. The internal concrete cover, approximately 250 mm, serves as an efficient protection for the liner against missiles during an accident while at the same time the liner is protected against corrosion. Furthermore, thermal expansion of the steel liner due to the elevated temperatures (155°C-180°C) during the initial phase of an accident is avoided, resulting in a saving of prestressing steel. The disadvantage of the embedment is that if leakage occurs during a test it may be difficult to locate and repair. For that reason the liner in some of the State Power Board plants has been provided with steel channels along all welds. The channels are connected in sections and with the outside, whereby leaking gas, if any, can be located and vented.

A structural detail which has been given much attention is the corner between the cylindrical wall and the bottom slab. Different solutions are shown in Figure 13, a sliding joint for Ringhals 2, an incomplete hinge for Barsebäck 1 (acting as a sliding joint during the prestressing operation) and built-in systems for Oskarshamn 2 and TVO Olkiluoto 1. The justification for a mobile joint with its more complicated system of bearings and liner connections is that stress concentrations are avoided and that the absence of the haunch facilitates the slipform casting with a certain saving in construction time. As the main purpose of the containment is to provide a leakproof seal it is, however, evident that the less sophisticated fixed-end design constitutes a safer solution, which is more independent of imperfections in the execution.

DESIGN CRITERIA

The containment is designed for a sudden rupture of one (or in some cases two) of the main pipes. Diagrams of transient pressures and temperatures in different parts of the containment are shown in Figure 14 for two conditions at Forsmark 1. The design pressure in this case is 550 kN/m^2 (absolute). It is conservatively assumed that 5% of the cladding metal in the fuel elements reacts with water causing generation of hydrogen gas. Maximum design temperature is 180°C.

Local reactions and missile forces and jet impingement during design basis accident are specified by the supplier of the nuclear systems. For Forsmark 1 the maximum value of such forces amounts to 4100 kN including an impact factor of 2.

The partition slab between dry-well and wet-well is designed for a differential pressure of 24 kN/m^2.

Special codes for the design of buildings for nuclear power plants do not exist in Scandinavia but the design philosophy, applied for prestressed reactor containments in Sweden and Finland are:

1. The prestressing force shall be sufficient to ensure that no resultant tensile force (membrane tension) occurs in the pressure-retaining structural part for design basis accident pressure.

2. For a loading condition corresponding to a pressure 1.5 x design pressure, the stress in the reinforcement bars and the prestressing steel shall not exceed 0.9 x the yield strength (or 0.2% proof stress). Cracks would be expected to develop but shall close after the pressure has been relieved. A limited local leakage of gas due to stress concentrations, primarily at penetrations, may occur and is acceptable.

3. For a pressure equal to 2 x design pressure large irreversible cracks and extensive leakage are allowed but collapse shall not occur.

Conditions 1 and 2 are analysed as serviceability limit states and condition 3 as an ultimate limit state. The prestressing is in general designed for condition 1. Vertical prestressing is, however, in some cases increased to cover the lifting forces corresponding to condition 3. Design details and construction sequence are planned so that tensile flexural stresses are kept small in the liner for design basis accident. Reinforcement bars are used to cover tensile stresses due to moments, temperature effects, local forces and requirements according to conditions 2 and 3. The amount of reinforcement bars necessary for these purposes is considerable, e.g. 90 kg/m^3 of concrete in the cylindrical wall for TVO Olkiluoto 1.

For the design of the prestressing, allowable stresses, and such like Swedish and Finnish general Codes for prestressing works have been applied. For Forsmark 1 the authorities in Sweden have approved the use of the ACI Committee 349 Criteria for Reinforced Concrete Nuclear Power Containment Structures (1972) together with a check that condition 3 above is fulfilled. For the containment of the Asea-Atom current design, BWR 75, the ACI-ASME Proposed Standard Code for Concrete Reactor Vessels and Containment has been applied.

CALCULATIONS

In the first phase, calculations for the containment proper are carried out assuming axi-symmetry. Several computer programes exist for this case. In the second stage, consideration is taken to large penetrations by applying a general computer programe for shells (EASE, STRIP or STARDYNE) on a portion of the structure. The same kind of programme is also used in later BWR designs for the analysis of the integral system consisting of the fuel element pools and the upper part of the containment.

Figure 15 shows the results of the analysis of the main loading conditions for TVO Olkiluoto 1.

PRESTRESSING WORKS

In the Table below the prestressing systems are indicated for different plants and reference is made to the Figures showing these arrangements of the prestressing tendons.

Plant	System	Figures
Oskarshamn 1	BBRV	
Oskarshamn 2	BBRV	7, 13
Ringhals 1	BBRV	9
Ringhals 2	BBRV	12, 13
Barsebäck 1	VSL	13
Barsebäck 2	VSL	13
Forsmark 1	VSL	17
TVO Olkiluoto 1	BSL	10, 13, 16

All tendons are standard units in ducts of steel sheeting or in slipform-made vertical channels. For BWR plants the largest BBRV tendons consist of 55 wires ϕ 6 mm with a breaking strength of 2670 kN. For Ringhals 2 (PWR) BBRV tendons with 139 wires ϕ 6 mm have been used. The breaking strength is 7000 kN. VSL tendons which have been used in BWR plants are 12 Dyform and 19 or 31 Supa strands ½ in. with a breaking strength between 2500 kN and 5700 kN.

In the tender documents for buildings in a nuclear power plant no particular prestressing system is specified. Only the magnitude and the distribution of the prestressing force are given and the contractor is free to choose any system that can be approved by the owner. In Sweden the BBRV, VSL and Dywidag systems are the predominent systems for prestressing works in general as they have the most efficient representation in the country.

In most cases horizontal cables are anchored in four buttresses so that each tendon encloses 180°. In later Asea-Atom designs only two anchorage buttresses are provided and the cables enclose more than 360°. Despite the long overlapping and the friction losses this arrangement has proved advantageous because of the saving of anchors and space for the anchorage assemblies. Vertical shafts have been provided at each of the four anchorage zones so that the bundle of wires or strands can be handily drawn through the ducts which have previously been cast in and the tensioning operation can be carried out independently of the works on the surrounding reactor building. The arrangement also fits very well to the general layout of piping and electric cable trays.

A major problem for the arrangement of the prestressing tendons is the great number of penetrations, some of them very large. It is desirable to concentrate the penetrations in horizontal and vertical strips as much as layout considerations for the installations permit. Figures 16 and 17 illustrate the problem. They represent two containments with the same dimensions but with different design pressures (460 kN/m^2 for TVO Olkiluoto 1 and 550 kN/m^2 for Forsmark 1) due to the difference in installed power (660 MW and 900 MW, respectively). It is evident that the higher density of prestressing force at Forsmark 1 has led to a much more complicated tracing of the cables.

The prestressing tendons at Oskarshamn 1 and 2, Barsebäck 1 and 2, Ringhals 1 and TVO Olkiluoto 1 have been or will be grouted with cement. In later plants which are now being built for the State Power Board, Ringhals 2, 3 and 4 and Forsmark 1 and 2, unbonded cables will be used according to American practice and the ducts will be filled with grease. The authorities approve both methods and different opinions exist among designers as to the suitability of one system as compared with the other. The main reason for unbonded cables is the possibility of testing the prestressing force and the advantage of being able to replace the cables, if necessary, during the lifetime of the plant. Some improvement of friction properties may also follow. On the other hand the unbonded cable is entirely dependent on the transmission of forces at the anchorage points. If a crack occurs under some loading condition, the unbonded cable is essentially inefficient in participating with the reinforcing bars in the transfer of the load through the crack. Furthermore it does not contribute to the formation of distributed smaller and more harmless cracks at increased load instead of concentrated large cracks. Consequently, more reinforcement bars or additional prestressing are needed for equal safety against leakage. The thermal expansion of the grease and the long ends of strands sticking out from the anchorage faces create some layout problems. The dependence on the anchorages which are exposed, and the presence of combustible grease, although encased, increase the risk of damage due to fire. Finally, filling with grease is much more expensive than grouting.

Friction coefficients have been analyzed from the results of the tensioning operations. In the equation

$$F = F_0 \, e^{-\mu \cdot k.x}$$

the following values have been derived:

Plant	System	μ (measured)	k (assumed)
Oskarshamn 2	BBRV	0.15	0.02 μ
Ringhals 2	BBRV	0.08	0.0015
Barsebäck 1	VSL	0.18	0.02 μ
Barsebäck 2	VSL	0.19	0.02 μ

For the early plants, requirements for testing of materials and for execution of the prestressing works did not differ from specifications and practice for prestressed structures in general. Some negative experiences gained in early works and the common trend of increasing demand upon quality assurance have made it necessary to impose more extensive specifications with respect to testing and inspection.

During the prestressing works at Ringhals 1, failure of some of the anchor heads (BBRV 55 wires ϕ 6) occurred and subsequent testing of the anchorage details with respect to hardness gave unsatisfactory results. All anchor heads which were accessible were replaced and the pull sleeves were substituted by longer ones. At Oskarshamn 2, some couplings of vertical tendons failed by shear in the threads of the coupling sleeve (BBRV 55 Type VV 170 coupling), resulting mainly from the fact that the anchor heads were not completely screwed on. In some plants the positioning and alignment of the cables and the anchorage assemblies have not been entirely satisfactory. This was the case both at Ringhals 1, where the tendons were fixed to their supports before casting of the wall, and at Barsebäck 2, where the tendons (VSL 12 strands ϕ ½ in.) were installed successively during the slipform casting operation. In the latter case the deficiencies in geometry caused some wires in the strands to break due to wear against the bearing plates, and wedged plates had to be inserted to correct the alignment.

These mishaps do not undermine the confidence in the use of the common prestressing systems on the market in reactor containment structures, but it stresses the need for careful planning and inspection of the prestressing operations as well as for an extended testing programme. Although no final standards for concrete containments have yet been worked out in the Scandinavian countries, later specifications contain rather extensive requirements with respect to testing, fabrication, installation, protection against corrosion, etc., mainly conforming to the ACI-ASME proposed standard code. The increase in testing applies mostly to anchorage components and tendon assemblies. Stress relaxation tests performed by the suppliers at the elevated temperatures (40°C-50°C) which prevail are rather scarce and have to be supplemented. Proper marking for identification of prestressing material is essential to ensure traceability to test reports.

METHODS OF EXECUTION OF THE CONTAINMENT

The execution of the cylindrical containment wall varies depending on the method for the installation of the embedded liner. The method in Figure 18 is patented by the contractor, AB Armerad Betong, and has been used for Oskarshamn 1 and 2 and Forsmark 1. The whole wall was cast in one slipform operation, during which an annular slot, 120 mm, was created in the wall as well as channels for the vertical prestressing tendons. Horizontal tendons (at Forsmark 1 only the ducts) were fixed to special supports in the course of the casting. Boxing out for penetrations was also installed successively. The steel liner was then assembled on top of the wall in ring elements, 1.2 metres high. After testing of the welds the steel cylinder was successively lowered into the annular slot and finally welded to the bottom steel plate. After cutting of openings in the liner and fixing of the corresponding penetrations, the slot was grouted on both sides of the liner.

For Barsebäck 1 and 2 and Ringhals 2 the steel liner was first erected by using the same method as for an oil storage tank, i.e. sheet steel rings were welded at ground level and added to the already completed part of the cylinder, which was successively jacked up. The inner and outer parts of the wall were then cast in two separate slipform operations. The prestressing cables (Barsebäck 1) or the cable ducts (Ringhals 2) were mounted before casting, while at Barsebäck 2 the cables were installed during the casting. For TVO Olkiluoto 1 the same method will be used with the exception that the steel cylinder will be erected in a conventional way from ground level without jacking.

STRUCTURAL INTEGRITY AND LEAKAGE RATE TESTS

In most of the Swedish plants structural integrity tests have been carried out at design pressure. The leakage rate test was performed at different pressure levels, each for a duration of 24 hours. The reference test pressure has amounted to about 85% of the design pressure. For Ringhals 2 and for plants now under construction, the structural test pressure and the leakage rate test pressure have been raised to 1.15 x design pressure and to the design pressure, respectively, in conformity with American standard.

The maximum acceptable leakage rate at reference pressure for BWR plants with a PS containment is 0.5% of the weight of the contained air and for PWR plants with a dry containment 0.1%, both measured over a 24 hour period.

The Swedish containments, which have been tested for structural integrity, have all performed well. The leakage tests have yielded the following results:

Plant	Design pressure kN/m^2 abs.	Leakage test pressure kN/m^2 abs.	Acceptable leakage rate %	Measured leakage rate %
Oskarshamn 1	440	360	0.5	0.18
Ringhals 1	510	440	0.5	0.17
Oskarshamn 2	490	440	0.5	0.035
Barsebäck 1	490	440	0.5	0.051
Ringhals 2	510	510	0.1	0.08

Measurements of deformations have shown good agreement with calculated values.

SCANDINAVIAN PRESTRESSED CONCRETE REACTOR VESSEL

Since 1967, work on a joint Scandinavian project has been going on for the development of a prestressed concrete reactor vessel for light water reactors, in the first place BWRs. Participants are nuclear power research and development organizations, power utilities, supplier of nuclear systems (Asea-Atom), supplier of components (Finnatom), consultants and contractors. The Swedish research and development organization, A.B. Atomenergi, is responsible for project administration. The main incentives for introducing a PCRV as an alternative to a steel vessel have been:

1. *Technical.* With increasing outputs, technical difficulties arise with the fabrication (welding, etc.) of thick-walled large steel vessels. Studies have indicated that construction of a PCRV for a BWR of 2000 MW_e is feasible.

2. *Economy.* Comparisons for a 900 MW_e unit have reavealed slightly lower costs for the PCRV. For larger sizes the PCRV is gaining in economic comparison. The total construction time can be reduced to some extent with PCRV and construction can be planned without having the delivery of the steel vessel on the critical path.

3. *Operation.* De-pressurization and refuelling are more rapid due to the absence of thick pressure bearing steel parts at high temperature.

4. *Safety.* The inherent rendundancy of a PCRV and the more favourable mode of failure (the PCRV cannot be subject to explosive bursting for any pressure) makes the PCRV attractive the more the safety standards are tending to rise. This applies especially where urban siting is considered. Inspection programme is less difficult for PCRV.

Of the motives accounted for above, the last one, safety, is today by far the most important.

The main differences between a PCRV for LWR as compared to gas-cooled reactors, of which there are several in operation or under construction, are the higher operating pressure of a LWR, the smaller dimensions of the cavity, the necessity of a removable lid and the special problems with insulation and corrosion, due to the presence of water and steam.

A large scale (1:3.5) model was built at Studsvik research centre in Sweden in 1969. Testing of the model and detailed description of the design of the PCRV have appeared in several published papers.[4,5] The essential features of the design are illustrated in Figures 19 to 22.

The Scandinavian PCRV is designed for a pressure of 8500 kN/m^2 and a temperature of 300°C (operational conditions 7000 kN/m^2 and 285°C, respectively). The main feature that separates it from other concepts is the water lock (Figure 21) which keeps the annular space between the casing and the liner dry and hence also the insulation, which consists of a stainless steel foil mesh. In the bottom, however, the insulation is filled with stagnant water for which reason it has to be thicker. In addition to the insulation, two independent systems of cooling pipes assist in keeping the steel liner at the inner concrete face at a moderate temperature (50°C).

The removable lid is kept in position by a number of steel struts (Figure 20). Sealing is ensured by a toroidal ring and double O-rings with inter-seal suction for leak detection.

In the present design, external hoop prestressing with wire winding methods (Taylor-Woodrow or BBRV) or bands, prestressed by radial jacking (Coyne et Bellier), has been presented, but prestressing with ordinary large size tendons in ducts is also possible. The latter system has been used in the model (BBRV). For vertical prestressing, tendons in ducts are foreseen.

The risk of a pressure build-up in the thick concrete wall due to a major crack in the liner and leakage paths in the concrete is met by allowance in the structural design and by the introduction of liner venting channels in two directions.

The Scandinavian PCRV is adapted for the same design of core and internals as in a steel vessel and for a PS containment of the same design as for Forsmark 1 (Figure 22).

Work on the Scandinavian project has been carried out in three phases. In the first two a design was worked out and later modified and the model was constructed and tested structurally with water for short- and long-term, cold and hot conditions. Repeated pressurization to 1.5 x design pressure and one overpressure test to 2.5 x design pressure were carried out. In the latter case no leakage occurred and the structure still behaved in a 'steel-elastic' manner, i.e. the cracks closed after unloading. Tests have also been performed for different prestress of the steel struts and with removal of some of them. Insulation tests have been carried out with gas-filled or water-filled insulation. Transient conditions during accident simulation with sudden loss of insulation gas have been tested without damage on any item of the structure.

Parallel to the Studsvik tests, a programme of lid failure tests has started at Risø in Denmark(6) and vessel failure tests with gas pressurization of 4 smaller models have been performed by Trondheim Technical University in Norway. Failure pressures were about 3.2 x design pressure(7).

As the tests have been successful so far, a third phase of the project has been decided and recently started. The third phase incorporates a verification programme, including further failure tests in Denmark on lid design and on the bottom slab, further failure tests on small models in Norway, temperature shock simulation in vessel liner, testing of the liner venting system, corrosion and radiolysis tests on the insulation, all to be carried out at Studsvik. Vibration tests on insulation casing and pipe assemblies are planned to be performed in France. A trail manufacture of the bottom slab section with a number of penetrations is planned to verify that required tolerances can be met.

After completion of the verification programme at the end of 1975, the PCRV is planned to be offered on the market.

In the third phase of the development programme, some organizations outside of Scandinavia (France, Italy, UK) are associated.

A layout of a BWR plant with the Scandinavian PCRV has been studied for urban siting in Stockholm for a nuclear plant intended for combined power generation and district heating.

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(6) ANDERSEN, S.I. and OTTOSEN, N.S.
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(7) FLUGE, F., GAUSEL, E. and LENSCHOW, R.
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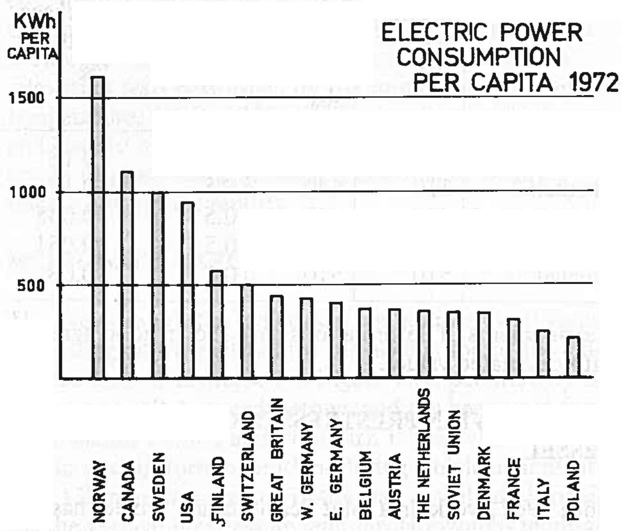
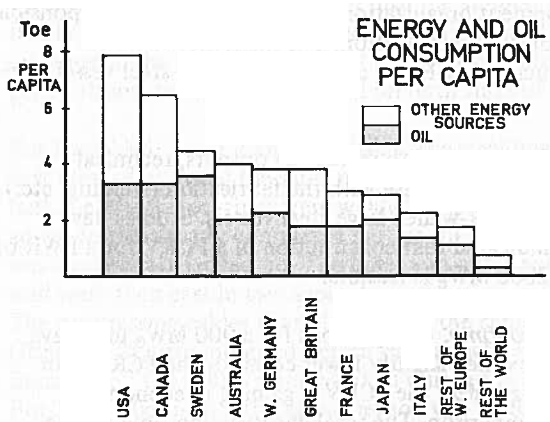


Figure 1.



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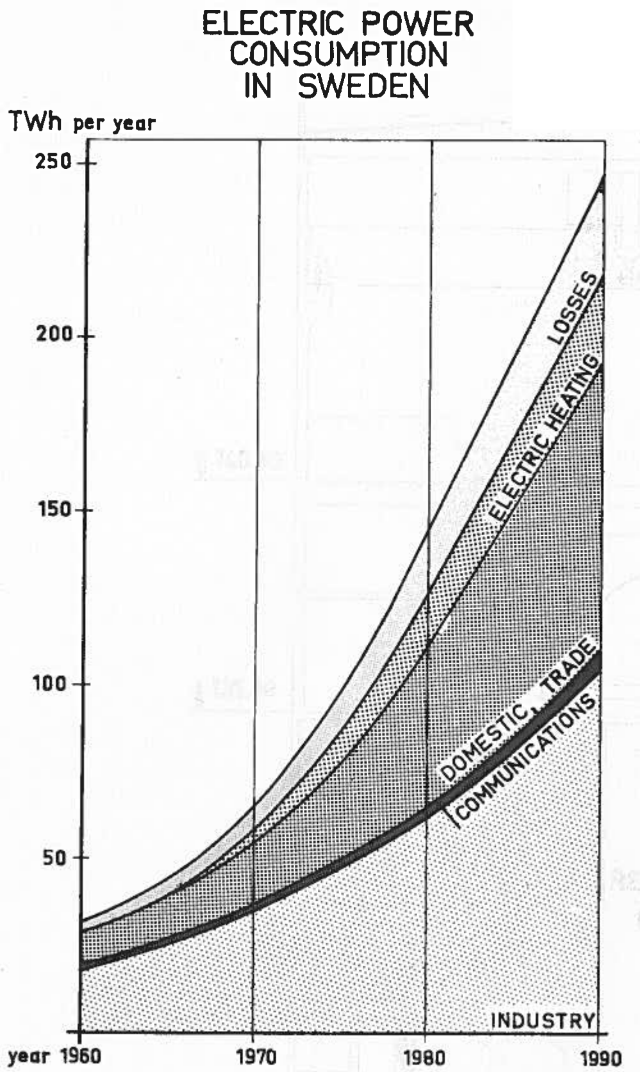


Figure 3.

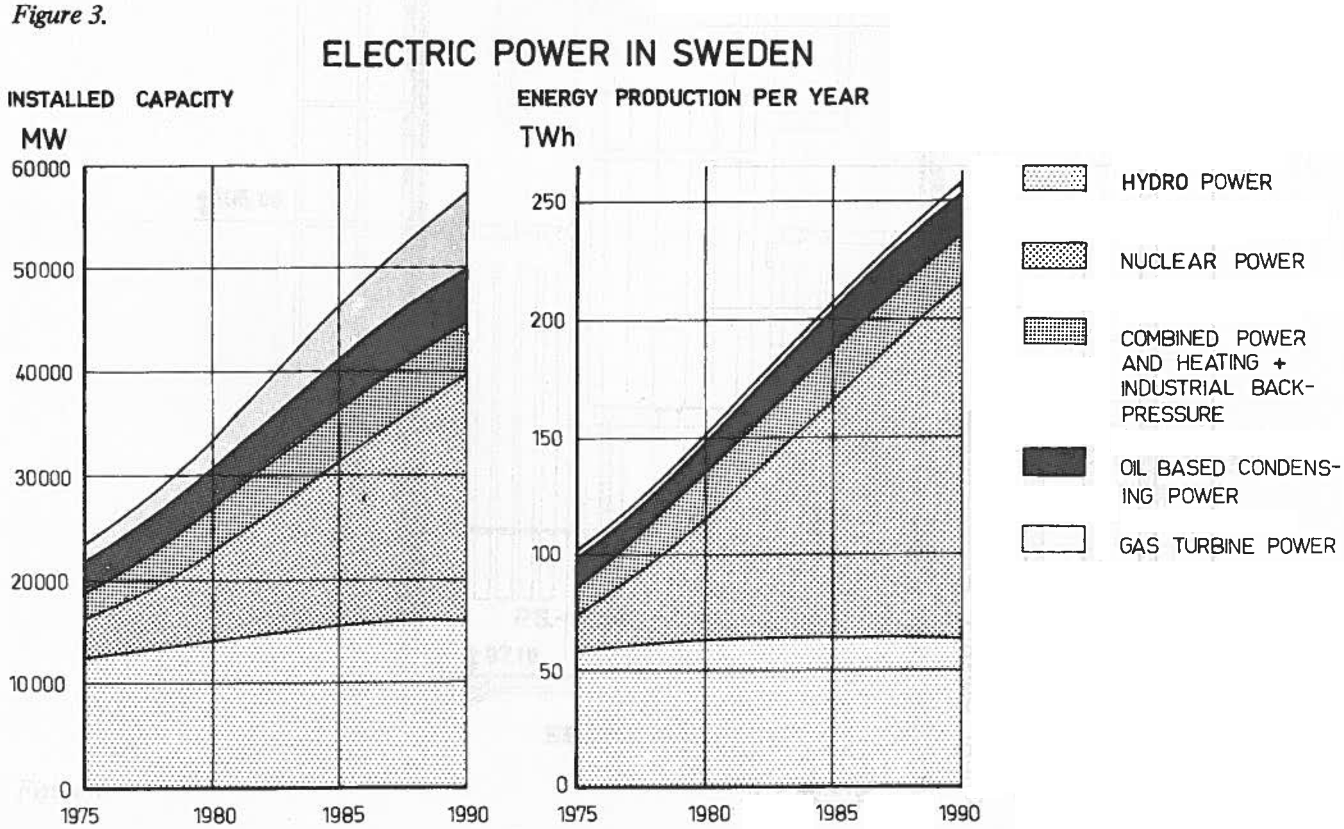


Figure 4.

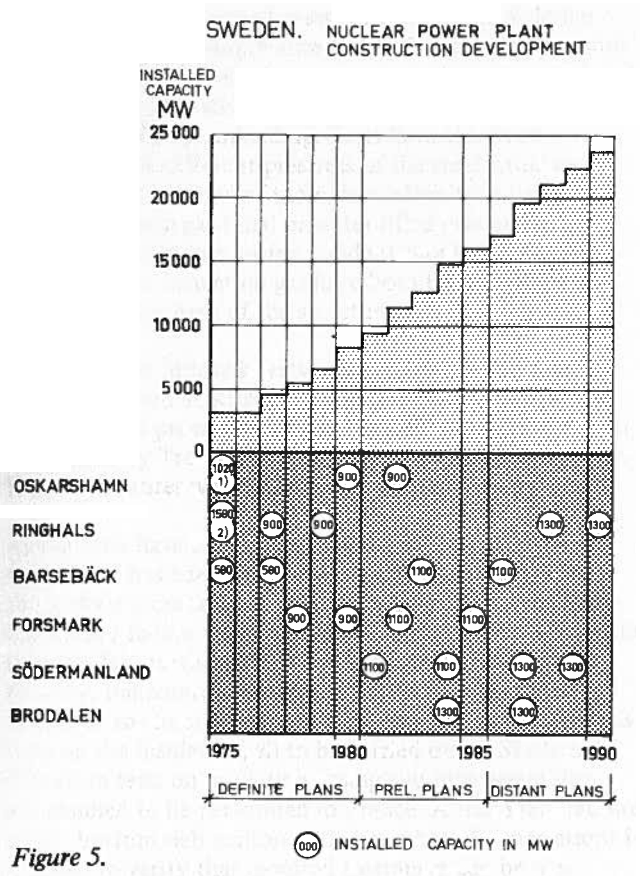


Figure 5.

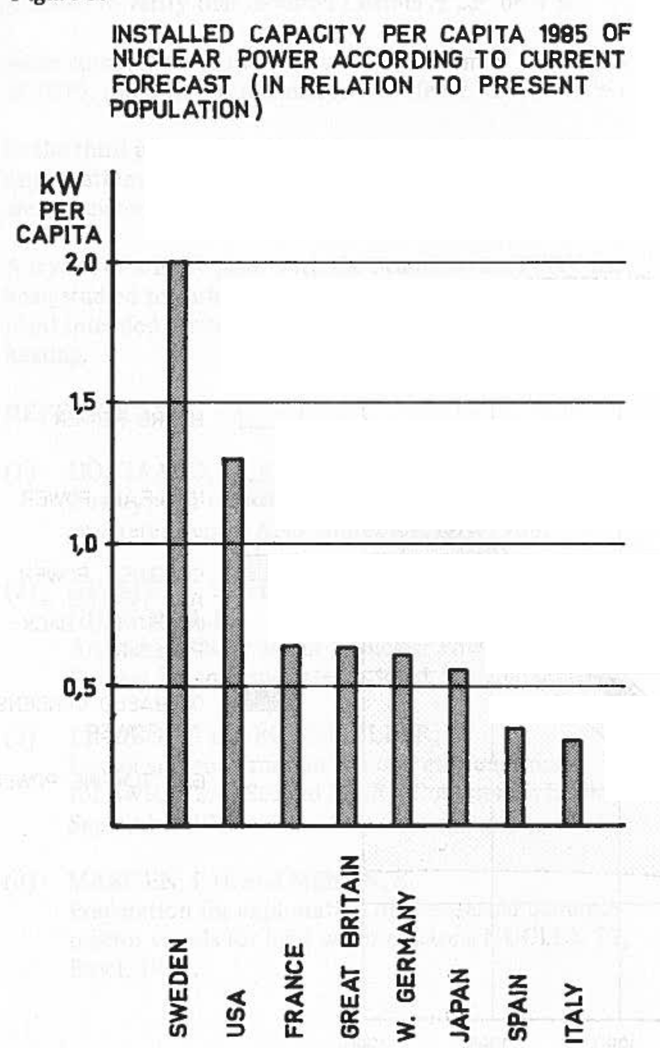


Figure 6.

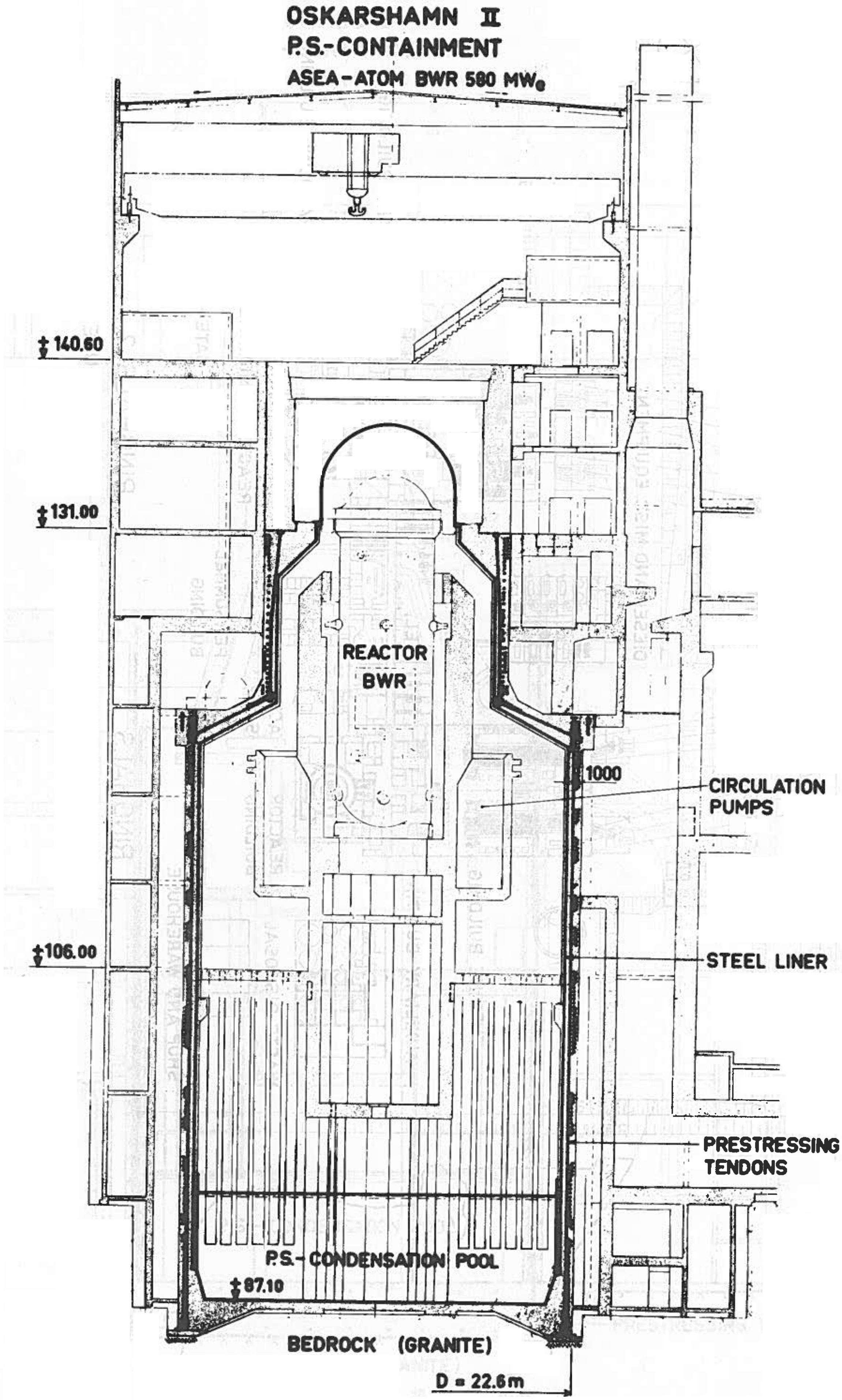

OSKARSHAMN II
P.S.-CONTAINMENT
ASEA-ATOM BWR 580 MWe
140.60
131.00
106.00
REACTOR
BWR
1000
CIRCULATION
PUMPS
STEEL LINER
PRESTRESSING
TENDONS
P.S.-CONDENSATION POOL
87.10
BEDROCK (GRANITE)
D = 22.6m

Figure 7.

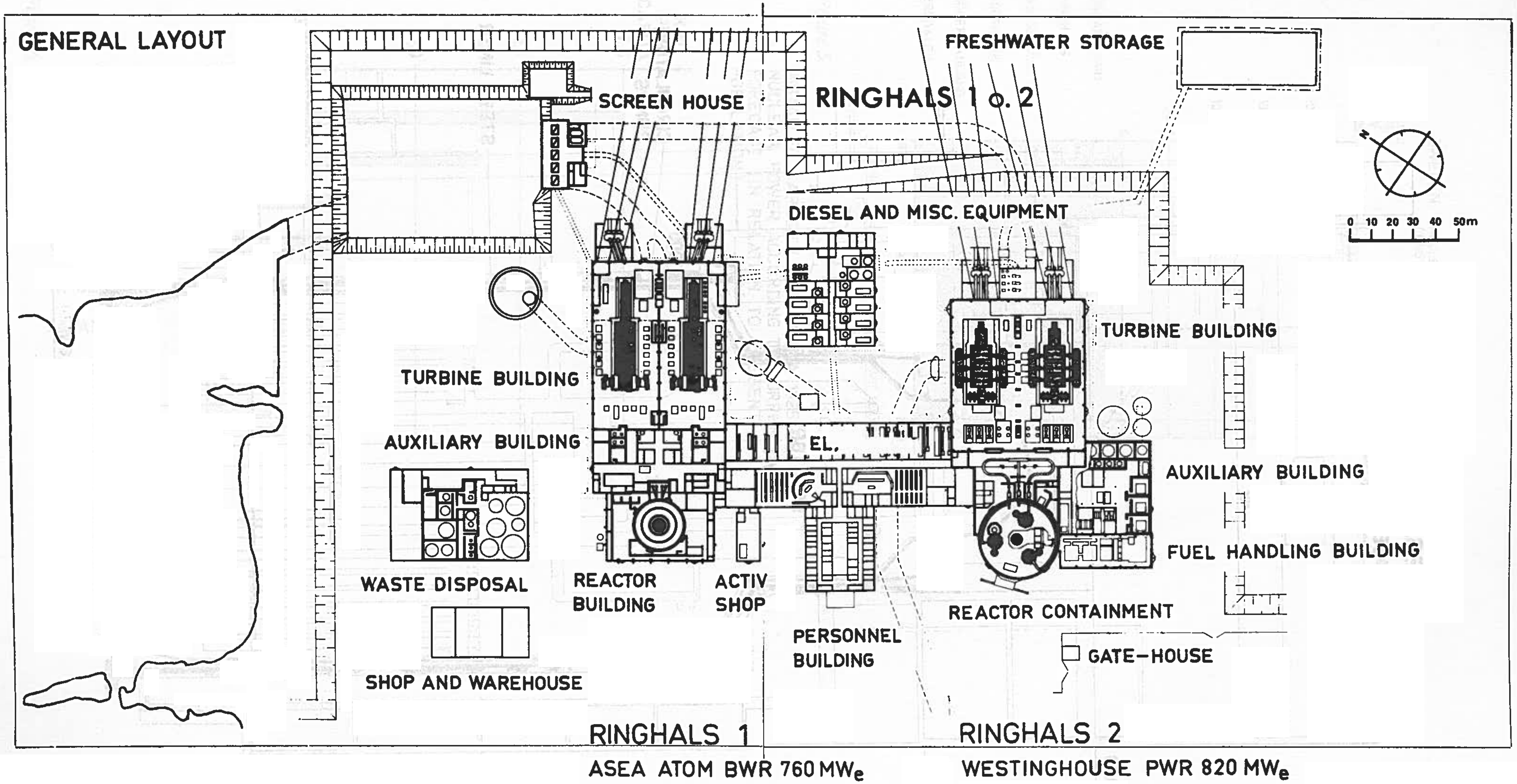

GENERAL LAYOUT
SCREEN HOUSE
RINGHALS 1 o. 2
FRESHWATER STORAGE
DIESEL AND MISC. EQUIPMENT
0 10 20 30 40 50m
TURBINE BUILDING
AUXILIARY BUILDING
WASTE DISPOSAL
SHOP AND WAREHOUSE
REACTOR BUILDING
ACTIV SHOP
EL.
PERSONNEL BUILDING
TURBINE BUILDING
AUXILIARY BUILDING
FUEL HANDLING BUILDING
REACTOR CONTAINMENT
GATE-HOUSE
RINGHALS 1
ASEA ATOM BWR 760 MWe
RINGHALS 2
WESTINGHOUSE PWR 820 MWe

Figure 8.

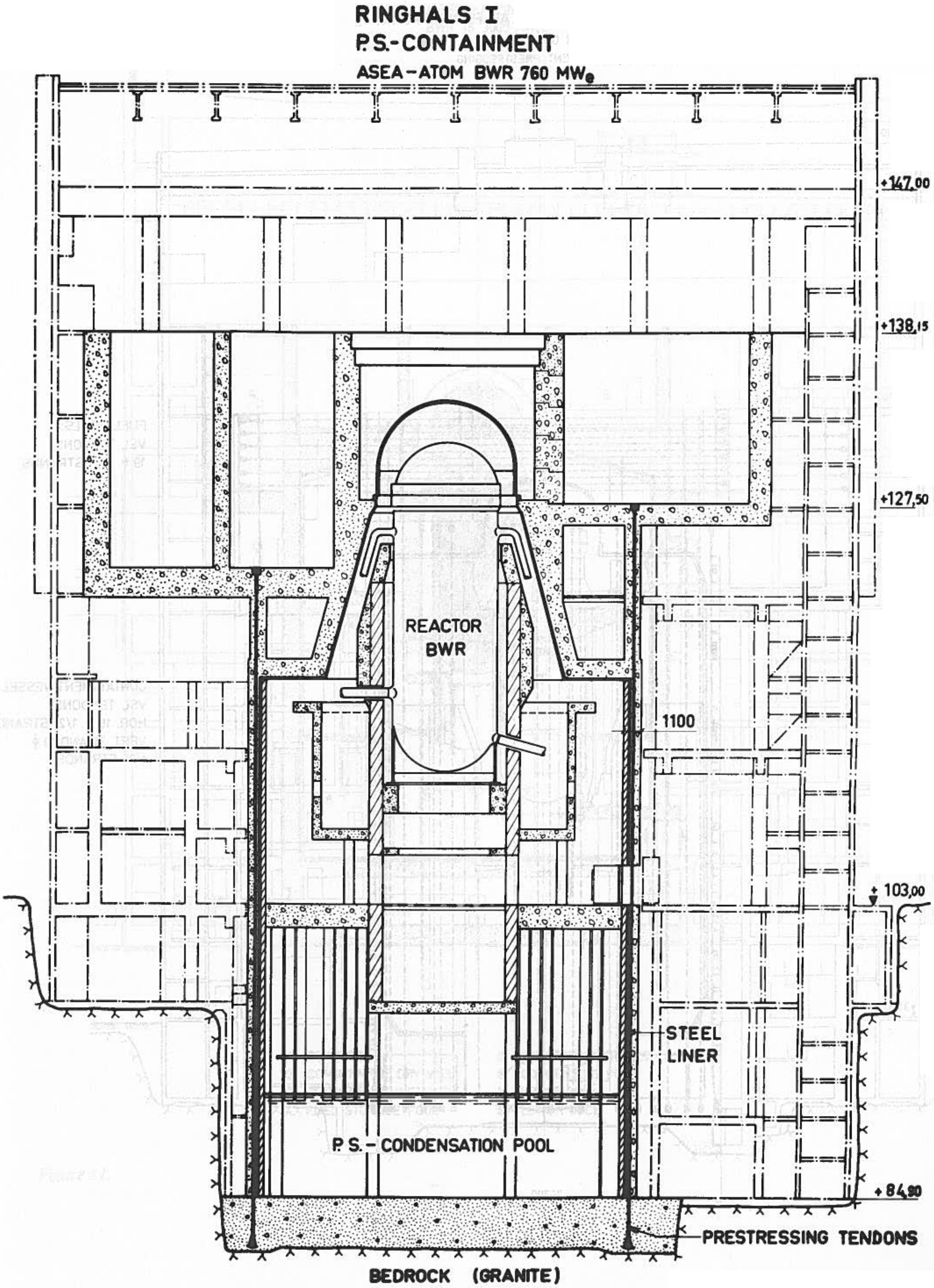

RINGHALS I
P.S.-CONTAINMENT
ASEA-ATOM BWR 760 MWe
+147,00
+138,15
+127,50
REACTOR
BWR
1100
+103,00
STEEL
LINER
P.S.-CONDENSATION POOL
+84,90
PRESTRESSING TENDONS
BEDROCK (GRANITE)
0
5
10
15
20m

Figure 9.

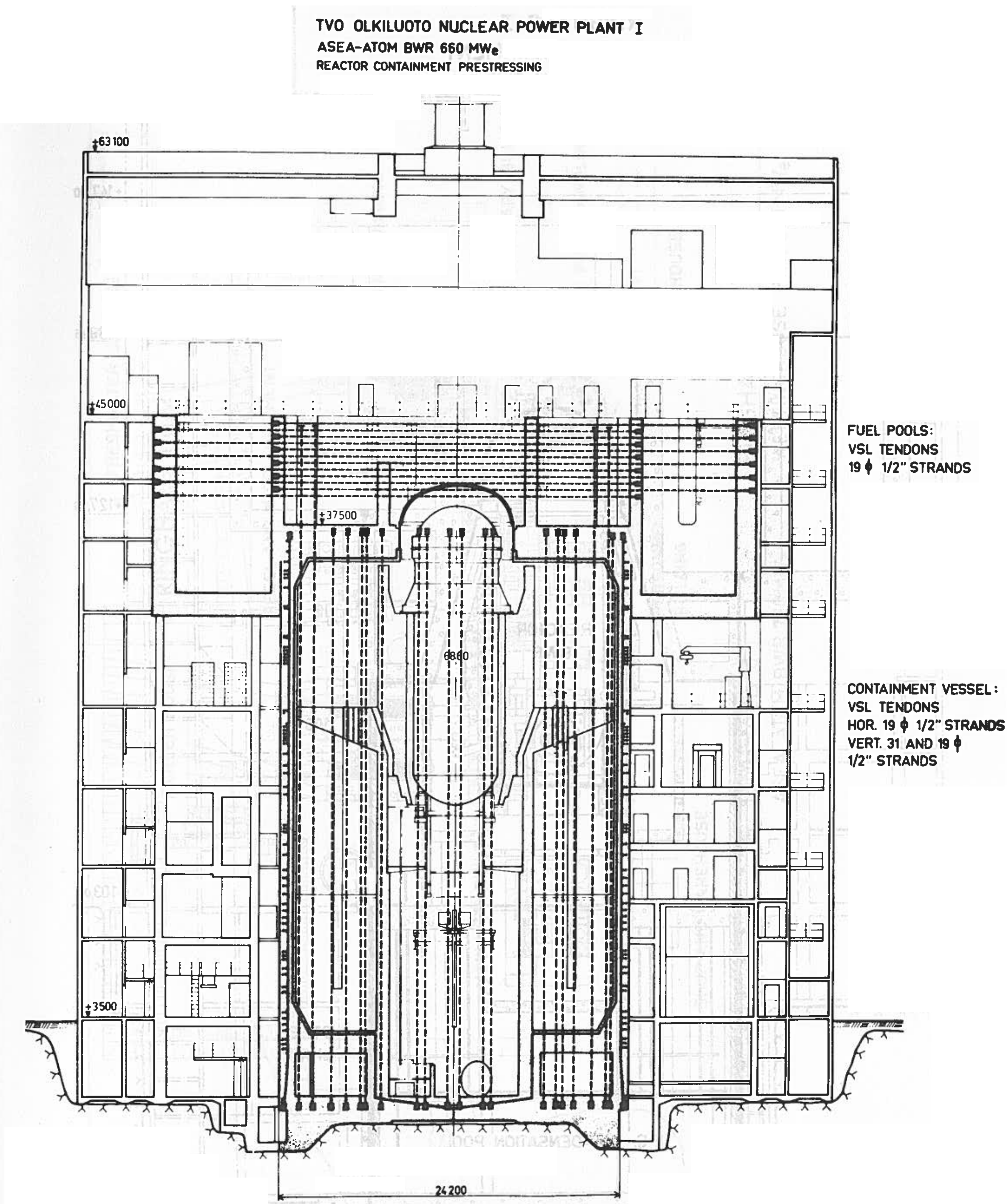
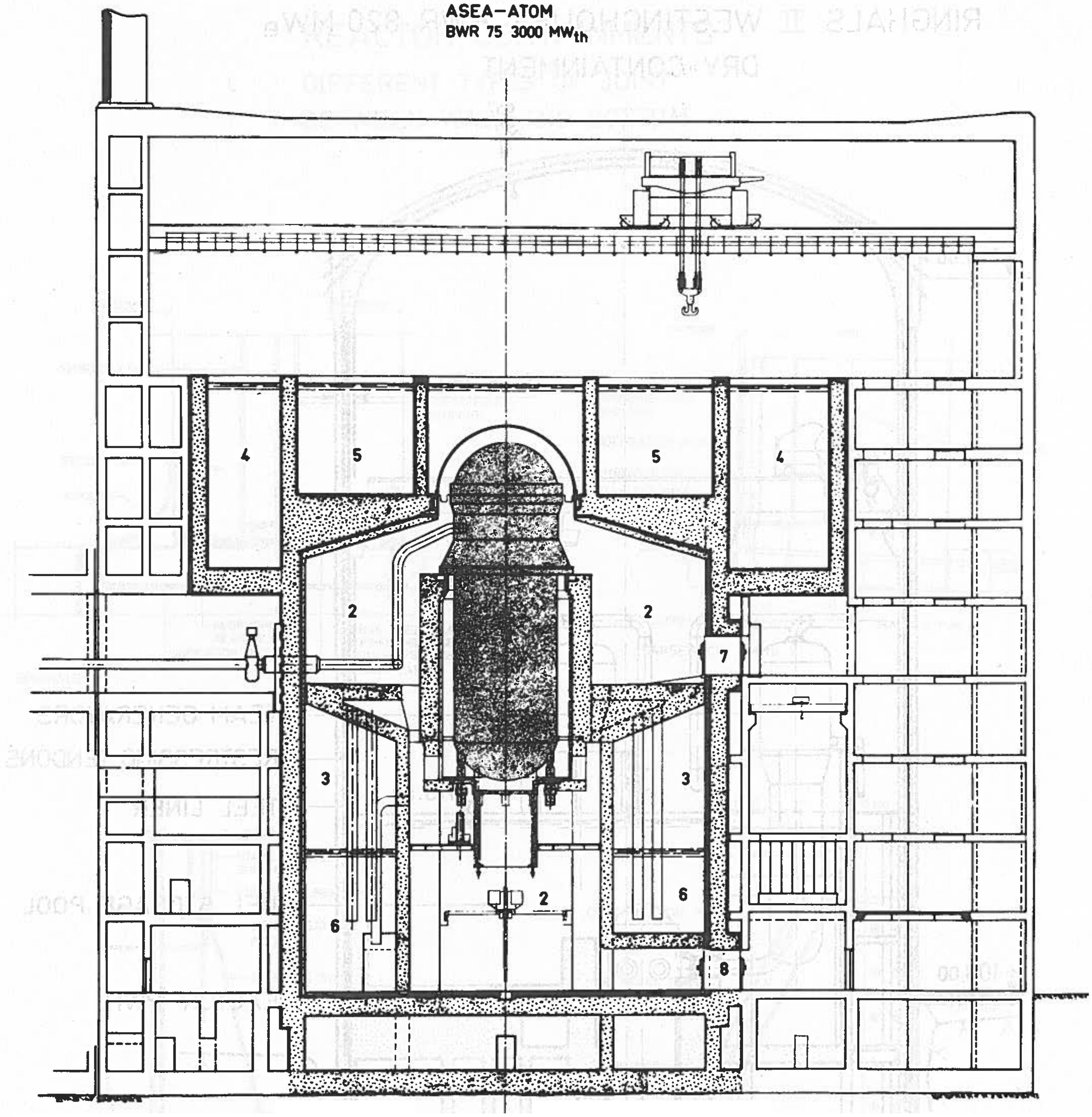

TVO OLKILUOTO NUCLEAR POWER PLANT I
ASEA-ATOM BWR 660 MWe
REACTOR CONTAINMENT PRESTRESSING
+63100
+45000
+37500
6860
+3500
24200
FUEL POOLS:
VSL TENDONS
19 ϕ 1/2" STRANDS
CONTAINMENT VESSEL:
VSL TENDONS
HOR. 19 ϕ 1/2" STRANDS
VERT. 31 AND 19 ϕ
1/2" STRANDS

Figure 10.



1. REACTOR
2. CONTAINMENT DRY WELL
3. CONTAINMENT WET WELL
4. FUEL STORAGE POOL
5. REACTOR INTERNALS POOL
6. CONDENSING POOL
7. UPPER LOCK
8. LOWER LOCK

Figure 11.

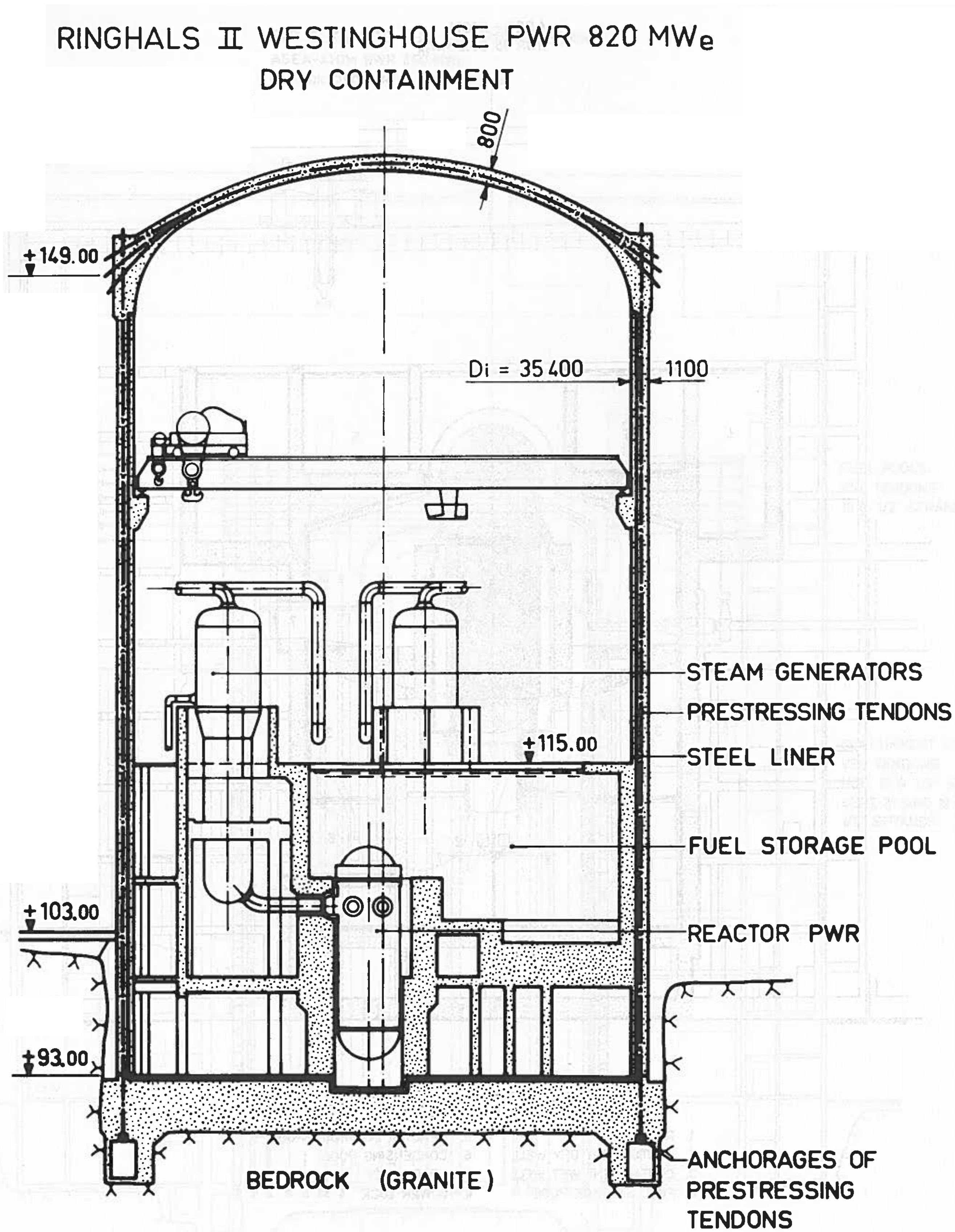

RINGHALS II WESTINGHOUSE PWR 820 MWe
DRY CONTAINMENT
800
+149.00
Di = 35 400
1100
STEAM GENERATORS
PRESTRESSING TENDONS
+115.00
STEEL LINER
FUEL STORAGE POOL
+103.00
REACTOR PWR
+93.00
BEDROCK (GRANITE)
ANCHORAGES OF PRESTRESSING TENDONS

Figure 12.

REACTOR CONTAINMENTS

DIFFERENT TYPES OF JOINT BETWEEN WALL AND BOTTOM

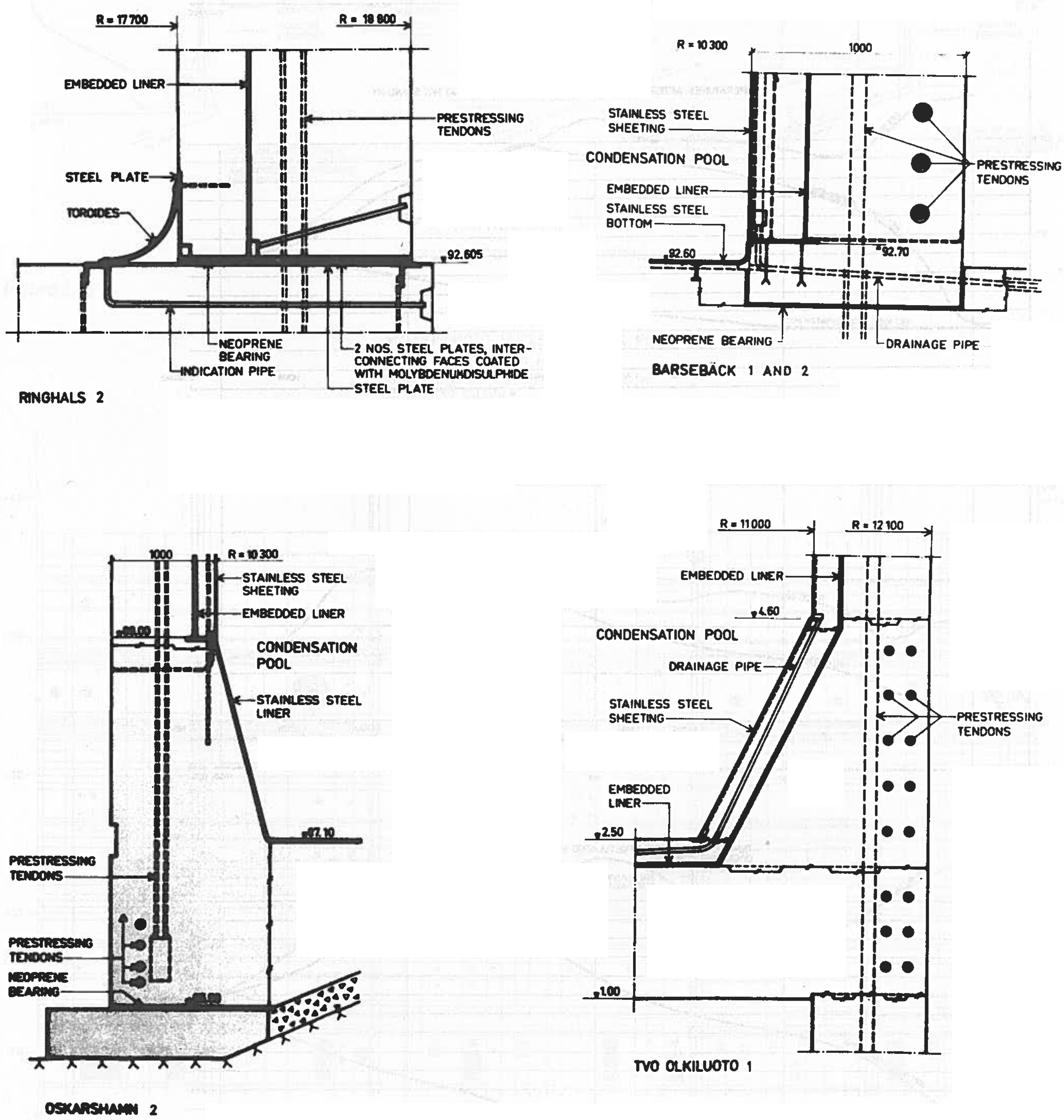
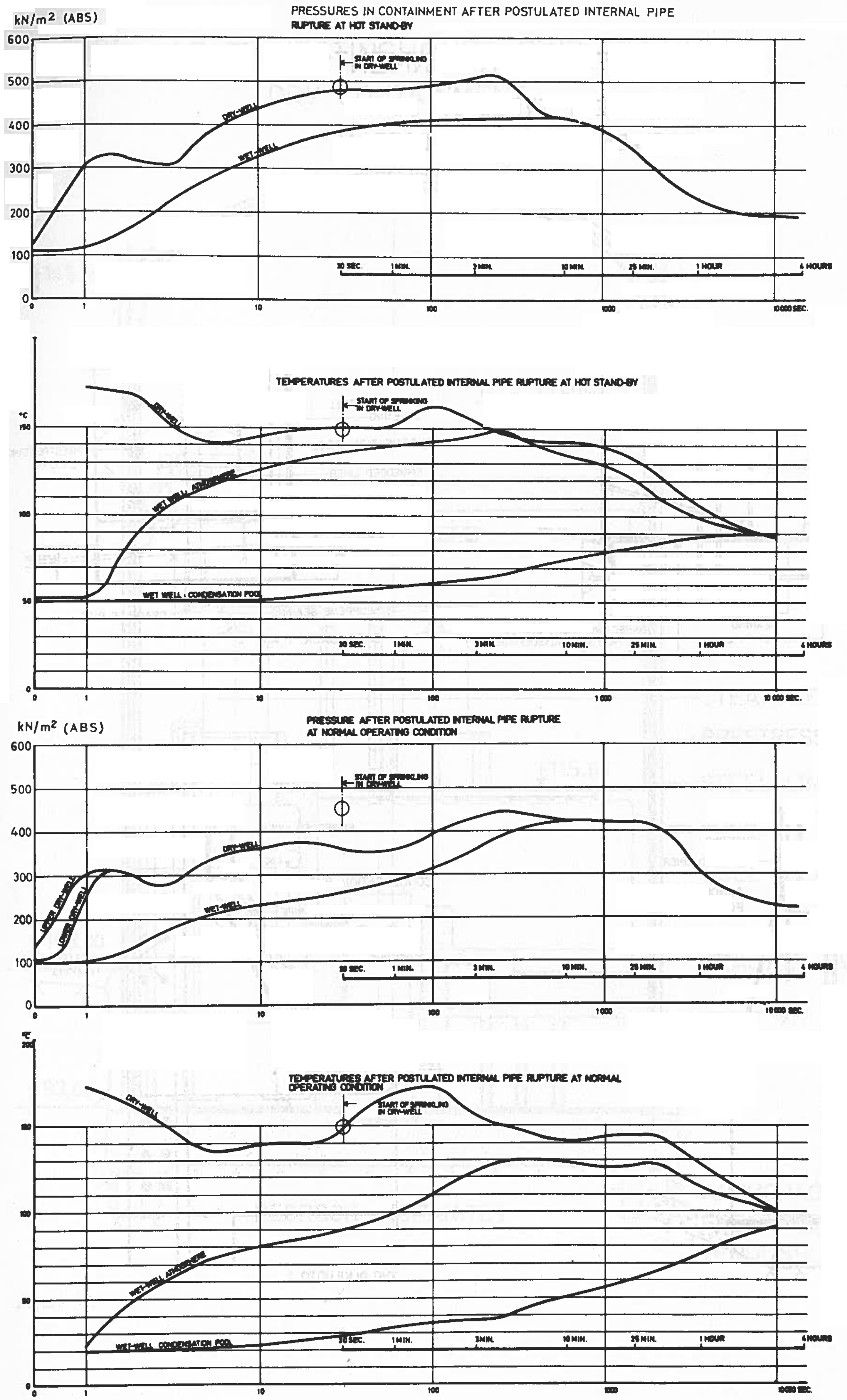


Figure 13.



FORSMARK I

Figure 14.

TRANSIENT PRESSURES AND TEMPERATURES FOR DESIGN BASIS ACCIDENT

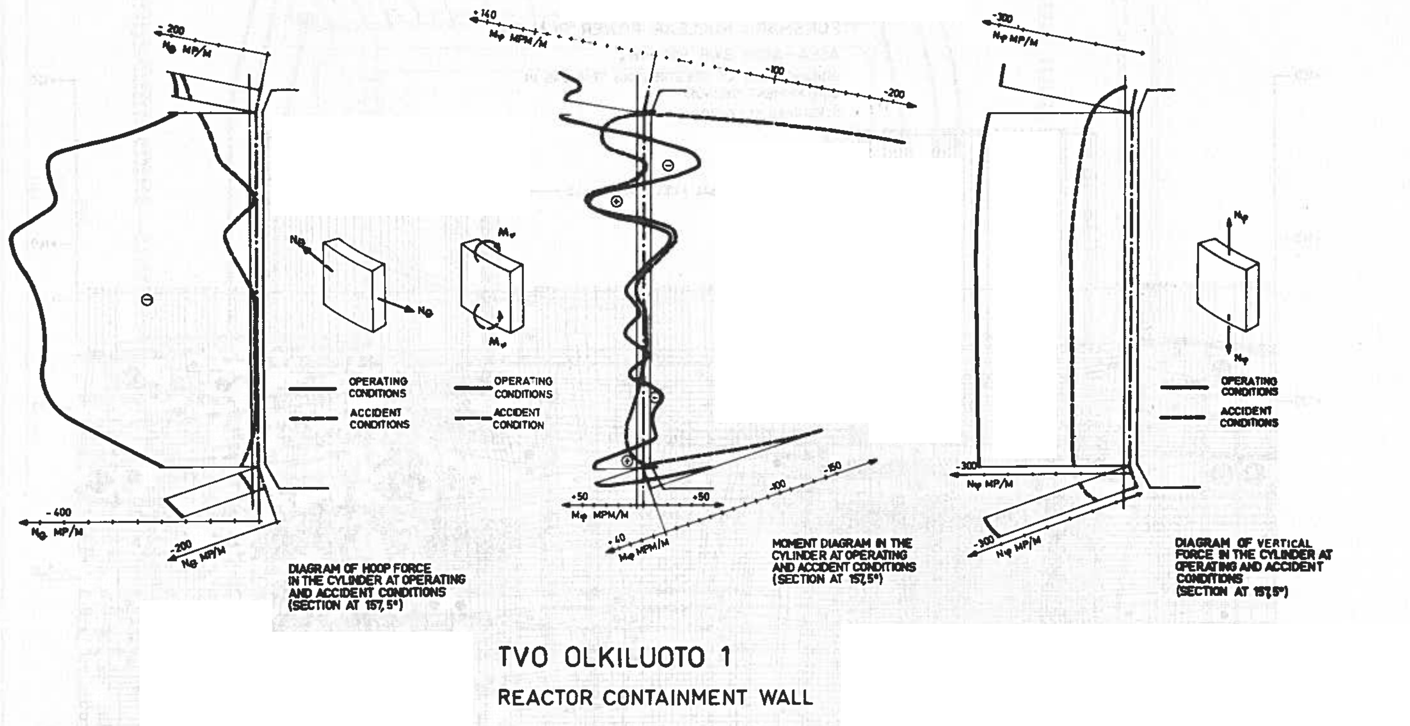


Figure 15.

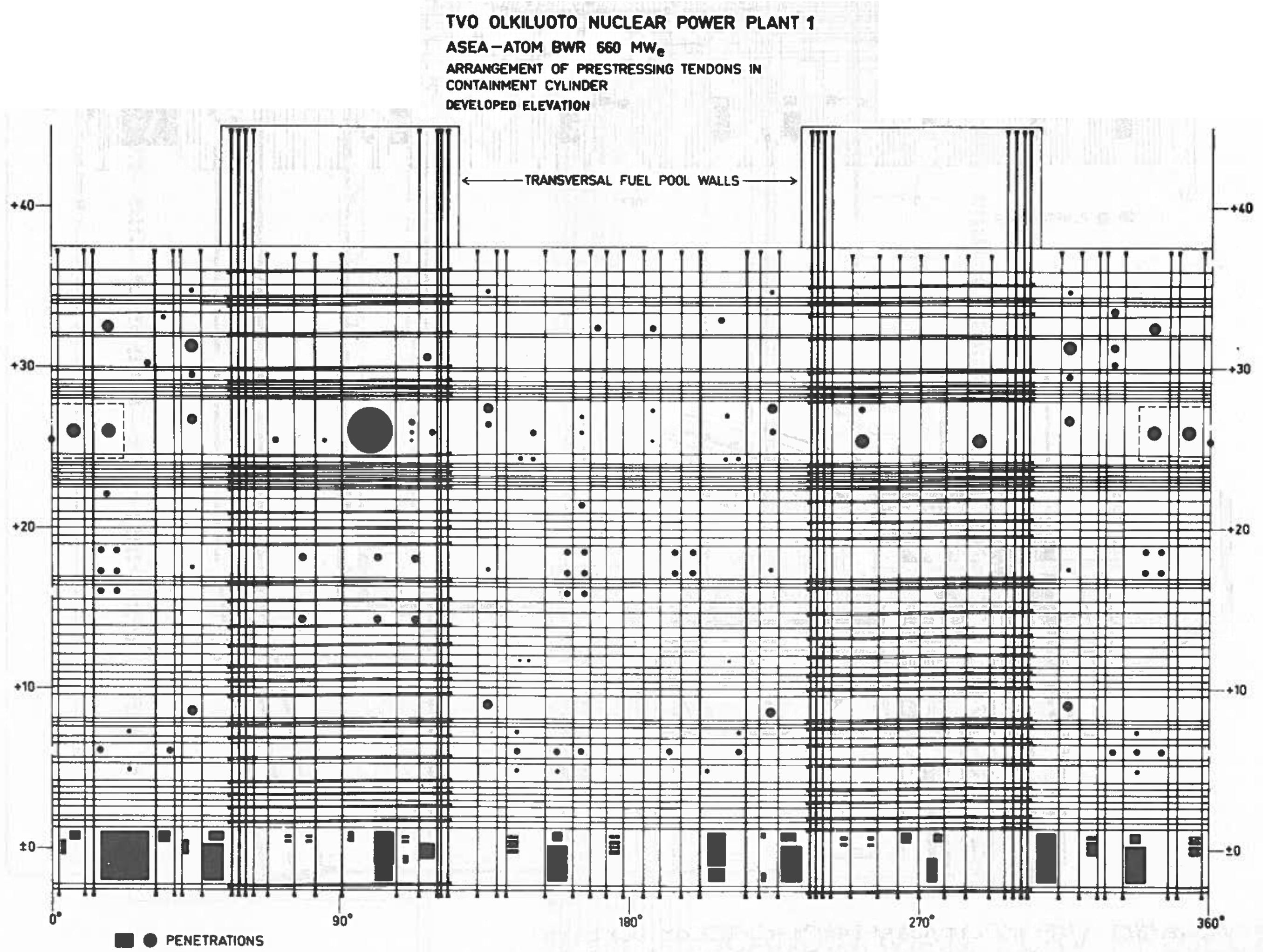


Figure 16.

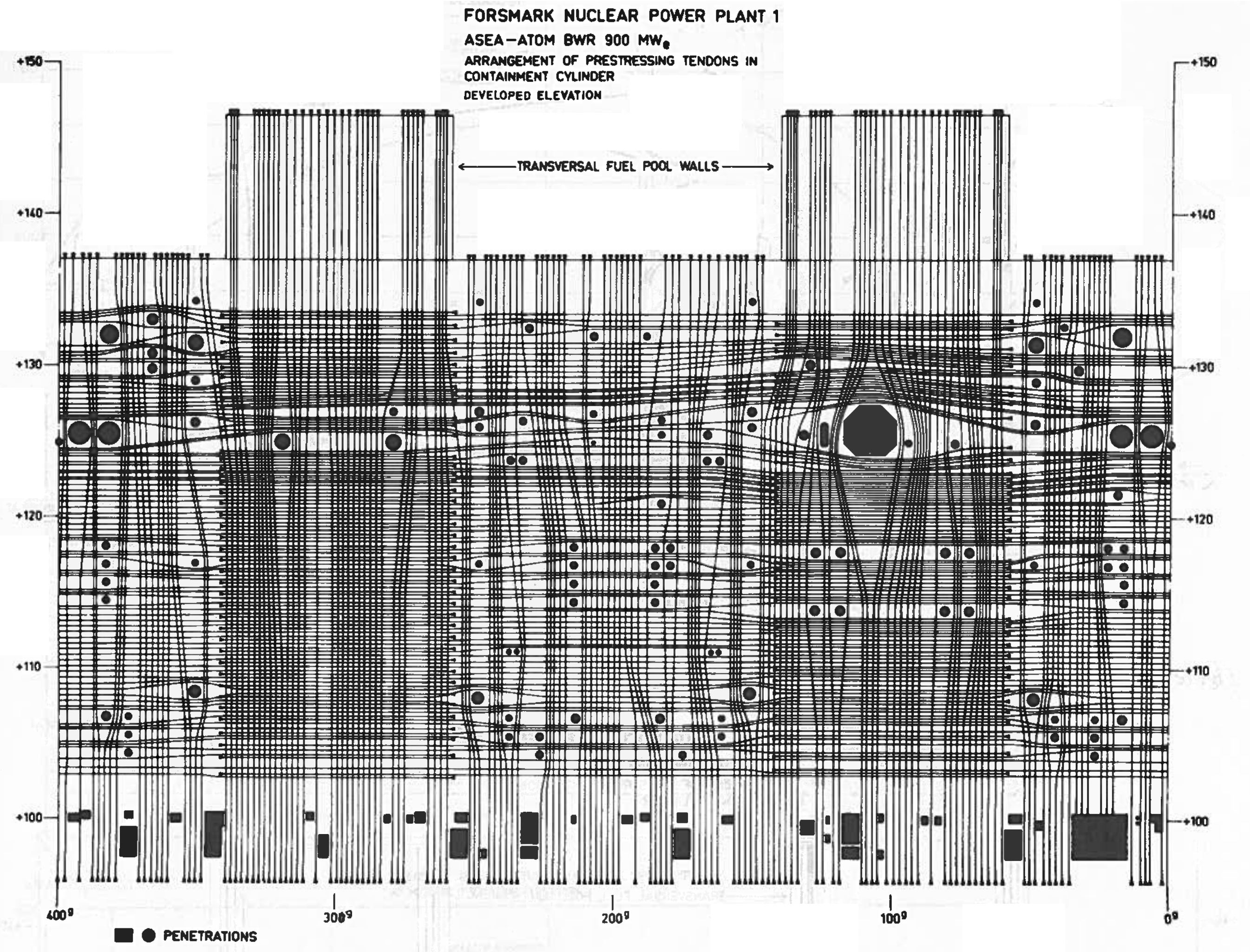


Figure 17.

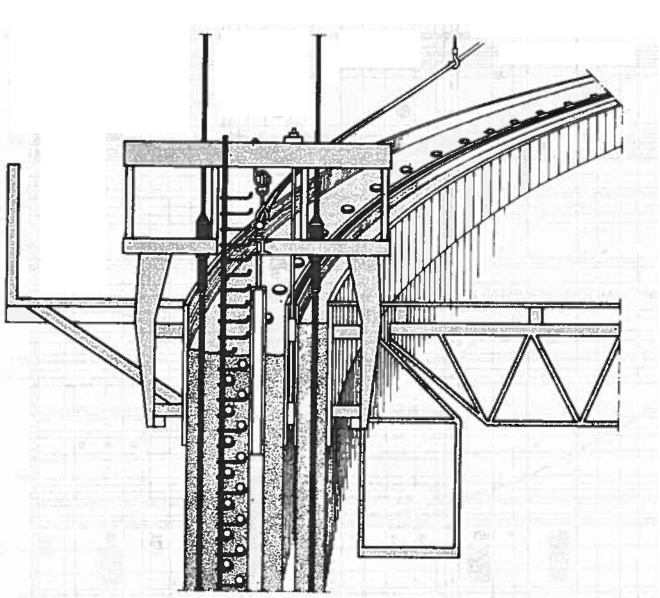


Figure 18. Slipform casting method for containment vessels at Oskarshamn 1 and 2.

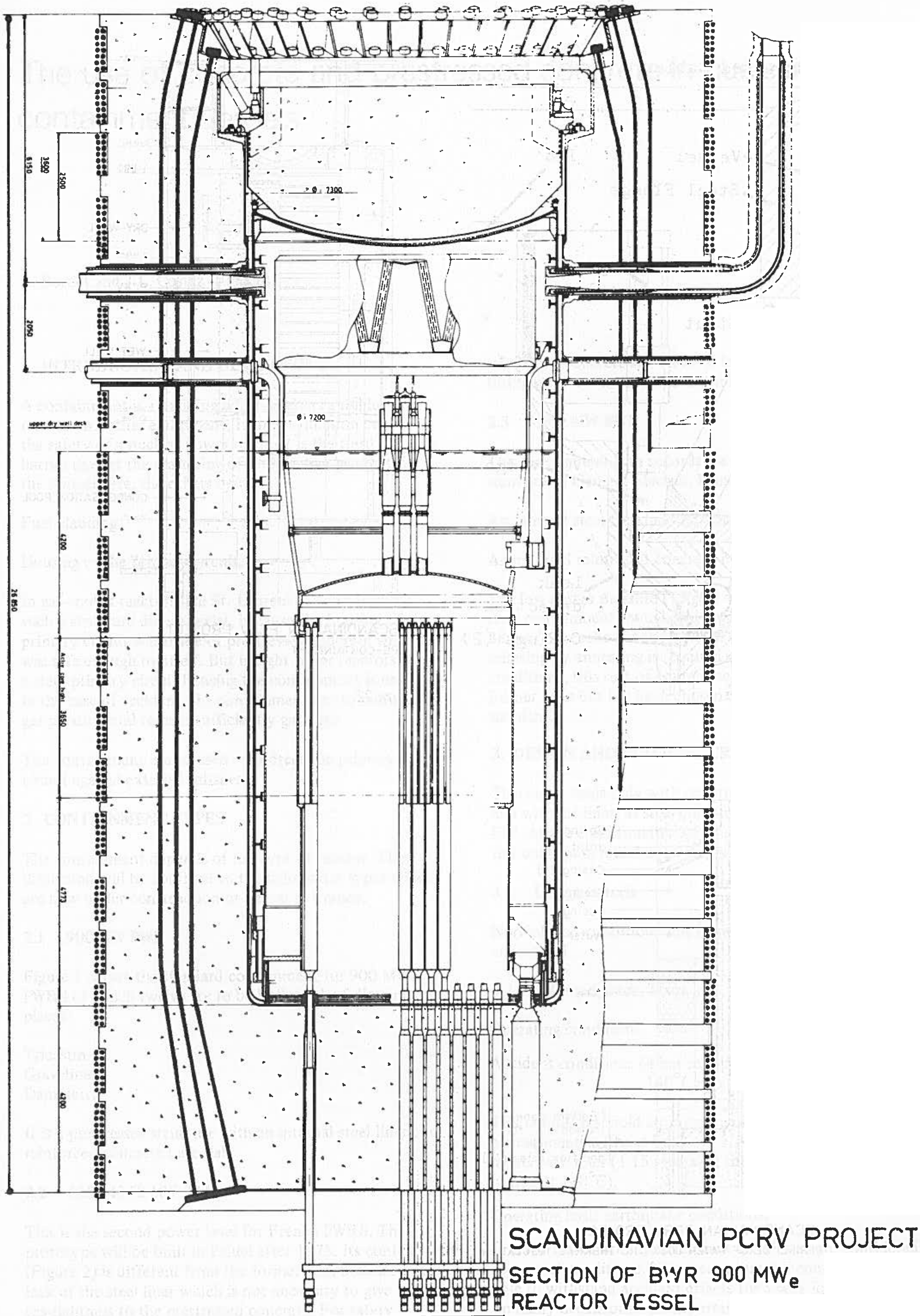

SCANDINAVIAN PCRV PROJECT
SECTION OF BWR 900 MW_e
REACTOR VESSEL
Ø 7300
Ø 7200
upper dry well deck
6150
3600
2500
2050
26 850
4200
3650
4773
4700

Figure 19.

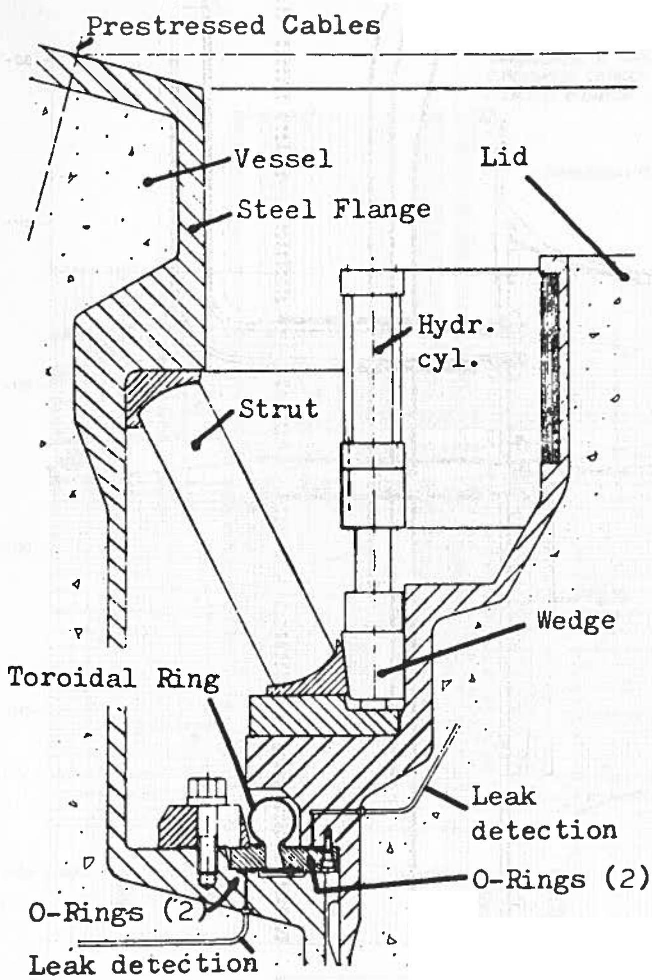


Figure 20.

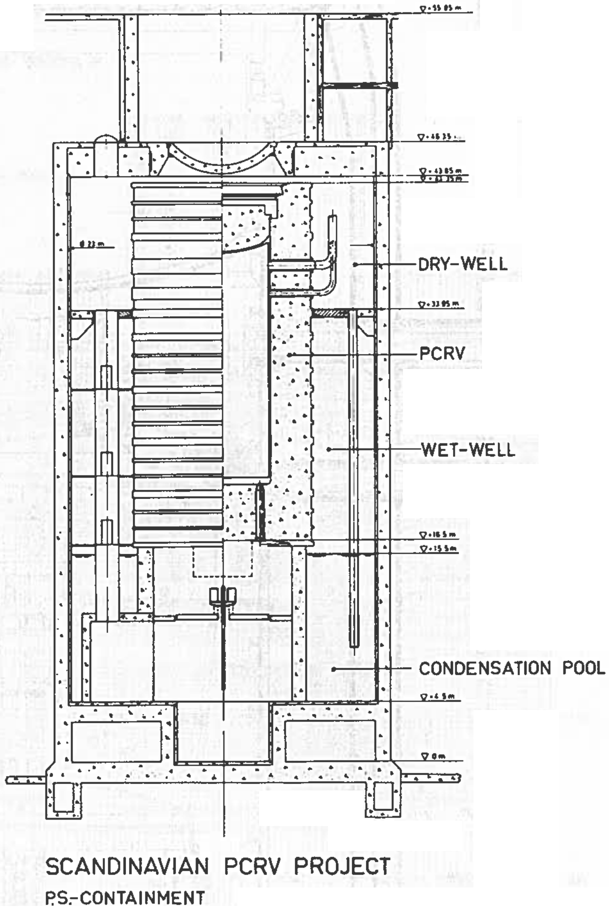


Figure 22.

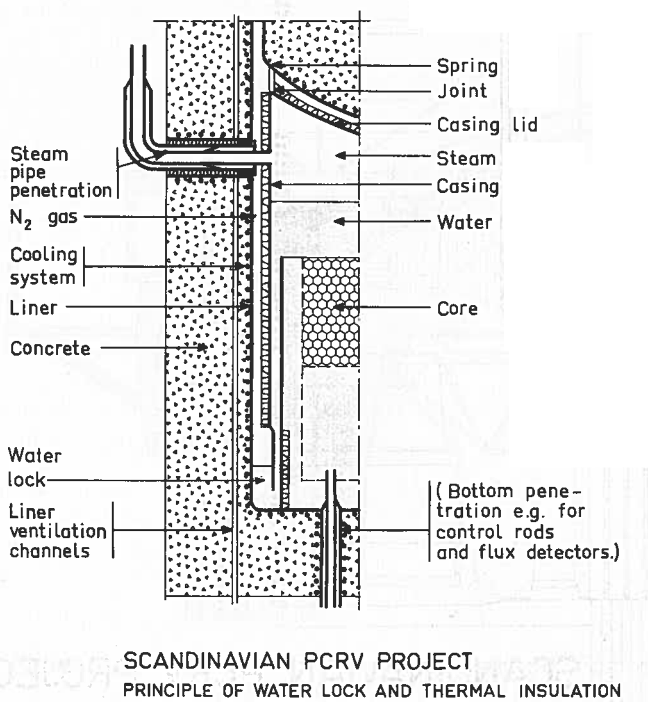


Figure 21.

The use of concrete and prestressed concrete in nuclear containment vessels

R. Bordet and J.L. Costaz (France)

1. INTRODUCTION AND DEFINITIONS

A containment is a building which shelters a nuclear reactor and other equipment. Its main function concerns the safety of a nuclear power plant. It is the third and last barrier against the scattering of radio-active products in the atmosphere, the others being:

Fuel cladding.

Housing of the primary circuit.

In gas-cooled reactors like St. Laurent 1-2 and Bugey 1, such a structure did not exist, because the housing of the primary circuit was made of prestressed concrete which was safe enough by itself. But in light water reactors with a steel primary circuit housing the containment is necessary; in the case of accident, the containment has to withstand gas pressure and remain sufficiently gas-tight.

The containment is also used to protect the primary circuit against external missiles.

2. CONTAINMENT TYPES

The containment depends of the type of reactor. This discussion will be confined to the light water types which are now under construction or design in France.

2.1 900 MW PWR

Figure 1 shows the standard containment for 900 MW PWRS of which twelve are to be built in the following places:

Tricastin
Gravelines
Dampierre

It is a prestressed structure with an internal steel liner and reinforced concrete base slab.

2.2 1200-1350 MW PWR

This is the second power level for French PWRS. The prototype will be built in Paluel after 1975. Its containment (Figure 2) is different from the former one, because of the lack of the steel liner which is not necessary to give a good gas-tightness to the prestressed concrete. For safety reasons, the containment is double and includes an external reinforced concrete structure. A leakage recovery and filtering system is provided for in the annular space.

2.3 1000 MW BWR

The containment is in accordance with the Mark III standard of General Electric. It includes:

An internal steel freestanding vessel;

An external reinforced concrete building.

The low design pressure (15 psig = 1 bar rel.) is suitable for steel containment though the limit thickness (1½ in. = 38 mm) is reached at the bottom (beyond this value, stress-relieving by annealing is required). Under French economic conditions, this sort of containment is more expensive than former ones but has been chosen to conform with the GE standard.

3. DESIGN AND BUILDING CRITERIA

This paper deals only with concrete containments with and without liner, as steel containments do not concern FIP. As the chief function of containment concerns safety, this concept is found at all stages of the project.

3.1 Design criteria

Normal load conditions and factored load conditions are considered.

3.1.1 Normal loads. There are:

Operating conditions.

Accident conditions (4 bar rel = 58 psig, 140°C max = 284°F max).

Test conditions: cold air pressure of 4 bar (58 psig) for containments without liner and 1.15 x 4 bar for containments with liner (1.15 represent the liner thrust when it is heated at 140°C).

Operating basis earthquake conditions.

It is worthy of note that accident conditions are considered as normal conditions. This means that the containment is able to withstand accident effects for a very long time or on many occassions. An important safety margin is connected with this concept.

3.1.2 Factored loads. These correspond to ultimate conditions defined by the yield point of steel:

Pressure increased by 1.5.

Safe shutdown earthquake (SSE).

Accident + safe shutdown earthquake.

Pipe ruptures.

External missiles.

3.2 Calculations

3.2.1 Approximate calculations to find dimensions of concrete and prestress. For a containment with a liner, prestressing forces balance the accident pressure and the liner thrust.

For a linerless containment, the concrete has to remain in compression beyond 10 bar (150 psi) under accident conditions but thermal phenomena in concrete do not cumulate with pressure effects because they do not act at the same time.

The mat thickness depends on the ground characteristics and the earthquake level. Convenient values are 3.50 m (12 ft) for bad ground and an SSE level characterised of 0.2 g horizontal acceleration.

3.2.2 Seismic calculations. The containment and its internal structures are divided into ten or twenty elements connected to each other by stiffness and damping characteristics. The ground is defined by its Young's dynamic modulus. Seismic motions are defined by horizontal and vertical accelerations and by an oscillator spectrum which is the average of eight Californian recordings.

The Transeisme code written in Fortan IV for CDC 6600 gives accelerations, stresses and strains for each element.

3.2.3 Finite element static calculations. When former calculations have determined the dimensions of the containment, a two-dimensional elastic computer method is used. The ground is also considered as elastic but its Young's modulas varies with depth below the surface and loading time.

The loads considered are:

Dead loads.

Internal pressure + liner thrust.

Prestressing forces.

Shrinkage of concrete.

Thermal effects under operating conditions.

Thermal effects under accident conditions.

Seismic effects.

For this last case, forces are not axisymmetrical: results of seismic calculation (§ 3.2.2) are introduced by means of Fourier series.

3.2.4 Liner circulations. The liner must always adapt itself to the concrete strains.

At the start of an accident situation, the liner is heated to more than 100°C (212°F) and the concrete is still cold. It reaches its yield point in compression and its thrust on the concrete is very high.

The liner is thickened around holes and calculation becomes very difficult.

3.2.5 Checking calculations. If need be, it is possible to check the structure with a three dimensional finite element method taking into account static and dynamic phenomena. Such a calculation gives a better representation of non-axisymmetrical effects such as:

Internal structures.

Cellular mats.

Seismic forces.

Missile effects.

Local forces.

3.3 Choice of prestressing system

For Fessenheim 1-2 and Bugey 2-3-4-5 containments, the contractor used Stup 12 T 15 units:

rupture strength: 300 t (660 Kip)
service strength: 170 t (375 Kip).

Wires are normal relaxation wires.

For 900 MW standard project, EDF chose the Stup 19 T 15 unit:

rupture strength: 470 t (1040 Kip)
service strength: 290 t (640 Kip).

Wires are very low relaxation class II wires.

BBR 84 ∅ 7 and VSL 5.31 systems are also considered on condition that the service strengths after deduction of all losses are the same as previously. As a matter of fact, the same drawings must remain valid for all the plants of the 900 MW standard type.

3.4 Building criteria

The most important points are:

Concrete quality.

Prestressing wires and grouting quality.

Liner quality.

3.4.1 Concrete. Sample tests give characteristic data on concrete. Its compression strength has to be greater than 400 bar (5800 psi) on cylinders after 28 days.

For linerless containments, the concrete has to be impervious enough. Quality of construction joints must be checked.

3.4.2 Liner. For lined containments, the concrete does not have to provide the seal as this is done by the liner.

The criterion of 0.1% volume loss of the free air contained in the vessel after 24 hours means that more or less perfect gas-tightness is required. Therefore, during construction, all welds are vaccuum tested and 10% are gamma-ray tested.

3.5 Final test

At the end of building operations, the containment is put under test pressure with cold air. For linerless containments the air is at the design pressure. For lined containments, a factored coefficient of 1.15 allows for the liner thrust during accident.

The required gas-tightness expressed as a percentage of the air in the vessel during 24 hours is:

0.1% for lined containments
1% for unlined containments.

In the last case, gas-leaks are collected in the annular space and filtered before being discharged.

3.6 Supervision of the structure

Periodic leak-tests are made under a sufficient pressure (⩾2 bar rel = 29 psig).

Measuring instruments are installed to monitor the structure:

Vibrating wires and thermocouples in the concrete.

Dynamometers on special ungrouted prestressing cables.

Topographic measurements.

Stress-gauges (for trial use).

4. ADVANTAGES OF THE LINERLESS DOUBLE CONTAINMENT

4.1 Reasons for the choice

The structure is normally not pressurised.

Accident pressure is relatively low and of short duration.

Gas under pressure contains four-fifths steam.

The problems are very different from thos of PCRVs. One has tried to do away with the liner, to save cost and simplify construction.

The first idea is to use paint instead of a liner but this has drawbacks:

Manufacturer's guarantees generally do not exceed five years.

The coating has to resist acids and bases.

It also has to resist high temperatures: 140°C (284°F) on average, 180°C (354°F) locally.

It is preferable to develop a solution in which gas-tightness is provided by the prestressed concrete alone. The rate of 0.1% could probably be obtained. Nevertheless, for safety reasons, an external reinforced concrete building has been provided in the first plants with leakage recovery and filtration of leaks in the annular space. Thus, radio-active ejections would be very small, even with a leak-rate exceeding all expectations.

4.2 Concrete gas-tightness tests

Many tests have been carried out to test gas and steam-tightness of concrete:

At CEA (a)* in Saclay on small test-pieces
At CEBTP (b)+ in St Rémy les Chevreuse on a large 10 ft high mock-up (Figure 3) under air and steam pressures up to 4 bar eff (58 psig).

There were only vertical prestress tendons for testing prestressing effect on cracks and a model included voluntary faults such as concrete defects and faulty construction joints.

The leak-rates measured (calculated for the containment itself) vary between 0.2 and 0.4% over 24 hours in spite of these faults.

This large scale model has also been used to study several types of methods of repairing cracks by injecting resins.

4.3 Advantages of this containment

In addition to substantial savings, it is worthy of note that:

It eliminates complications, due to the liner in every field: design, calculations, building, schedule.

The pressure test is significant.

Gas tightness is provided in the mass and does not only depend on membrane behaviour.

The prestressing tendons are well protected against weather by the external building.

There is greater strength against external missiles owing to the double barrier.

*(a) Commissariat à l'Energie Atomique.
+(b) Centre d'Etudes du Bâtiment et des Travaux Publics.

5. CONCLUSION

After being used to insure containment resistance against internal pressure, prestressed concrete is to provide a partial seal against gas leaks. This must always be accompaned by better quality for design and building. Improvements in techniques founded on past experience allow us to view this question with confidence.

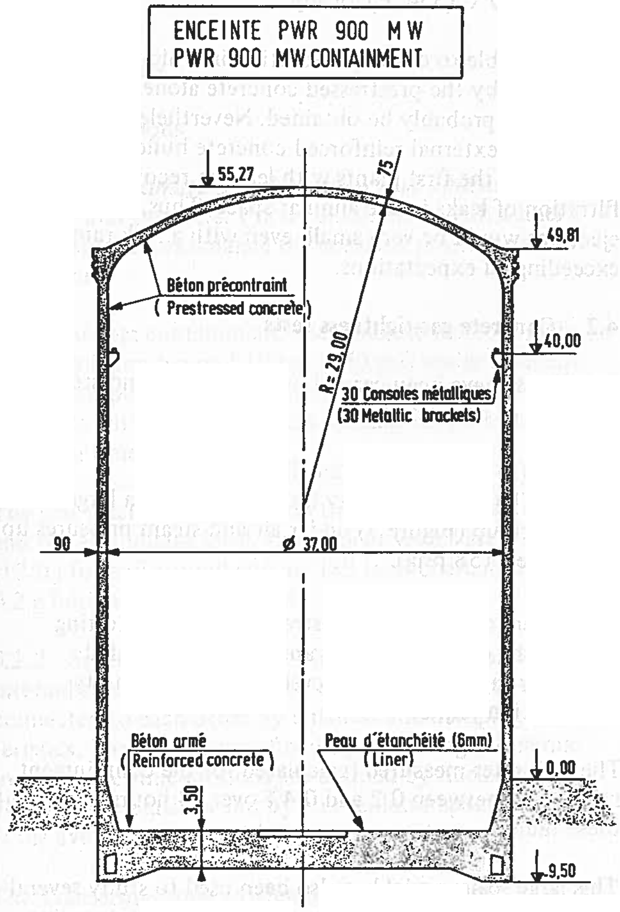


Figure 1.

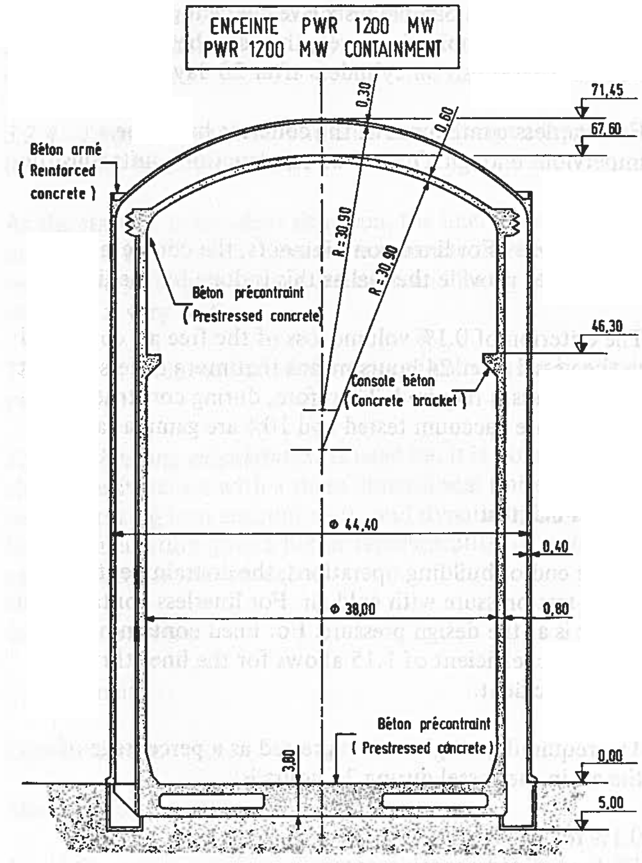


Figure 2.

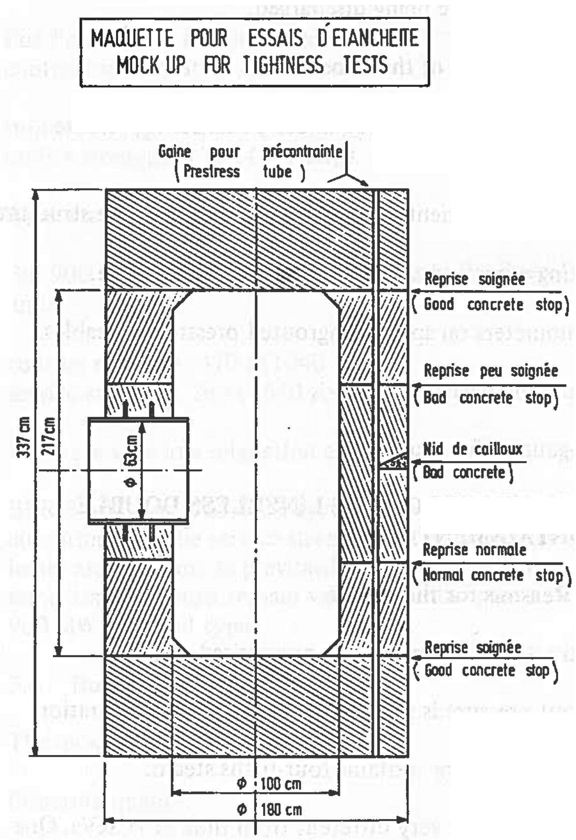


Figure 3.

Prestressed concrete pressure vessels, recent work and future prospects in the United Kingdom

R.E.D. Burrow (UK)

INTRODUCTION

Fourteen prestressed concrete pressure vessels have been built or are under construction in the United Kingdom, twelve of which are for the Central Electricity Generating Board and two for the South of Scotland Electricity Board. Of these, two vessels at the Oldbury Nuclear Power Station have given successful service since 1968 and two at Wylfa since 1970. Four others at Hinkley and Hunterston Nuclear Power Stations are in the final stages of testing and commissioning, and six vessels at the Hartlepool, Heysham and Dungeness stations are at an advanced stage of construction. This paper briefly reviews the most recent work associated with this programme of construction and comments on the information obtained from periodic surveys of the vessels already in service.

British Standard 4975 Specification for prestressed concrete pressure vessels for nuclear reactors, published in July 1973, expresses current British practice in the design of these structures and comments are offered on some important aspects of this Standard.

The paper concludes by discussing possible future requirements for vessels in the UK in the light of developments in reactor technology and against the background of national and world attitudes to nuclear power.

RECENT WORK

In the most recent part of the UK nuclear programme, concerned with the construction of Advanced Gas-cooled Reactors (AGRs), two distinct types of pressure vessel have emerged. Some significant features of these two styles are outlined below.

Single cavity style of vessel

Figure 1 illustrates the general arrangement of the prestressed concrete pressure vessels at Hinkley Point 'B' and Hunterston 'B'.(1) In this style of design the vessel is cylindrical in shape, enclosing a single central cavity within which is contained the reactor core and shielding and twelve boiler units.

The arrangement of the prestressing tendons is a development of that used for the Oldbury Pressure Vessels, and consists of sixteen layers of tendons within the thickness of the vessel walls following helical paths, alternately clockwise and anticlockwise, at approximately 37° to the horizontal. The tendons continue in annular extensions of the vessel walls beyond the end slabs and anchor in the top and bottom surfaces of the vessel; radial components of force from the extended tendons providing the prestress necessary for the end slabs. This arrangement makes it possible to provide galleries at the top and bottom of the vessel walls from which access is available at all times for stressing during construction, and for the inspection, testing and replacement of tendons when the vessel is in service (Figure 2).

The form of prestressing tendon is particularly ingenious since it provides a tendon with an equivalent ultimate strength of 1040 tons (1057 tonne) which can be prestressed one strand at a time using a light automatic jack. The system is an adaptation of the CCL 707 system in which four cables of 7 x 0.7 in. (18 mm) diameter strands, in separate seam-welded steel ducts, are grouped closely together to provide an equivalent 28 strand tendon. The strands are stressed and anchored individually by conventional barrel and wedge grips. The seven grips of each anchor bear on a (8½ in.) 190 mm square plate which transmits the cable load into the concrete through a cast steel trumpet. The seven-strand anchorages are arranged in groups of four in a series of fabricated metal beams which are filled with grout and pre-set radially across the walls at the top and bottom faces of the vessel (Figure 3). These are incorporated in the structure in the course of casting the vessel walls.

It has been found, in comparison with other arrangements of linear tendons, that the helical system uses a smaller number of anchorages for a given total tonnage of tendons. Because of this, and as a result of the simple form of anchorage and stressing equipment, the stressing system for the Hinkley and Hunterston vessels has proved to be exceptionally convenient and economic.

Multi-cavity style of vessel

Figure 4 illustrates the general arrangement of the prestressed concrete pressure vessels for Hartlepool and Heysham(2). In this style of design the reactor core and its built in shielding are contained in a main central cavity while the boilers are housed in eight vertical pods passing through the full height of the vessel walls and linked by gas ducts to the top and bottom of the main void. Gas circulators are mounted below each boiler and steam and feed connections are made through the top closure of each boiler cavity.

The vessels are prestressed longitudinally using the 28/0.7 CCL system (Figure 5). This provides tendons made up of 28 'Dyform' strands of 0.7 in. (18 mm) diameter, giving a minimum ultimate strength of 1040 tons (1057 tonne) and a working load of 780 tons (792 tonne).

Each tendon is contained in a 6 in. (150 mm) diameter duct which terminates at each end in a trumpet casting where the strands are deviated to anchor individually in a steel bearing plate. The bearing plate seats directly onto an embedded steel casting, which transmits the tendon load into the concrete through a series of flanges on its outer surface. These 28 strands are stressed simultaneously by a single jack which incorporates a load cell. Wedges, anchoring individual strands in the bearing plates, are pressed home hydraulically by a separate ram in the nose of the jack. The prestressing jack can be fitted with a special shimming foot to allow restressing. De-tensioning is carried out one strand at a time using a monostrand jack.

The Taylor Woodrow wire winding system[3] (Figure 6) is used for circumferential prestressing. This builds up concentrated prestressing bands by winding 0.2 in. (5 mm) diameter high tensile steel wire under tension into channels preformed in the vessel walls. Each layer of wire is separately anchored, and wound directly into the grooves formed by the layer below. There are 33 layers of wire giving each band an ultimate strength of 13600 ton (13818 tonne). 20 bands are arranged on a regular module down the external surface of the vessel wall. The circumferential channels are formed in precast concrete units which provide permanent formwork to the outside vertical surfaces of the vessel.

Wire wound tendons give a significant saving in the quantity and cost of circumferential prestressing and allow the intensity of prestress to be varied as required over the height of the structure. They are particularly suited to multi-cavity vessels where the cylinder walls are used to house the boiler pods.

Prestressed concrete boiler closures

The closures to the 32 boiler pods of the Hartlepool and Heysham vessels (Figure 7) are designed as precast concrete cylinders 11 ft (3.35 m) in diameter and 5 ft-6 in. (1.73 m) deep, prestressed by wire winding[4]. Each boiler is suspended from its closure, through which pass nine penetrations carrying steam and feed pipes. The temperature of the unit is controlled by a cooling water system and surface insulation, and the windings are protected from mechanical damage by a steel casing.

The closure is prestressed by nine individually anchored layers of 0.104 in. (2.64 mm) diameter wire. Four layers are sufficient to give the required prestressing force and ultimate strength, and the remainder are included to provide the required degree of 'redundancy' in the design.

Each closure is secured by 48 bolts to its supporting flange within the boiler pod. These are backed up by a series of removable shear keys latched under a reinforced concrete ring which is attached to the vessel by selected tendons of the longitudinal prestressing system.

The surveillance of the Oldbury and Wylfa pressure vessels

Key aspects of all nuclear power stations in the UK are checked annually in a systematic procedure of surveillance which includes a detailed survey of the prestressed concrete pressure vessels. An important aspect of such inspections is a verification of the physical condition of the steel and value of load in a random sample of prestressing tendons.

Satisfactory methods have been evolved for checking the load in prestressing tendons by re-applying the prestressing jacks and loading these until the anchors lift clear of their seatings. It is necessary to measure both the load and the anchor movement very accurately to determine the true lift off load and account must be taken of the effects of friction. Prestressing tendon forces at both Oldbury and Wylfa have, so far, stayed reasonably close to the predicted values.

Physical examinations have shown no deterioration of the prestressing systems of these vessels apart from minor pitting on some exposed strands at Wylfa in the external circumferential tendons. This has been remedied by improving the operation of the heating and ventilating plant to give a lower relative humidity around the tendons.

Nuclear pressure vessels may well be the only structures on which rigorous measurements are being made of the long term behaviour of prestressing tendons and the information which is being accumulated from them is of considerable interest.

British Standard 4975 Prestressed concrete pressure vessels for nuclear reactors

This Standard[5] was published in July 1973 after more than five years work by the drafting panel. Prudently it provides a framework for the design of prestressed concrete pressure vessels which is closely linked to modern general practice as represented in Code of Practice 110 The structural use of concrete. Wherever possible the clauses of that code are applied unaltered, but a number of problems have required special treatment. In particular, the operating and overload conditions which must be carried by a prestressed concrete pressure vessel depend on the type of reactor which it contains, on the nature of the safeguards built into the reactor itself and on the design and reliability of the ancilliary plant controlling vessel temperatures and pressures. These can only be defined by the designer of the reactor system. The Standard (Section 1.1.) therefore relieves the vessel designer of responsibility for defining the detailed design loadings to be used and requires these to be specified by others.

CP 110 is based upon the limit state philosophy of design. This was not considered practical for pressure vessels, due mainly to the complicating effects of temperature loading. However the option is left open for designers to put forward designs based on limit state methods, where these appear more appropriate. (Clause 3.2.5).

Specific criteria are kept to a minimum. For example, the treatment of local stresses in clause 3.2.3.1.4 and in Appendix C is in the form of guidance covering the practical as well as theoretical factors which may influence the acceptability of high local stresses. Similarly, section 3.2.2. dealing with vessel analysis states the functional requirements to be met. Guidance on the capabilities of different methods of analysis is given briefly in an appendix, and practical factors influencing the validity of the calculations are noted where appropriate in the text.

With ungrouted tendons, the safety of the vessel structure is specially dependent upon the integrity of the prestressing system, and rigorous proving tests are required before any system can be accepted for use (Section 5.3). These tests cover the efficiency of the stressing, re-stressing and de-stressing procedures, the ultimate strength and strain capacity of the stressed tendon and the strength of components transferring the anchorage load to the vessel concrete. CP 110 bases strength calculations on the characteristic strength of the materials of construction. In the case of prestressing steel, 4975 retains the more onerous requirement that no material supplied may be below a guaranteed ultimate tensile strength (GUTS).

Design limits for concrete stresses given in the Standard include criteria for the assessment of multi-axial stresses. These require conservatively that the difference between maximum and minimum principal field stresses in the concrete should not exceed 33 or 40% of the cube strength, depending on the loading conditions under consideration. This ensures that deformations of the concrete due to creep remain predictible. The increase of permissible stress is only allowed when the minimum principal stress exceed $0.125U_W$.

The Standard is a first document and a valuable codification. However the technology of prestressed concrete pressure vessels continues to develop very quickly and new work will soon be necessary to update the text.

FUTURE PROSPECTS

Future prospects for nuclear power in the UK depend greatly on the way in which our energy policy is reshaped over the next few months and the rate of economic growth which takes place in the years ahead.

The installed generating capacity in the UK at the end of 1973 was 65 000 MWe. Bainbridge[6] has estimated that by 1985 this will need to rise to 12 500 (Figure 8). If the same annual growth rate (5½%) is assumed to continue to the year 2000, the requirement at that time reaches 27 5000 MWe. Allowing for the replacement of existing plant this would mean that 130 new generating stations of 2000 MWe capacity might need to be constructed in the UK during the next 26 years, a high proportion of these being nuclear power stations.

At the time of writing, debate continues in the UK about the type of reactor which should be chosen as the basis of our next programme of power station construction. However, whatever the choice, prestressed concrete pressure vessels or containments will be needed for all nuclear power stations and the volume of work ahead is considerable.

It seems highly likely that one high temperature gas cooled reactor will be ordered quite soon and that this may be followed by a programme of such stations[7]. The pressure vessel used for this system is a direct development from the earlier AGR vessel shown in Figure 4.

If the programme includes pressurised water reactors based on American technology or the British designed SGHWR, the pattern of future work will include prestressed containment structures such as are already familiar in the United States. Hitherto, these structures have been provided primarily to limit the effects of failures or faults in the reactor system. In today's disturbed social conditions however, increased attention is being given to the possibility of damage arising from aircraft crashes, explosions due to accident, informed sabotage or insurrection. Thus future nuclear power stations may embody additional protection covering all areas whose damage could affect public safety.

If development of the fast breeder reactor (Figure 9) continues successfully, a substantial part of our generating capacity at the end of the century may be powered by this type of reactor. In the present design, the reactor core and primary heat exchanges are immersed in a pool of liquid sodium. This pool is contined within a steel membrane which is hung within a substantial concrete containment vessel. The purpose of this vessel is to support the sodium tank and contain, without leakage, the worst loadings which could arise from a fault condition within the reactor.

CONCLUSION

Although the pace of nuclear work in the UK has been disappointly slow for the past few years, there is every reason to believe that this is a temporary situation. The evident need to reduce our dependence on fossil fuels, and the growing demand for energy must lead to a growing programme of nuclear power and the civil engineering and prestressed concrete work which this will engender.

In this paper it has only been possible to comment briefly on a few aspects of a wide subject. An International Conference is being organised in York, England on 8-12 September 1975 the theme of which is Experience in the Design, Construction and Operation of Prestressed Concrete Pressure Vessels and Containments for Nuclear Reactors. It is hoped that this will provide a forum for the presentation of world experience with this important form of prestressed structure. (Full details may be obtained from: A.J. Tugwell, Committee Secretary, Institution of Mechanical Engineers, 1, Birdcage Walk, Westminster, London SW1H 9JJ).

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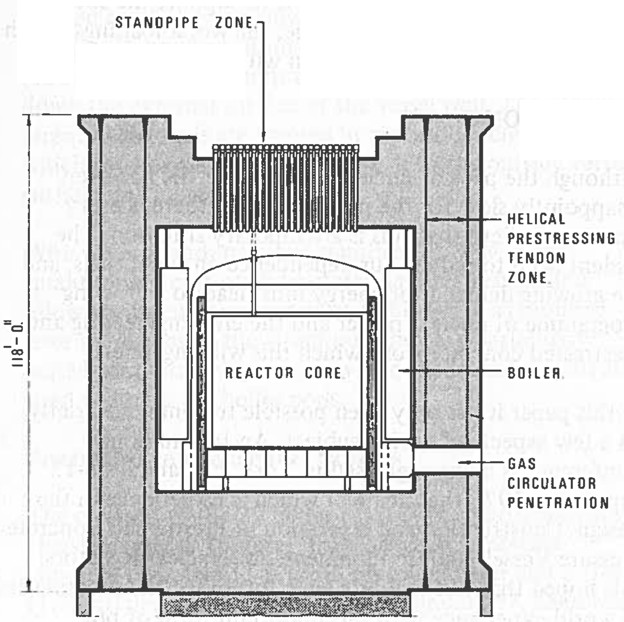

Figure 1. Section through Hinkley Point 'B'/Hunterston 'B' pressure vessel.

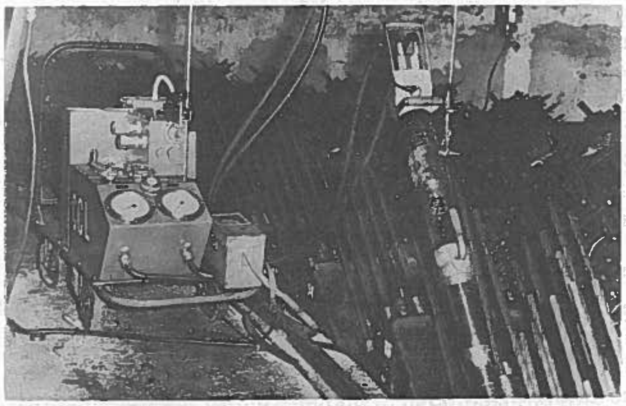

Figure 2. Hinkley Point 'B', prestressing in progress.

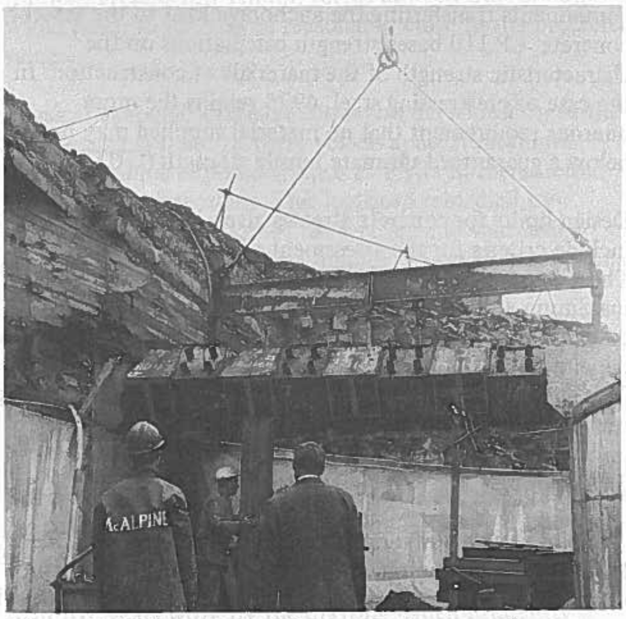

Figure 3. Hinkley Point 'B', positioning an anchorage beam in the lower stressing gallery.

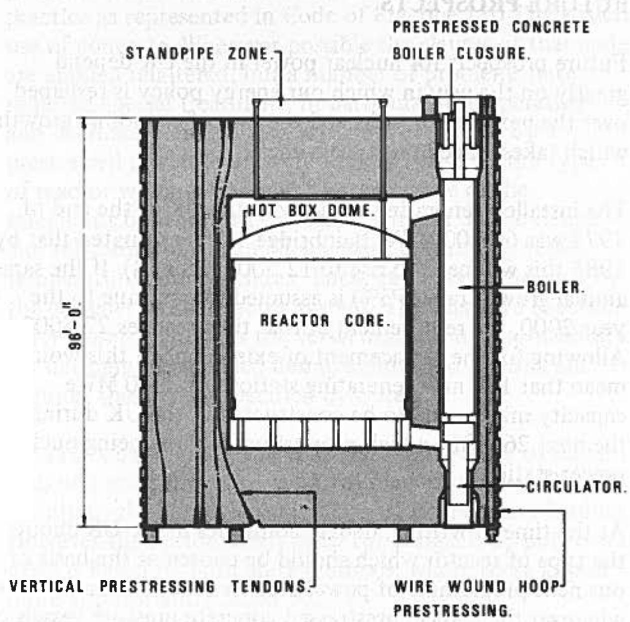

Figure 4. Section through Hartlepool/Heysham pressure vessel.

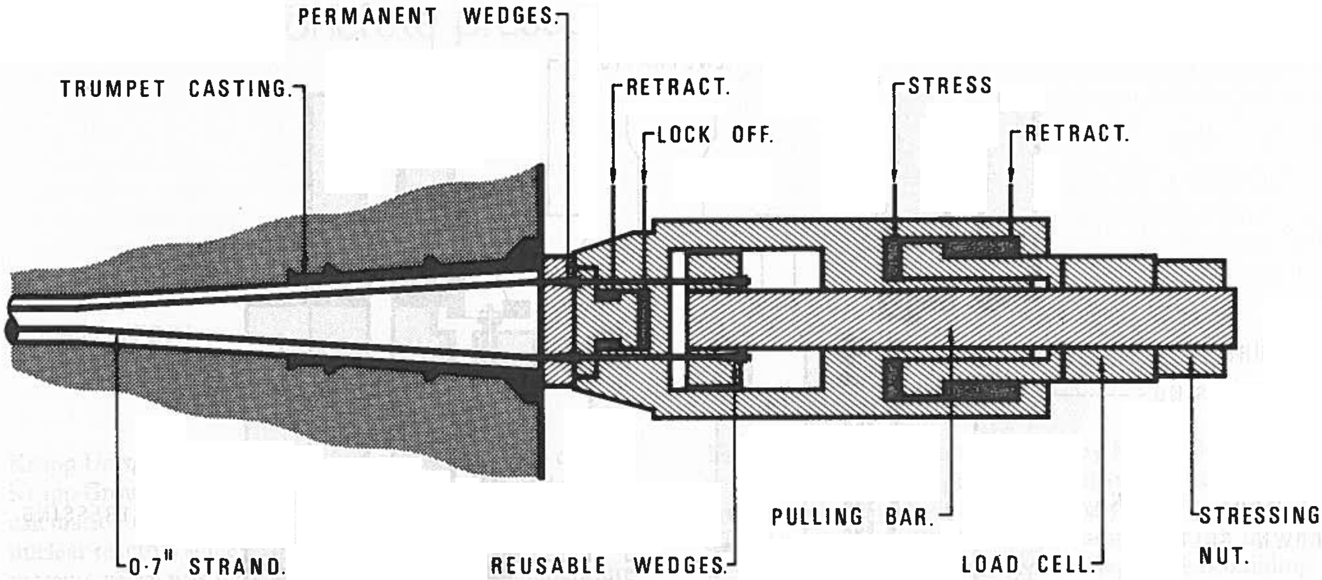

Figure 5. CCL 28/0.7 in. anchorage and stressing jack.

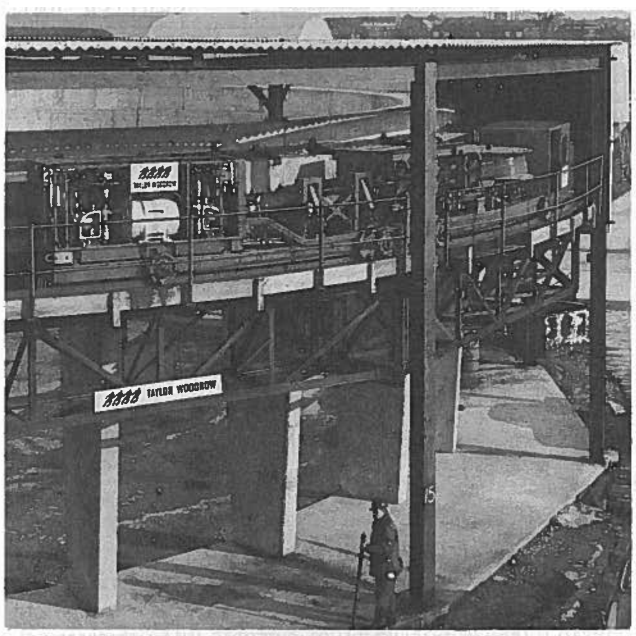

Figure 6. Hartlepool/Heysham PV wire winding system.

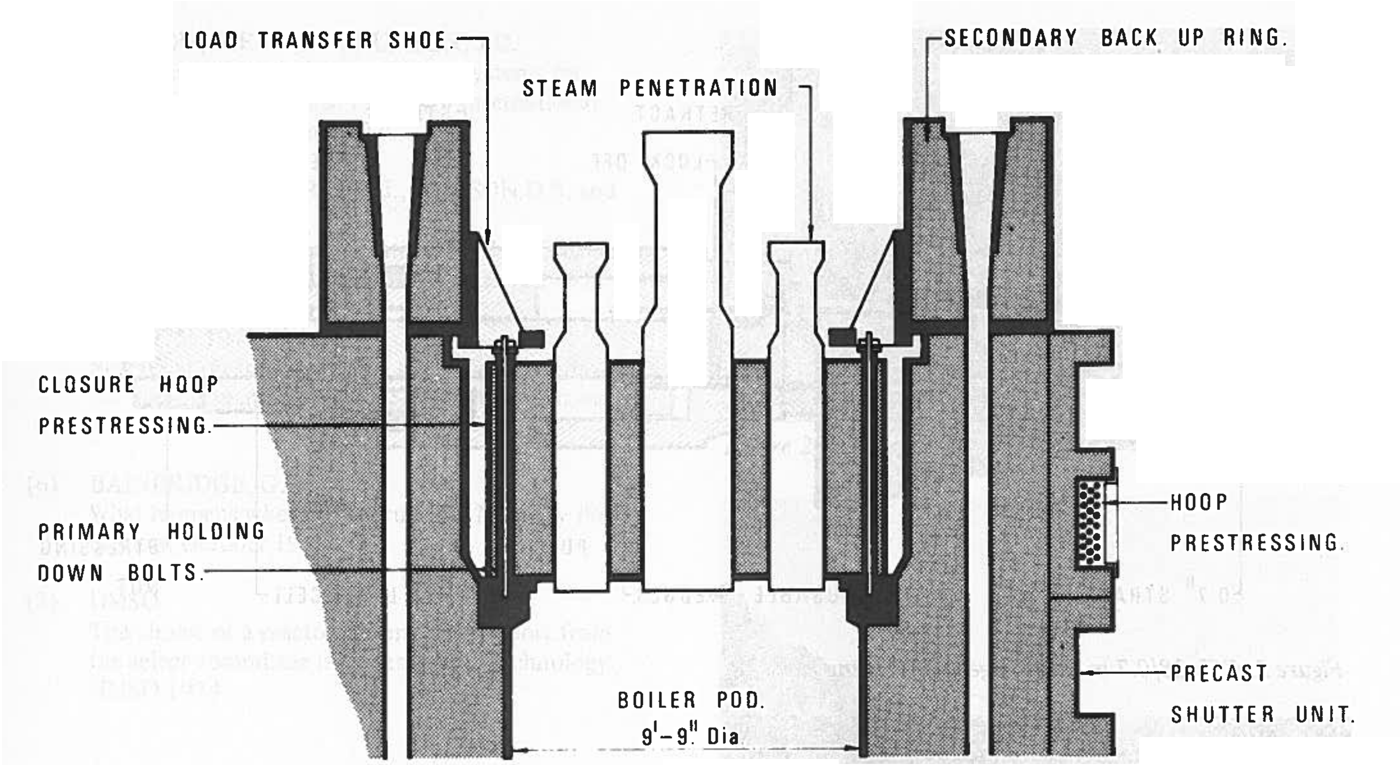

Figure 7. Hartlepool/Heysham PV prestressed concrete boiler closure.

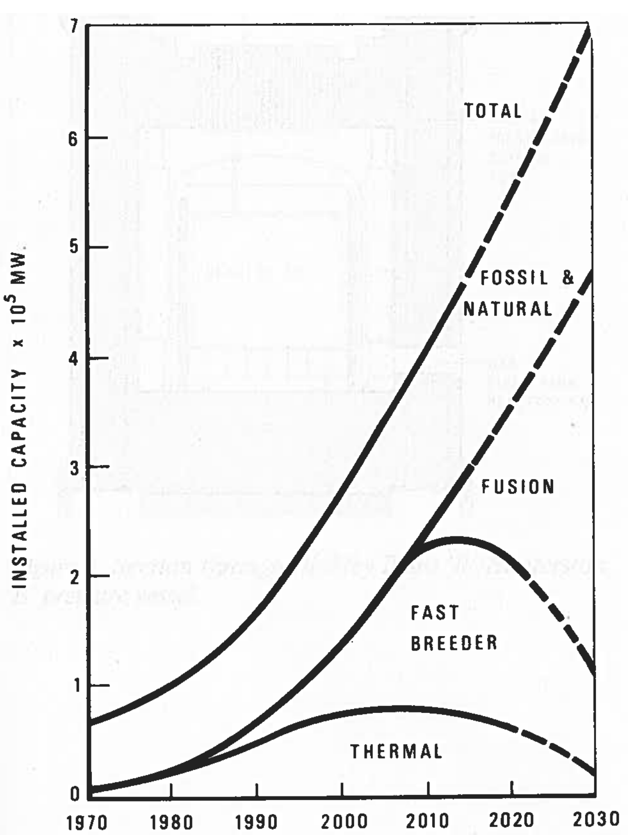

Figure 8. Projection of British installed generating capacity –Dr. G.R. Bainbridge,(6) Figure 3.

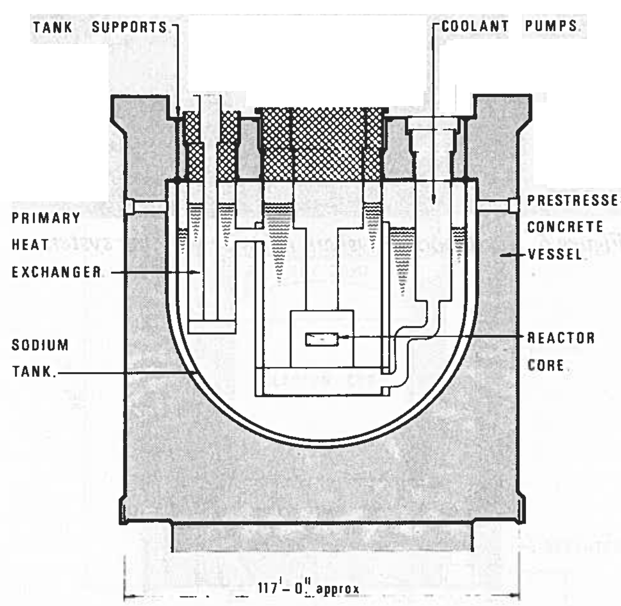

Figure 9. Proposed liquid metal fast breeder reactor vessel.

Prestressed concrete pressure vessels for nuclear reactors

F. Bremer (FDR)

Krupp Universalbau, the Civil Engineering Division of the Krupp Group, has been engaged in the design and design calculation of prestressed concrete pressure vessels for nuclear reactors since 1967. Development of this type of pressure vessel was initially planned for a high-temperature nuclear reactor in connection with a specific requirement. The design concept arrived at was made the subject of an international contest for its execution. Being awarded the contract as main contractor for the operative pressure vessel complete with seals, cooling system, insulation and penetrations would have been in the interest of Krupp and would have fitted into the capabilities of this company. The advantages of such an approach in awarding major contracts are well known and need not be discussed here.

For a number of specific reasons, however, the contract for all deliveries and services involved in the construction of the pressure vessel was awarded to a consortium with Krupp Universalbau acting as lead company in all matters technical and commercial.

The deliveries and services of this consortium include the following:

All Stress and strength analyses for the prestressed concrete pressure vessel and the liner;

Thermal studies in respect of cooling and insulation;

Preparation of all drawings requiring approval and of all working drawings;

Delivery of the prestressed concrete pressure vessel, of the sealing elements, the cooling system, the insulation, and assembly of the penetrations;

Instrumentation and start-up of the pressure vessel.

As a member of the consortium, Krupp Universalbau acts as lead company, is responsible for the overall engineering and for the preparation of the working drawings for the prestressed concrete structure, and has undertaken the general site management and the planning, supply, installation and start-up of the instrumentation of the vessel, and the start-up of the pressure vessel itself.

Structures of this type are very rare in the world. A few similar ones are now in process of construction in France and Great Britain. In Federal Republic of Germany it is the very first of its kind. In the United States, a similar structure of this type was completed in 1973.

The design calculations were made by Krupp Universalbau with the aid of spatial quantum mechanics. The computer programs employed for the stress analysis are company-owned and so is the IBM 370-165 computer used. In Germany, structures of this type are regarded as building structures, and the governmental department of the State North-Rhine-Westphalia responsible for such structures awarded Professor Zerna the job of examining all studies in connection with the mechanical properties, while the TÜV—the German Technical Supervisory Board—was asked to furnish its expert opinion on the metallic components. According to the information available to the public on other pressure vessels, it can be said that this German-built pressure vessel is the first that has ever been examined at considerable expense on the basis of a real three-dimensional stress analysis.

This, however, appeared to be urgently necessary because of the extraordinary geometric disturbances in the upper half of the vessel. From the plan it can be seen that openings with a diameter of about 2 m had to be arranged in vertical planes disposed at angles of only 30° to one another. The considerable geometric disturbance in the centre of the mid-portion of the cover should also be noted. These are required to enable control and shutdown rods for the reactor to be inserted. The performance data of the reactor are:

Electrical capacity	300 megawatts
Reactor coolant:	helium
Hot gas temperature:	+ 770°C
Cold gas temperature:	+ 250°C
Helium pressure:	40 atm

The vessel serves the purpose of protecting the environment from the effects of the thermonuclear process taking place in its interior. The higher safety requirements involve much more exacting demands being placed on design, calculation and construction than those normally imposed on high-grade civil engineering work. The fact that the reactor loads imposed on the pressure vessel and on its sealing elements, such as pressure, temperature and radiation, can be determined much more reliably and do not represent statistical mean values of anticipated loadings as is usual in general civil engineering, has a favourable effect on the safety analysis.

Their supervision and control can be carried out permanently with high accuracy. For this reason it is permissible with structures of this type to allow a higher utilization of all materials. In the case of concrete, the multi-axial mechanical properties were also taken into account.

As aggregate, broken material derived from Devonian massive limestone was used. This limestone exhibits high strength and has an amorphous crystalline structure with low linear thermal expansion, which in the laboratory was found to be $7 \times 10^{-6} \frac{m}{m\ deg.}$ max. For reasons of safety, a value of $7.5 \times 10^{-6} \frac{m}{m\ deg.}$ was substituted in the calculations Compared with the quartzitic concrete made with quartzitic aggregates normally used in western Europe, this means a reduction of the linear thermal expansion of about 30%. The mechanical stresses generated in the pressure vessel by the heat gradient are reduced by the same amount.

The concrete used has a 90-day compressive strength (test cube) of about 700 kp/cm^2. This corresponds to a 90-day strength (standard cylinder) of 500 kp/cm^2. For reasons of safety, a cylinder compressive strength of only 460 kp/cm^2 was adopted.

The figures all apply to the uniaxial compressive strength. Tests for suitability, and the quality assurance checks are much more extensive than with other structures. Re-calculation has shown that the costs for the above checks amounted to DM 30 per m^3 of concrete.

The amount of cement contained in the concrete used was 370 kg/m^3 of the solid concrete. A blast-furnace cement of the HOZ 450 L grade, i.e. low-heat cement with a minimum strength of 450 kg/cm^2 was selected.

An adiabatic test showed a heat of hydration of about 50°C. Deviating from normal European practice, very large concreting steps were used in order to keep the number of joints to a minimum and to comply with the time schedule. In one step up to 900 m^3 of concrete was placed without any interruption. Figure 1 shows a breakdown of the concreting or placement steps. For the cylindrical portion, only complete annular sections up to a height of 2 m were placed. The central portions of the cover received a joint at about half the thickness. The heating rate depended on the height of the sections, provided the cylindrical wall thickness remained unchanged.

Considerable attention was paid to the risk of cracks forming during the cooling of the concrete. Thermodynamic calculations and experiments carried out in advance on a 1 : 1 scale furnished the data for selecting the sections shown here. The built-in instruments give no indication of cracks nor are any cracks visible on the outer surface.

With the reactor in normal operating condition, residual tensile stresses are permissible in the concrete. For this reason the requirement that a concreting joint should have almost the same strength as the undisturbed concrete was justified.

In this respect, optimization was carried out methodically permitting 90% up to 95% of the flexural tensile strength of the undisturbed concrete to be produced in a joint without resorting to any chemical contact agents.

Freedom from cracking of thick-walled structures also requires that temperature differences between the set concrete and that to be placed are not too great. This problem was likewise studied carefully in advance. Damage due to such influences has not been detected. The temperatures during the first stages of concreting were found to agree fairly well with the theoretical values previously calculated.

In extreme cases the need for minimum temperature differences may require cooling of the fresh concrete in winter when the temperature of the set concrete may be at or below the freezing point. Prestressing in a vertical and tangential direction is accomplished by the use of large tendons, their effective breaking load being 920 tonnes. Sigma prestressing steel of grade 145/160 was used. Each bundle included 145 or, for vertical prestressing, 151 wires.

The method of BBVR, Zurich, or its German licencee – Suspa of Langenfeld –, was used for tensioning. DIN 4227, the Standard generally applicable to prestressed structures in Germany, was not used for utilizing the prestressing steel, the reasons being the same as those underlying the higher utilizsation of the strength of the concrete. Utilization amounts permanently to 70% of the cable breaking load. To reduce the risk of corrosion, the bundles were mainly pulled into the tendon ducts only after completion of the entire structure. No corrosion-protective coating was applied during the site construction period because it was found that the as-rolled reinforcing steel would definitely have not rusted during the first ten weeks of construction on site. After thorough studies and much thought, cement mortar injection was selected to provide permanent protection from corrosion. At first there were some doubts as to whether it would be possible to completely fill the voids with cement mortar considering the very compact arrangement of single wires. Tests carried out on a 1 : 1 scale gave a surprisingly good result, however, when the viscosity and flow rate of the injection mortar were at the required values.

Voids were not recognizable and none were detected. For supervision during service, some of the cables are fitted with electric load cells. In such cases, cement mortar cannot be injected, of course. For such cables it is proposed that an absolutely acid-free paraffin be used as a protective coating or an anti-corrosion grease made by the Shell company that has proved highly successful for a number of years in Great Britain.

Since cables of the above size are not available on the German market, a separate approval procedure had to be carried out for this specific case. For this purpose it was necessary to perform a number of cable breaking and friction tests with the original cable.

The cable breaking tests were carried out at Bochum in the rope testing station of the Ruhr coal-mining industry; the friction tests took place at the testing grounds of BBVR at Frick in Switzerland, using the original cable.

For specific reasons, the design had to be frozen in 1970 with respect to its geometry and configuration. Today, for such a vessel, large tendons would certainly not be used for producing the horizontal prestress, but preference given to a winding procedure. In 1970 no such procedure was available on the world market which gave the requisite tensioning force. Today, there are British, German, Swiss and American procedures suitable for producing such prestress by winding. This method of winding offers substantial advantages over the one used in this present structure. While they are mainly of an economic nature, they also benefit the design because they eliminate the dense arrangement of prestressing cables employed in the case described here. Figure 2 shows a future vessel of this kind.

Un-tensioned reinforcement constitutes a special engineering problem. Under German specifications any tensile stresses set up in the concrete must be fully absorbed by un-tensioned reinforcing bars. The only easing of this rule is that for certain forms of constraints the yield point may be used as the limiting stress for the steel. However, in all cases where stress components are due to equilibrium conditions, the maximum allowable steel stress must be used which incorporates a safety factory of 1.75 against tensile failure of the steel. There are sound reasons for re-considering the need for this requirement without this affecting in any way the justified demand for more safety.

For instance, in all cases, where a certain tensile stress level can definitely not be exceeded, one possible approach would be to do without any reinforcement to absorb tensile stresses, and to permit partial prestressing for tensile stress fields exceeding this level. The latter method is already common practice in some countries.

On average, this structure embodies the following specific reinforcements:

Prestressed reinforcement 155 kg/m^3 of concrete

Un-tensioned reinforcement 115 kg/m^3 of concrete.

The maximum amount of un-tensioned reinforcement is as much as 190 kg/m^3 of concrete in certain local areas. Such a construction is no doubt undesirable. For the un-tensioned reinforcement, ribbed bars of self-hardening steel with a minimum yield point of 4200 kg/cm^2 and a minimum tensile strength of 500 kg/cm^2 was used. The maximum allowable stress of this steel was 2400 kg/cm^2 for equilibrium forces.

The stress and service life analyses were handled with the same priority as the adequacy studies carried out on the pressure vessel. For reasons of stability, the bond between the liner and prestressed concrete pressure vessel is capable of resisting shear and tensile forces. However, tests have shown that the shear-resistant bond begins to yield in the course of time, depending on the concrete temperature and load cycles, as a result of reactor pressure variations and the starting and shutdown procedures. Therefore, it is not advisable in each case to rely on participation of the liner in the load-bearing behaviour of the concrete under normal reactor operating conditions.

The expenditure involved in testing is roughly equal to that involved in the prestressed concrete and amounts to about 15% of the purchase cost of the liner. It is of interest to note that the thermal insulation in the present case consisted of several layers of austenitic metal foils. With an insulation thickness of about 70 mm, the temperature gradient was around 200 deg. C.

Protection from heat is provided by the insulation and the cooling system. For reason of better heat transfer, the cooling system is welded to the liner. It consists of pipes with an inside diameter of 27 mm spaced on average about 100 to 150 mm apart.

Under operating conditions the mean surface temperature on the inside surface of the concrete is about + 42°C. On the external surface it is roughly kept constant at about + 22°C. There is thus a temperature gradient of 20 deg. C across the wall thickness.

At first sight this appears to be very small. It should be noted, however, that this gradient is responsible for about 35% of the prestressing steel that goes into the structure. This adds up to no less than 550 tonnes of the total tonnage of prestressing steel.

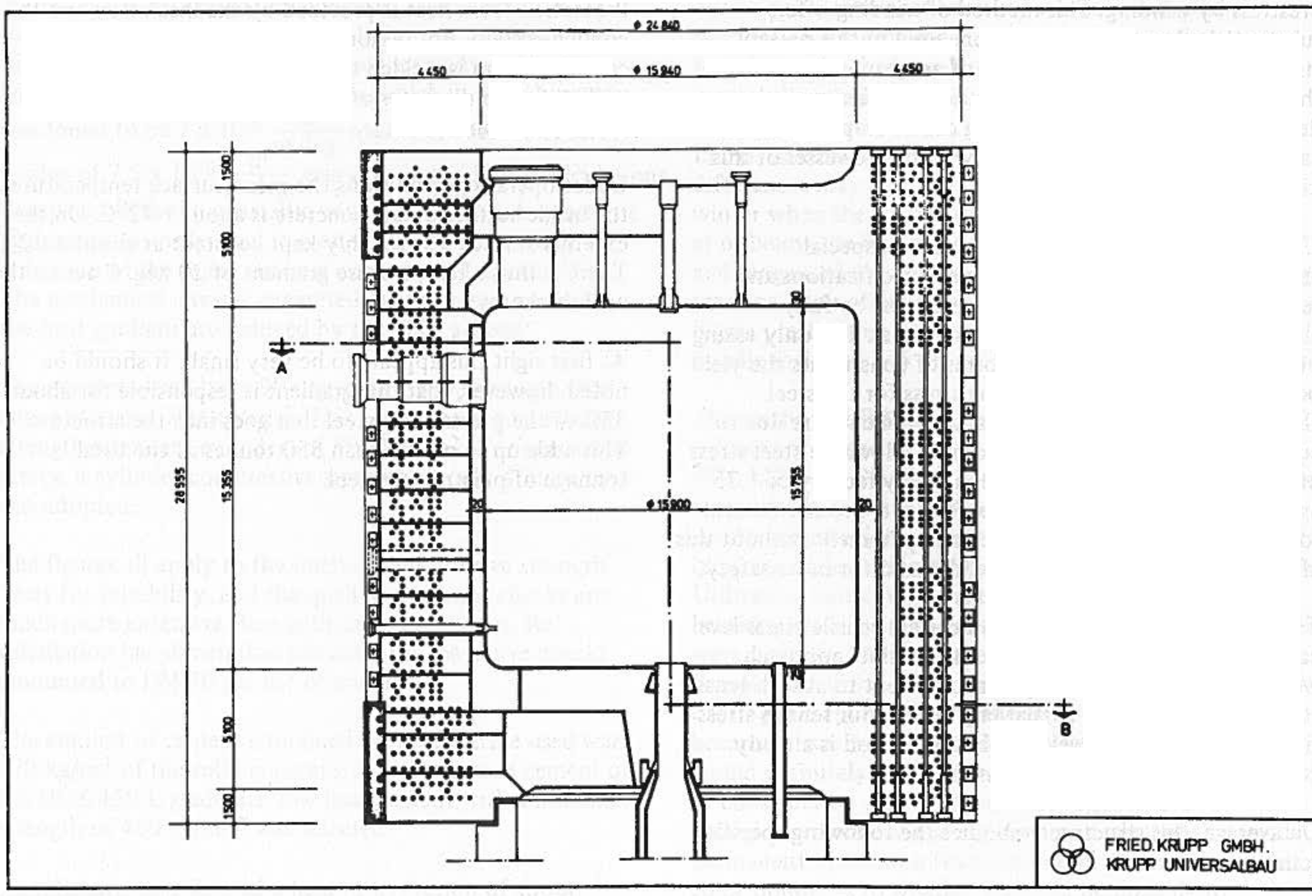

Figure 1.

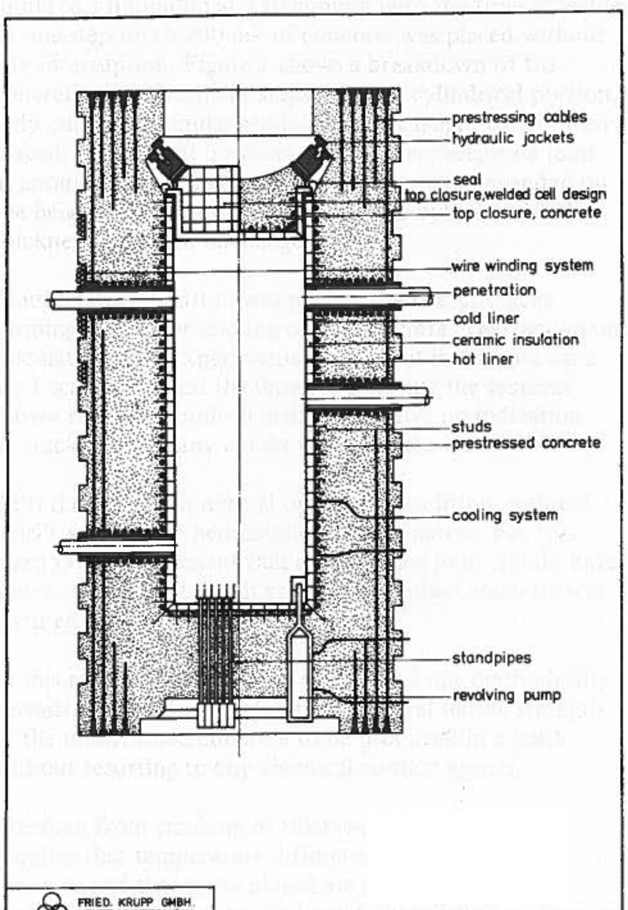

Figure 2.

Design of the prestressed concrete containment for the Bellefonte Nuclear Plant

H.L. Perry and E. Burdette (USA)

INTRODUCTION

In the past three to four years there has been a significant trend toward much larger unit generating capacities (1100 to 1300 MWe) for nuclear power plants in the United States. This trend has been caused by economic advantages of larger units, environmental (air pollution) problems associated with equal size coal-fired units, and a rapid increase in demand for electrical power for homes, industry, and businesses.

The US Atomic Energy Commission has established safety guidelines for the design of nuclear power plants to assure that the public is protected from the hazards associated with nuclear plants. As a result, prestressed concrete containments with thin steel liners (¼ in. to ½ in.) have evolved as one of the best methods for containing postulated accidental releases of high energy radioactive steam from the reactor coolant system (reactor, steam generators, piping, and equipment) for a pressurized water reactor (PWR) nuclear plant.

Physical sizes of the major mechanical equipment (reactor, steam generators, pressurizer, pumps, etc.) have increased with increasing rated generating capacities of nuclear plants. As a result, larger energy releases are postulated to occur during accident conditions. This postulated increased energy, released in the forms of radioactive steam, which pressurizes the containment, subsequently requires larger containments in order to optimize the design at an internal pressure between 50 and 60 psig.

BELLEFONTE CONTAINMENT

TVA's Bellefonte Nuclear Plant, which is a two-unit PWR power plant, is utilizing two fully prestressed concrete containment vessels (PCCV) which are identical for design purposes. These structures provide containment for two 1300 MWe (megawatts electrical) reactors.

Three unique features of the Bellefonte prestressed containment, which classify it as a 'prototype' according to AEC Safety Guide 1.18, serve to illustrate that it represents the latest thinking in United States containment design practice.

They are:

(1) prestressing tendons with an ultimate capacity of greater than 500 tons;

(2) only four buttresses are used for hoop prestressing;

(3) prestressed grouted rock anchor tendons are used to resist uplift forces instead of a thick concrete foundation mat.

The containment is a cylindrical concrete structure, capped with an elliptical domed roof, and has a foundation ring anchored with prestressed grouted rock anchors into a massive-bedded, tightly-jointed limestone rock foundation. A construction joint separates the containment foundation ring and the interior concrete base slab. (See Figures 1 and 2 for general configuration and dimensions of prestressed containment and adjacent structures).

The entire containment structure is lined on the inside face with a ¼ in. thick welded steel liner plate to provide leak tightness from radiation inside the containment. The liner is attached to the concrete cylinder and dome by means of an angle and channel grid system welded to the liner plate and embedded in the concrete. (See Figure 1).

Prestressing system

The cylindrical portion of the containment is prestressed by an unbonded post-tensioning system composed of hoop and vertical tendons. There are four vertical buttresses spaced 90° apart around the containment. Hoop tendons are anchored at buttresses 180° apart, bypassing the intermediate buttress. In plan the hoop tendons are spaced near the outside face. This facilitates obtaining a more uniform and efficient hoop prestress.

Vertical tendons are located in the centre of the wall and are spaced uniformly along the circumferential direction. Vertical tendons are coupled directly to rock anchor tendons of the same capacity by using a coupling device which is located in the tendon access gallery. (See Figures 3 and 4). TVA decided to use post-tensioned rock anchor tendons instead of the conventional heavily reinforced concrete foundation mat for three reasons:

(1) dense, sound foundation bedrock at the Bellefonte site is readily adaptable to using rock anchors;

(2) the successful development of large post-tensioned rock anchors over the past several years;

(3) cost estimates indicated that a heavily reinforced roundation mat would have cost approximately $¼ million more for each containment structure.

Dome tendons are arranged in three layers with the layers oriented at 120 degrees to each other. The tendons in each group are placed in parallel vertical planes across the dome.

All tendon assemblies for the containment and rock anchor tendons are buttonheaded wire tendons (minimum of 170-¼ in. diameter wires in each tendon) capable of developing a minimum ultimate strength of 1000 kips each. All tendons have the same cross-sectional area and the same specified minimum ultimate strength capacities.

DESIGN LOADS AND ANALYSIS APPROACH

The basic approach to the design of the containment follows the guidelines established by ACI/ASME Committee 359 and published in a code entitled Standard Code for Concrete Reactor Vessels and Containments (hereinafter referred to as ACI (359) Code). This code has been issued for trial use and comment and is due to be published in final form in mid-1974. The United States Atomic Energy Commission (AEC) is adopting this Code as the standard design criteria for concrete containments and reactor vessels as part of an all out effort to standardize nuclear power plant design.

The Bellefonte containment is designed for the load combinations listed in Table 1 at the end of this paper (a brief explanation of each load symbol is given in the Table). It should be noted that, in the load combinations listed in the Table, tornado wind loads only effect the design of the foundation ring of the primary containment because the secondary containment (Figures 1 and 2) is designed to take tornado wind loads and tornado generated missile loads. (The secondary containment at Bellefonte is required only for environmental reasons of preventing any radiological leakage to the environment because of anticipated periods of minimal air circulation at the site).

Referring to Table 1, two types of load combinations are used in the design, service loads and factored loads.

Service load conditions are any conditions encountered during construction and in the normal operation of a nuclear power plant. Included in these are the anticipated transients and test conditions during normal and emergency startup and shutdown of the nuclear steam supply, safety, and auxiliary systems.

Concrete stresses permitted under service load conditions are based upon elastic analysis and straight–line theory of stress and strain in flexure.

Factored load conditions are those which result from a postulated single rupture in a pipe of the reactor coolant system as well as extreme environmental conditions (earthquake and tornado wind) postulated to occur during the life of the facility. Also included are combinations of single failure of the reactor coolant system plus extreme environmental conditions which are considered credible.

Concrete stresses permitted under factored load conditions are also based upon elastic, straight line theory in which the reinforcing steel stress is permitted to go to yield, but no yielding is allowed as in the ultimate strength design approach of the present ACI Building Code.

It should be emphasized at this point that the containment is designed to behave under *elastic* conditions so that the behavior of the containment is completely predictable for all loading conditions.

PRESTRESSING DESIGN AND CRITICAL LOAD COMBINATIONS

Referring again to the load combinations in Table 1, load cases (7) and (9) control the design of vertical prestressing tendons and rock anchor tendons. The critical applied loads are P_a, T_a, E or E^1 with their appropriate load factors. These loads are applied to the entire containment while loads R_a, Y_j, Y_m, and Y_r are concentrated loads applied at isolated locations and do not significantly affect the overall containment design. Therefore, the loads that are used to design the vertical prestressing (F) are the critical applied loads aforementioned (excluding T_a) and the principal resisting loads D and L. Load case (6) in Table 1 controls the design of hoop and dome prestressing tendons. Again, the critical applied load P_a and resisting loads D and L are equated with their appropriate load factors to the final prestressing force (F).

It should be briefly mentioned here that even though forces from restrained thermal expansion of the ¼ in. steel liner during accident load conditions were considered in the containment design, this force was not included in determining the prestressing force. The reason for this is that the transient temperature gradient through the liner and concrete (T_a) is a self-relieving type of load. As the temperature (under accident conditions) of the ¼ in. steel liner on the inside face of the containment increases to a maximum of approximately 240°F, the concrete on the outside face will crack (tensile stresses in the outside face concrete will exceed $6\sqrt{f'c}$ for this condition). Cracking of the concrete will progress toward the liner. At the same time, the high compressive strain in the liner will be reduced significantly until the compressive force in the liner is balanced by the tensile force in the outside face reinforcing steel. (The stress in the prestressing tendons will only increase 2% for this condition since the prestressing steel is unbonded and will strain over the full length of the tendon.) Therefore, by excluding this load in the design of the prestressing system, many tons of prestressing steel were saved.

Before determining the final prestressing forces (F) for the Bellefonte containment, tendon stress losses were calculated. These losses consist of friction, creep of concrete, tendon relaxation, shrinkage, and elastic shortening of the concrete.

Jacking forces on each tendon will be 80% of ultimate ($0.8\ f_{pu}$). The tendons will be anchored at $0.75\ f_{pu}$, and the initial stress in the tendons after stressing all tendons will not exceed $0.70\ f_{pu}$. (These allowables are in conformance with the ACI (359) Code). The final stress in the tendons after all losses have occured is between 0.55 and $0.60\ f_{pu}$.

Loads exerted on the containment from the prestressing tendons are considered as follows:

(1) Loads from vertical tendons are considered to act at the top of the containment ring girder.

(2) Loads from horizontal (hoop) tendons are considered to exert inward pressure on the cylinder walls.

(3) Loads from dome tendons are considered as pressures exerted normal to and tangential to the surface of the dome.

As indicated in (2), hoop prestressing is designed to exert a uniform inward pressure in order to provide pre-compression for the postulated accident pressure on the inside of the containment. However, a uniform hoop prestress of 1.5 P_a (load case 6) from the base of the cylinder to the spring line of the dome produced exceedingly large discontinuity moments at the base of the cylinder which would have required large amounts of reinforcing steel and/or a haunched concrete section at the base. The solution to this problem was to vary the hoop prestress near the base of the containment using a linear variation of prestress from zero prestress at the base to full hoop prestress 20 ft above the base. By using this method, the 3 ft-6 in. cylinder wall thickness was maintained all the way to the base of the containment, thereby saving considerable concrete and special formwork. Also, substantial savings in prestressing steel and reinforcing steel required for the discontinuity moment at the base are affected.

Sequence of prestressing is very important in design and in stressing operations for the containment. For the Bellefonte containment no significant amount of dome and hoop prestressing will be allowed until most of the vertical prestressing is accomplished, since vertical stressing is effective in resisting discontinuity moments in the cylinder. Also, throughout stressing operations, stressing positions will be alternated in order to prevent concentration of multiple stressed tendons adjacent to multiple unstressed tendons.

REINFORCEMENT AND CONCRETE REQUIREMENTS

Mild steel reinforcement is primarily required in minimum amounts in the outside face of the wall and dome for crack control, and on the inside face to take discontinuity moment tensile stresses.

Allowable concrete, prestressing tendon, and reinforcing steel stresses are in conformance with the ACI (359) Code except for minor exceptions. The allowable membrane compression stress in the concrete of the cylinder and dome for service load conditions is taken as 0.30 f_c in order to limit creep deformations. This allowable membrane is used in the containment design to size the thicknesses of the wall and dome under loads exerted from initial prestress (Case 2c in Table 1). The minimum strength of concrete for the Belletonte containment will be as follows:

Foundation Ring–4000 psi (*90 day strength)
Dome and Cylinder–5500 psi (*90 day strength).

DESIGN OF PRESTRESSED ROCK ANCHOR TENDONS

General design conditions

The stability of the containment structures under the action of overturning moments generated by earthquake loadings is assured through the use of prestressed grouted rock anchor tendons. Rock anchorage is accomplished by aligning and grouting into rock, tendons of the same size as the vertical tendons in the cylinder wall. Grouting is accomplished in two stages with sufficient length grout columns in each stage to develop the entire prestressing force. The length of the first stage grout column was established by full scale tests at the Bellefonte site. However, each tendon installation for the containment becomes, in essence, a proof test for the adequacy of the first-stage grout column, since the entire prestress force is applied prior to second-stage grouting. (See Figure 3).

Under normal operating conditions of the nuclear plant, the prestressed anchorage maintains a compressive force in the rock from the base of the containment walls to the bottom of the anchorage. A net upward force can occur only when an earthquake occurs simultaneously with accident pressure. (Load combinations 7 and 9 in Table 1).

Procedure for determining depth of rock anchors

Uplift loads from accident pressure plus earthquake (½ SSE) are resisted by downward loads such as dead load, prestress, soil, and such like acting on a 90° wedge extending from mid-depth of the first-stage grout column to the top of the foundation rock or concrete. Also included in these resisting loads is the foundation rock within the wedge. (The 90° wedge that resists uplift generates a certain circumferential length under the containment cylinder). (See Figure 3). For the accident pressure plus SSE (Case 9 in Table 1), the wedge is assumed to extend to the base of the anchorage.

The weight of rock within a wedge is assumed as the buoyant weight. The circumferential length of rock wedge, used to calculate resistance to uplift is the rock engaged by those tendons under which compression between the base and rock no longer exists as a result of the combined earthquake overturning and accident loads. For the Bellefonte containment this circumferential wedge subtended an angle of approximately 145° from the centre of the containment.

OTHER IMPORTANT DESIGN CONSIDERATIONS

Tangential shear

The horizontal forces resulting from earthquake loads produce tangential shear stresses in the plane of the shell.

There has been considerable controversy over the past few years as to the method of design to use for tangential shear forces. AEC has currently accepted the shear wall approach in Section 11.16 of the ACI-318-71 Code, for tangential shear design for prestressed concrete containments only.

*TVA concrete mixes contain fly-ash admixture which imparts desirable properties such as continued strength gain with age, increased workability, and lower permeability and heat of hydration.

Penetrations

Large penetrations such as the 22 ft diameter equipment hatch and the 10 ft diameter personnel hatches for the Bellefonte containment require special design considerations. Tendons are deflected out of plane; concrete is thickened around the penetration; additional reinforcing steel is required; and special design analyses are required using finite element computer programs.

Computer programs

The principal analysis of the containment structure was performed using the GENSH5 program developed by the Franklin Institute Research Laboratories (FIRL). This is basically a layered, thin shell of revolution analysis which has the capability to analyse the entire containment structure for axisymmetric loads (cylinder wall, ring girder, and dome) as a single model. Non-axisymmetric loads may be considered through the use of Fourier series.

A finite element axisymmetric program, AMGO32, was used to analyze the dome-ring girder-wall intersection to check the results of the GENSH5 program in this region. The AMGO32 program has the capability of analyzing a thick or thin shell of revolution.

Another finite element program, AMGO33, which is a plane stress, plane strain program, was used for analyzing the buttresses in the containment. These finite element programs were also utilized to analyze the effect of the prestressed rock anchorage on the foundation rock.

Large openings such as the equipment and personnel hatches were analyzed by the Shallow Shell capabilities of the ICES-STRUDL-11 finite element computer program.

Thermal stresses in the containment cylinder wall and dome were calculated by GENSH5 and checked using a program developed by TVA (THERMCYL).

SUMMARY AND RECOMMENDATIONS

The Bellefonte containment utilizes one of the largest available prestressing tendons presently in use. It is also one of the largest containments, 273 ft high and 142 ft diameter (outside dimensions). The large size of the structure was necessitated by the more stringent present day design philosophies of postulated accident and earthquake loadings, as well as the economic need for larger generating units. The use of prestressed rock anchor tendons, an elliptical domed roof, and variable hoop prestressing at the base of the containment cylinder wall effected substantial cost savings for the Bellefonte containment. Further cost savings were realized by eliminating, through careful analytical investigation, the thermal stresses in the liner from the final design prestress forces. (This method reduced prestressing forces by approximately 100 kips/ft).

Efficient computer programs such as the GENSH5 program, which will analyze the containment with one model, are a necessary tool for safety related structures like the Bellefonte containment.

In conclusion, looking at the present of containment design, there is a definite need for more consistent design criteria for containments, particularly in the areas of tangential shear design, impact effects of tornado-generated missiles, and thermal effects on liner and concrete. Testing programmes should be instituted in these areas. Some of these programmes are coming into reality at this time; however, the entire industry needs to support these programmes with funding, research personnel, and test facilities to make the programme successful. Another example of an area that needs further investigation is the use of grouted tendons in the containment instead of greased tendons. Inclusion of grouted tendons as a design alternate should give more competitive bids in the future as soon as a surveillance programme is developed for grouted tendons that is acceptable to the AEC and is economically acceptable to the prestressed containment industry.

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Proposed standard Code for concrete reactor vessels and containments. Copyright 1973.

APPENDIX

Table 1. Loading combinations

A. *Service Loads*

(1) $D + L + F + P_t + T_t$
(2a) $D + L$
(2b) $D + L + W$
(2c) $D + L + F_o$
(3) $D + L + F + P_v + R_o + T_o + E$ or $*W$

B. *Factored Loads*

*(4) $D + L + F + T_o + P_v + R_o + W_t$
(5) $D + L + F + T_o + P_v + R_o + E^1$
(6) $D + L + F + 1.5P_a + T_a + R_a$
(7) $D + L + F + 1.25P_a + T_a + 1.25E + R_a + Y_j + Y_m + Y_r$
*(8) $D + L + F + 1.25P_a + T_a + 1.25W + R_a + Y_j + Y_m + Y_r$
(9) $D + L + F + P_a + T_a + E^1 + R_a + Y_j + Y_m + Y_r$

In combinations 6, 7, 8, and 9, the maximum values of P_a, T_a, R_a, Y_j, Y_r, and Y_m, including an appropriate dynamic factor, shall be used unless a time-history analysis is performed to justify otherwise.

For combinations 7 and 8, local stresses due to the concentrated loads Y_r, Y_j, and Y_m, may exceed the allowables provided there will be no loss of function of any safety-related system.

Where L reduces the effects of other loads it shall be taken as the minimum possible value.

Load definitions

D = Dead load
L = Appropriate live load, including construction loads, hydrostatic loads, and *lateral earth pressures
F = Final prestressing force (after losses)
F_o = Prestressing force at transfer (initial prestress)
P_v = Vacuum pressure load due to inadvertant spray operation (P_v = 4 psig)
R_o = Pipe reactions during normal operating conditions
T_o = Thermal effects and loads during normal operating or shutdown conditions
P_t = Loads due to test pressure ($P_t = 1.15$ P)
T_t = Thermal loads at time of test
P_a = Design accident pressure load (P_a = 50 psig)
R_a = Pipe reactions under thermal conditions generated by the postulated pipe break and including R_o
Y_r = Equivalent static load on the structure generated by the reaction of the broken high-energy pipe during the postulated break, including an appropriate dynamic factor
Y_j = Jet impingement equivalent static load on a structure generated by the postulated break including an appropriate dynamic factor
Y_m = Missile impact equivalent static load on a structure generated by or during the postulated break, like pipe whipping, including an appropriate dynamic factor
T_a = Thermal effects and loads generated by the postulated pipe rupture including T_o
E^1 = Loads due to safe shutdown earthquake (SSE)
E = Loads due to earthquake equivalent to one-half safe shutdown earthquake (½ SSE)
W = Wind load
†W_t = Tornado load

*These loads or combinations apply only to the secondary containment wall and are to be used in the analysis of the foundation ring only.

†These loads are applied to secondary containment wall; therefore, they are used only in the analysis of the foundation ring.

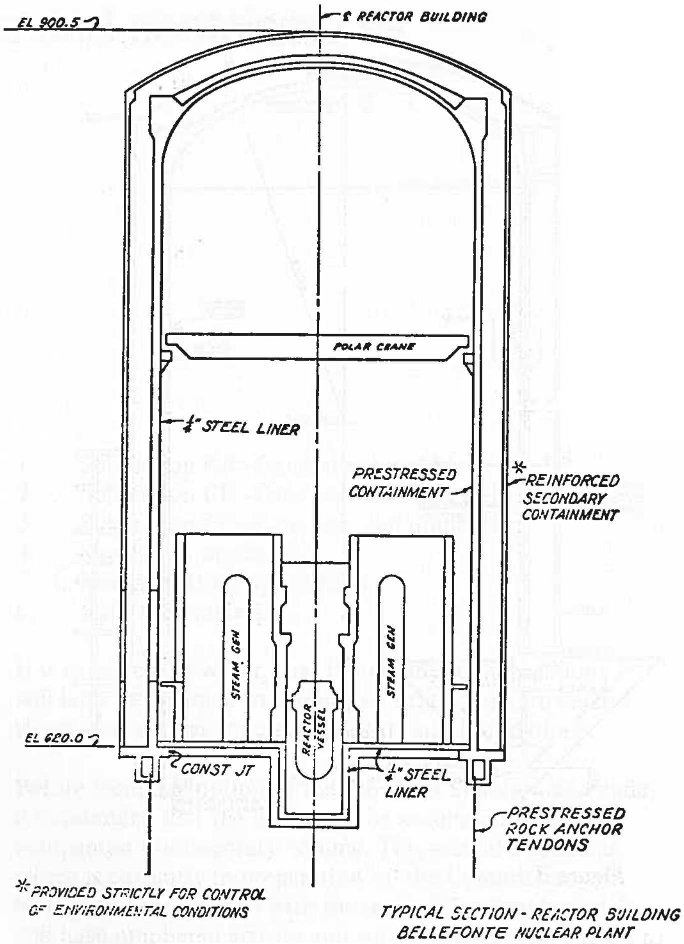


Figure 1.

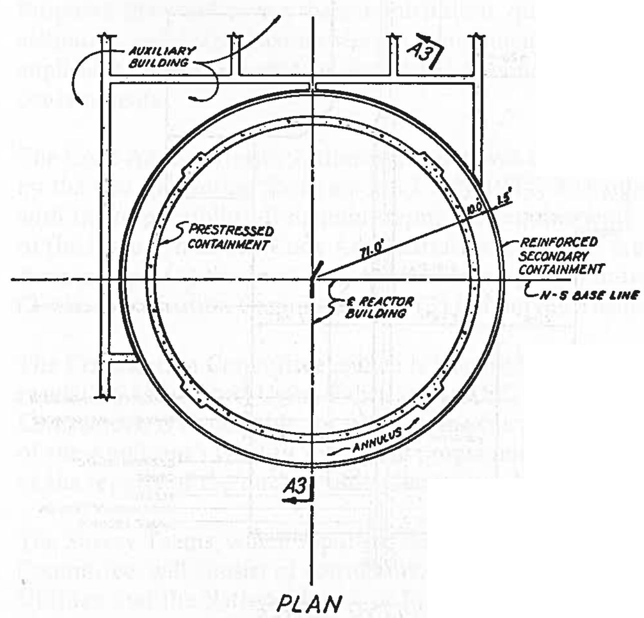


Figure 2.

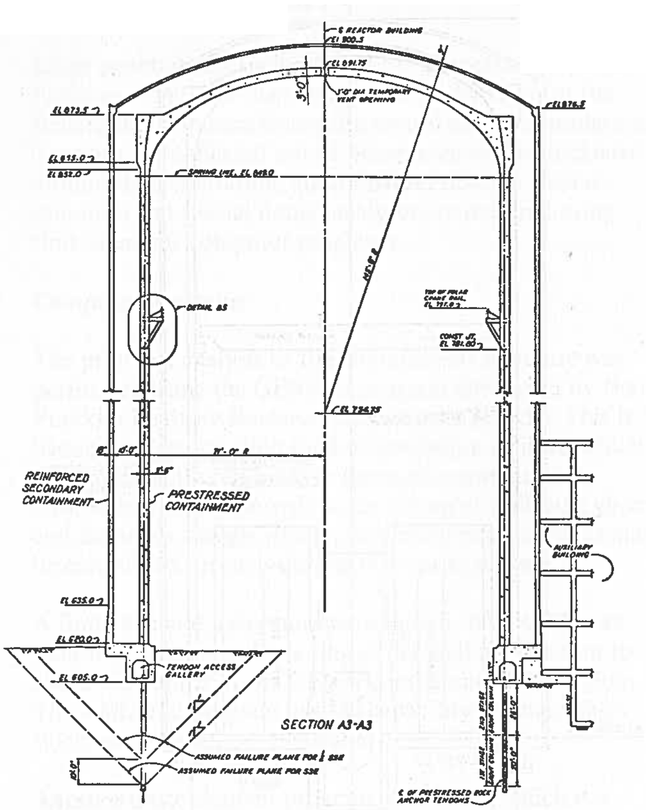
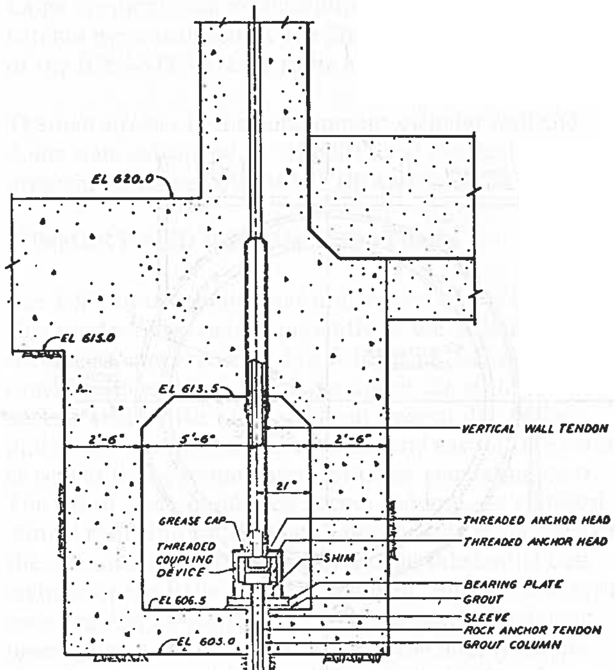


Figure 3.



TENDON COUPLING DEVICE

Figure 4.

Code for concrete reactor vessels and containments

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INTRODUCTION

A Proposed Standard has been prepared by the Joint ACI-ASME Technical Committee on Concrete Pressure Components for Nuclear Service (ACI-ASME Committee 359). The Standard constitutes the requirements for the design, construction, and use of concrete reactor vessels and concrete containment structures for nuclear power plants. Publication of the Standard was approved in April 1973 by ACI and ASME on a Trial Use and Comment basis for a period expected to last about one year, after which there will be public hearings leading to its formal adoption as a code in 1974. The document will form Division 2 of ASME Boiler and Pressure Vessel Code, Section III.

BACKGROUND

The basic materials for the Proposed Standard were drawn from documents prepared by two predecessor committees: ACI Committee 349, Criteria for Nuclear Containment Vessels, and ASME Boiler and Pressure Vessel Code Committee, Section III, Division 2. In September 1971 a joint committee, whose membership includes individuals from both ACI and ASME and many others actively involved in the field, undertook the task of creating a single standard from the two documents. At the same time, they incorporated additional technical material and provisions for implementing administrative agreements reached by the two Societies.

The Joint Committee has three primary goals: (1) To establish rules in the form of a code for the design, construction, inspection, and testing of composite concrete and steel components for reactor vessels and containments for nuclear power reactors; (2) to interpret these rules when questions arise regarding their intent; and (3) to periodically update code provisions making full use of the expedited procedure for revision of standards as necessary. In order to carry out these goals, provision has been made for the regular publication of code addenda and responses to code cases.

For the convenience of those following closely the design and construction of nuclear power plants and their components, the Proposed Standard has been organized similarly to that of ASME Boiler and Pressure Vessel Code, Section III, Division 1, Nuclear Power Plant Metal Components. It is at present divided into three main subsections, two groups of Appendixes, and a section including convenient materials, as follows:

1. Subsection CA–General requirements
2. Subsection CB–Concrete reactor vessels
3. Subsection CC–Concrete containments
4. Mandatory appendixes
5. Nonmandatory appendixes
6. Reference materials.

It is expected, however, that the number of subsections will later be enlarged to encompass other concrete reactor structures, such as concrete supports and foundations.

Before formal adoption of the Proposed Standard as a code, it is intended that the document be accompanied by a companion Commentary volume. This second document, which is currently in preparation by the Committee, will be organized in parallel with the text of the Standard and will have numbered articles and sub-articles corresponding to it where necessary.

ADMINISTRATION

The Subsection CA (General Requirements) portion of the Proposed Standard covers the administration, quality assurance, and authorized inspection requirements applicable to concrete reactor vessels and to concrete containments.

The Code Administrative Authority, which was developed by the two sponsoring Societies on 11 July 1971, is charged with the responsibility for administering the requirements of the Code. Under the Code Administrative Authority are three groups: (1) the Joint ACI-ASME Technical Committee, (2) the Certification Committee, and (3) the Survey Teams.

The Certification Committee, which is balanced among regulatory Inspectors, Users, Fabricators, AEC, and Contractors, is responsible for determining the adequacy of the Applicant's Quality Assurance programme by review of the reports of the nuclear survey team.

The Survey Teams, which report to the Certification Committee, will consist of consultants from ACI, ASME, Utilities, and the National Board of Boiler and Pressure Vessel Inspectors. The makeup of the Survey Teams will be based on establishing a balance of competence in the areas requiring evaluation by the Code, including quality assurance techniques, construction methods, welded fabrication, materials control, field change control, and acceptance testing.

CONTENT OF CODE

General requirements

This Code, as set forth in Subsection CA, presents the rules governing the composite metal and concrete assemblies including their material constituents, parts, and appurtenances that collectively constitute concrete reactor vessels and concrete containment structures (Figure 1) that function as an integral part of the gas or liquid pressure boundaries in nuclear power plant primary coolant systems and containment structures. General definitions are given, together with the responsibilities and duties of the parties participating in component construction. Document preparation, distribution, review, approval, and filing of the various design and construction documents are discussed.

Subsection CA also describes the requirements for a quality assurance programme, the performance of inspection and qualifications of inspection personnel, and rules for marking, stamping, and reports. The quality assurance programme includes the planning, managing, and conducting of quality assurance programmes for controlling the quality of work performed under this Code, and rules governing the evaluation of such programmes prior to the issuance of certificates of authorization for the construction and installation of concrete components.

Concrete reactor vessel and containments

Subsections CB and CC cover, respectively, the requirements for prestressed concrete reactor vessels and prestressed and/or reinforced concrete containment structures. Each of these two subsections has its own material, design, construction-fabrication, examination, and testing sections, and is organized for convenience similarly to the ASME Section III, Division 1, Nuclear Power Plant Metal Components. In some instances, this has meant almost complete repetition of requirements between Subsections CB and CC in such areas as materials and examination. In effect, the Subsections on reactor vessels and secondary containments are thus written to stand by themselves. Although this decision added considerably to the length of the document, this organization was felt to be useful until the detailed requirements of reactor vessels and containments became better established.

The components covered in these subsections include (1) structural concrete pressure-resisting shells and shell components; (2) shell metallic liners; and (3) penetration liners extending the liner through the surrounding shell concrete.

The Subsections CB and CC, materials articles of the Code, set forth requirements for concrete and concrete materials, materials for reinforcing and prestressing systems and for vessel liners, welding procedures, and quality assurance programmes as applied specifically to materials and materials manufacturers.

Design

The design requirements of the Code for reactor vessels and for containments are different owing to the different nature of the service conditions experienced by these structures. However, the areas covered for each type of vessel are similar and include general design conditions, load criteria, analytical design criteria and serviceability and strength limits, and structural concrete design details. For the metallic liner, design analysis procedures and design allowables, along with liner design details, are prescribed.

For concrete reactor vessels it is required that the structure be designed for the service conditions stipulated in the design specifications by maintaining concrete shell levels of stress, strain, and deformation to limits that ensure an essentially elastic response under normal service life of the vessel. The CRV must also be designed to have a structural capacity that is greater, by the margins spelt out in these design requirements, than that required to resist the load level associated with postulated extreme occurrences.

As an additional requirement, the vessel is to be so proportioned, reinforced, and prestressed that as its structural capacity is approached, vessel response will be gradual, observable, and predictable and in a manner such that the minimum ultimate load capacity is clearly developed. This requirement is intended to provide a high degree of assurance that vessel material deterioration, should it occur, would not result in a hazard for the installation. In the implementation of this criterion, the designer must investigate possible modes of failure, identify those that do not meet the above criterion, and establish by analysis or model study that the design of the vessel is such that failure in a non-conforming mode will occur at a pressure greater than the failure pressure for the lowest conforming mode.

For concrete containments, the design criteria apply to structures having steel reinforcement, prestressed tendons, or any combinations thereof, and metallic liners.

The metallic liner is to be designed for functionality by maintaining levels of stress, strain, and deformation to limits that can be related to proven levels of high leak-tight integrity.

The primary criteria for the containment, as demonstrated by the design calculations, shall consider factored as well as service-load conditions. For factored load condition, the following requirements are to be met: (1) The summation of external and internal forces and moments satisfy the laws of equilibrium and will not bring the section to a general yield state, and (2) tensile yielding in the reinforcement is acceptable when thermal gradient temperature effects are combined with Item (1), provided that the temperature-induced forces and moments reduce as yielding in the reinforcement occurs and the increased concrete cracking does not cause deterioration of the containment.

Construction-fabrication

The requirements for fabrication, construction, and installation of reactor vessels and containments apply to operations performed in a shop or at a field site. However, nozzles and electrical penetrations unbacked by structural concrete are to be fabricated in accordance with ASME Section III, Division 1, of the Boiler and Pressure Vessel Code.

For the concrete portions of both of these structures, it is required that the Construction Specification prescribe the sequence and specific procedures for stockpiling, batching, mixing, conveying, depositing, consolidating, curing, and construction joint preparation. The Construction Specification must state whether stockpiling, batching, and mixing are required to be performed at a separate on-site facility or at an off-site facility specifically devoted to this project, or under what conditions and provisions alternative facilities are to be permitted.

Also discussed are fabrication and installation requirements for reinforcing systems, prestressing systems, and liners.

For both reactor vessels and containments, there is an article covering construction testing and examination as applied to concrete, reinforcing and prestressing systems, and metal liners.

The structural integrity tests are similar for both reactor vessels and containments. In general, in order to demonstrate that concrete reactor vessels or containments respond satisfactorily to required internal temperatures and pressure loads, a program of measurements developed by the designer must be followed to provide correlation with theoretically predicted response and to prove the adequacy of the structure with respect to quality of construction and materials.

Overpressure protection

For the protection of concrete reactor vessels against overpressure, the Code provides that the vessels shall be protected while in service from the consequences arising from the application of steady-state or transient conditions of pressure and (coincident) temperature that are in excess of the design conditions specified in the Design Specifications, for which system overpressure protection is required.

For reactor containments, which operate under radically different levels of pressure from those seen by reactor vessels, requirements are correspondingly different. As a result, discussion in this area includes when and how protection against overpressure shall be provided. There is also a discussion of vacuum and pressure relief devices and their use.

Inservice inspection

The sections on inservice surveillance of reactor vessels and containments are different in content since, again, their service requirements are different. The requirements for vessels include requirements for tendon prestress force and corrosion monitoring vessel response to pressurization monitoring and liner material surveillance. Any other items requiring monitoring are to be designated in the design specification. Monitoring may be either continuous or intermittent, depending upon the expected rate of change, and consequence thereof, of the item being monitored.

For the reactor containments, the rules presented provide that, when implemented by a surveillance programme, they will constitute an appropriate means of assessing the structural and leak-tight integrity of the structures and their penetrations. To accomplish this the containment structures and their penetrations are to be designed so that (1) periodic integrated leakage rate testing can be conducted at containment design pressure, (2) all important areas, such as penetrations and seals, can be inspected, and (3) an appropriate surveillance programme can be carried out.

It should be noted that the sections on inservice surveillance will eventually be removed from the present document and made available for incorporation in ASME Section XI, Rules for Inservice Inspection of Nuclear Reactor Coolant Systems.

Appendixes

The mandatory appendixes include tables of materials for metallic items and components, methods of determining concrete multiaxial compressive strength modification, a glossary of terms and symbols, and methods for approval of new materials. They also include data report forms, porosity charts, qualification of inspection and non-destructive examination personnel, and non-destructive examination methods.

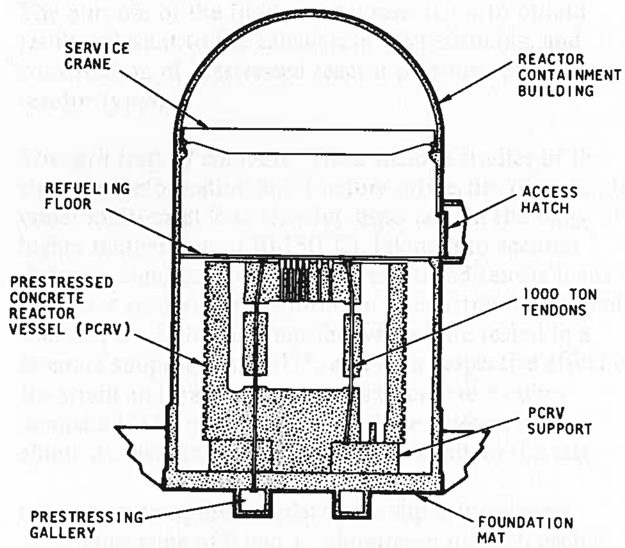


Figure 1. Typical concrete reactor vessel and concrete structures.

Research for prestressed concrete reactor vessels in Germany

H. Goffin (FDR)

INTRODUCTION

Long-range studies of the future structure of energy supplies in the Federal Republic of Germany indicate high growth rates, especially for natural gas and nuclear energy. Thus in the 1980s even with assured supplies of mineral oil, which will still be the main source of energy with a share of some 50%, the demand for nuclear energy will be 20 times higher than it is today, assuming a 70% increase in the consumption of primary energy for the next 12 years. For the Federal Republic of Germany this means that, allowing for the shutdown of power stations, approximately an additional 90 000 MWe will have to be installed by 1985, with some 40 000 to 50 000 MWe being supplied from nuclear power stations.

With this great task in mind, comprehensive research work has been initiated under four nuclear programmes with close international contact, designed to develop advanced reactor types and an efficient reactor-building industry, with special attention focused on safety precautions. The extremely high requirements in this area in particular, together with the extraordinary loading conditions, as a result of operational and safety considerations, have created problems of a novel type for civil engineers. Quite apart from leading to difficulties in the authorisation procedure, especially with respect to those cases where fragmetary knowledge or fundamentals of material laws and parameters and realistic calculation and dimensioning methods have prevented satisfactory answers to novel-type questions.

These sort of problems caused delays in the building of nuclear power stations, which, together with excessive boundary value considerations in connection with high safety requirements, often had an adverse effect in terms of economy.

RESEARCH FOR THE DEVELOPMENT OF PRESTRESSED CONCRETE REACTOR VESSELS

Basic programme

In view of the situation just outlined, civil engineerin research for nuclear power stations was given supreme priority. As a part of this research, the Deutscher Ausschuss für Stahlbeton (German Committee for Reinforced Concrete) was assigned the task, by the Ministry of Research and Technology, of supervising and coordinating the basic programme of German research and development in prestressed concrete reactor vessels. For this programme, which allows for safety requirements as well as economic and technical aspects of civil engineering, is specifically designed to develop prestressed concrete reactor vessels for light water and high temperature reactors, approximately DM 4 m per year has been provided for a provisional period of some four years; from these funds some 25 research projects in the areas of fundamental and type-oriented research have so far been supported.

Fundamental research

The purpose of the fundamental research is to obtain results relevant to the calculation, dimensioning, and construction of prestressed reactor pressure vessels for all reactor types.

Strength tests of concrete. These include studies of the strength, deformation and fracture properties of concrete under multi-axial load transfer, especially in the range of higher temperatures (20-150°C), taking into account different combinations of compressive and tensile loads. For these studies, on the problem of constraint-free load transfer, the following transfer systems are tested in a separate subprogramme(1)*, and their respective effect on the strain and fracture properties of concrete cubes compared. The main purpose of these systems is to eliminate restraints on the transverse strain in the test:

(a) rigid compression plate with slip foil packages consisting of 6 and 12 aluminium foils(2), each 0.02, 0.05 or 0.1 mm in length, lubricated with molybdenum disulphide;

(b) load brushes consisting of 90 mm steel rods (bristles), 4 x 4 mm in cross section(3);

(c) 'slack compression plate', made up of 16 single punches embedded in silicone rubber within a frame so that the punches are relatively easy to move towards each other. By using this device a large uniform stress distribution can be achieved. A supporting rubber plate acts like a hydraulic cushion(1).

*Numbers in parenthesis refer to Institutes carrying out Research. A full list is given at the end of this paper.

The established decrease in compressive strength depending on transfer system—rigid plate, rigid plate with slip foils, brushes, soft compression plate—illustrates the way in which the system of transfer (due to restraints on transverse deformation) affects test results, and therefore also permits appropriate rating of their respective significances.

Multi-axial studies are carried out on cubes with edges 10 or 20 cm in length. Figure 2 shows the testing device with load brushes(3).

Tensile forces are applied by gluing the brushes on the concrete. During all load increments the deformation properties are registered by means of embedded strain gauges and integral surface measuring. For the incorporation of internal strain gauges a special method was developed which proved to be very reliable.

As part of the preliminary studies, another topic was investigated: the influence of reinforcement on the multi-axial strenth of concrete, with special reference to the appropriate structure of test specimens(4).

A knowledge of the influence of changes in stress and temperature on compressive strength as well as on stress/strain properties is desirable for an accurate assessment of the long-time strength and ultimate bearing capacity of the concrete vessel. In this connection, dry and wet concrete is subjected to temperature changes in the range 20 to 200°C. Temperatures are changed on an hourly, daily, or weekly basis; in addition to strength and deformation, cracking inside the prismatic test specimens is also investigated. Another factor which needs to be determined is the remaining residual strength following a limited number of great changes in stress have taken place, and after overloading up to the near short-time strength limit.

Studies for an increase in tensile strength and elasticity of concrete by steel-fibre reinforcement are being carried out, particularly, in regions where local stresses occur in prestressed concrete reactor pressure vessels—such as openings or pipe inlets. It is not possible to compensate the tensile stress peaks by prestressing. The concentration of bonded reinforcement in these regions leads to an aggravation of design and construction. Of a special advantage here would be a quasi-homogeneous and isotropic material, which could render it possible and at the same time steer purposively to a definite tensile strength and to increase ductility towards impact loads.

This objective would then be followed up by appropriate studies of production, material characteristics and economical uses of steel-fibre concrete* with particular reference to its application to the construction of prestressed concrete reactor pressure vessels.

In the first stage, the main points to be established are:

(a) Static and dynamic strength properties which depend on the steel-fibre content and distribution, (and consequently also on the mixing method used), concrete composition, the type of steel fibre used (length, shape and diameter of fibre, galvanised or bright fibres).

(b) Creep and shrinkage, properties, especially tensile creep.

(c) Thermal conductivity against variation of fibre content.

(d) Resistance to increased temperatures.

First results indicate that the tensile bending strength of steel-fibre reinforced specimens largely depends on both the tensile strength of the un-reinforced concrete and on the fibre content; however, above a certain fibre content there is no further increase in the tensile bending strength. Because of the limited length of the fibres (l≈100 x diameter), there is a certain slip which means that the advantages of the fibre strength are lost.

The use of galvanised fibres, in comparison with bright fibres, has an adverse effect on strength properties.

Material parameters

In order to formulate material laws adapted to the specific conditions of thick-walled vessels, a series of basic investigations is carried out to determine the material parameters to be used, taking into account various influencing factors. The following is some of the projects included.

Investigations of moisture changes of concrete under increased temperatures, and of the influence of moisture content and degree of maturity on thermal conductivity(1). On six large concrete beams, the successive changes in the thermal field as well as the moisture flow are measured in the direction of the longitudinal axis, under the influence of a temperature gradient of 60 deg C. Shrinkage properties are also studied at the same time.

First preliminary results suggest that shrinkage stresses due to moisture emission in a reactor vessel only occur on the outer and inner surfaces.

Subprogrammes are carried out to determine, on cylindrical specimens, calibration curves for the relation of thermal conductivity as a function of moisture content and degree of maturity of concrete.

In addition, the distribution of temperature and moisture fields is also determined by theoretical investigation(6).

Creep properties of concrete subjected to multi-axial loading with different stresses along the various principal axes, cannot be adequately covered by present approaches, since these are based on a constant transverse creep ratio (Poisson's creep ratio).

Therefore, for a universal creep law for multi-axial stress conditions, disc-shaped test specimens 20 x 20 x 5 cm are to be investigated under bi-axial loading to determine the dependence of the transverse creep ratio on:

*Concrete reinforced with steel fibres: straight, short, possibly profiled, distributed unevenly and non-directionally across the diameter (0.1 to 0.8 mm, l≈diameter x 100).

(a) Stress condition
(b) Temperature (up to 120°C)
(c) Type of aggregate
(d) Load period.[3]

Further investigations are directed towards information about:

(a) Creep and creep recovery of samples under load for 10-15 years and

(b) Creep of 15 year-old samples which have never been subjected to uniaxial loading[3].

Influence of radioactive radiation on the material characteristics of concrete[5]. In a review of pertinent bibliography, the influence of radioactive radiation on concrete properties has been investigated. In attempt to represent this influence by means of appropriate functions met with serious difficulties, owing to substantial differences in the conditions under which the various experimental investigations had been performed. However, a certain trend was apparent: at a radiation dose greater than 7 x 10^{19} neutrons/cm^2 (slow neutrons), a reduction in strength, an increase in volume, and a change in the thermal conductivity of concrete are likely; but this influence only becomes clearly visible at 10^{20} neutrons/cm^2, or with an increasing dose of fast neutrons.

To determine the coefficient of thermal expansion of concrete, the following questions are to be investigated in more detail [5]:

1. Is there any theoretical means of forecasting the coefficient of thermal expansion of concrete based on the properties of the components, i.e. cement stone and aggregate?

2. How do differences in temperature properties of various grain fractions of different mineralogical deposits influence temperature expansion of concrete, and what grain fractions could possibly be replaced with aggregates of high thermal expansion coefficients, without substantially increasing the thermal expansion of the concrete involved, but improving its working properties?

3. How can the thermal expansion of concrete be reduced in general?

On the question of corrosion protection for prestressing steels, a study was made of galvanised high-strength prestressing steel with reference to:

(a) Quality and structure of zinc coating,
(b) Mechanical characteristics of prestressing steel,
(c) and adhesion of zinc coating and their abrasion properties[7].

These investigations showed that as a result of galvanisation, the strength and deformation characteristics of steel more or less changed. In some cases, there was a substantial loss in anti-corrosion protection, too, owing to abrasion and mechanical influences on steel surface.

Calculation methods–theoretical and parametric studies of reactor vessels. Two research programmes are concerned with computer-oriented calculation methods, based on Finite Elements[8] and Dynamic Relaxation[4], respectively. Incorporated in the program systems are continuously improved material laws describing the non-linear material characteristics in long-time analysis as well as cracking and fracture behaviour in limit state conditions so as to create, by realistic calculation methods, the basis for a safe as well as an economical vessel design.

The investigations on the influence of upper and lower material boundary values on the state of stress and strain of vessels produce points of reference for further specific material investigations.

A recalculation of the test results obtained on model vessels has by and large confirmed the validity of the calculation assumptions and methods for operating conditions, using a 1:5 THTR model, a 1:47 cast resin model[4], and a Scandinavian top closure ultimate load test.

This fundametnal theoretical work is carried further by studies in optimisation for prestressed concrete pressure vessels of high temperature or light water reactors[4]. Taking into account characteristic factors of influence, systematic comparative and parametric investigations are carried out to work out what civil engineering conclusions should be drawn fro different concepts of vessel design and construction, and to establish design criteria based on such conclusions.

Vessel instrumentation

An appropriate vessel instrumentation is to be developed in order to create the basis for checking vessel properties under operating conditions as well as for verifying load assumptions and calculation methods. Highly sensitive gauges for laboratory research are also being developed.

Specific research work. Based on the long-term observation of the development of nuclear power stations, estimates show that in the next twenty years light water reactors are likely to increase their efficiency to unit outputs of 1500-2500 MWe. Although steel pressure vessels, definitely successful for this reactor type, would seem to be feasible for an output of up to approximately 2000 MWe, again, when taking into account the difficulties of transportation and construction for larger steel pressure vessels as well as the added specific requirements in terms of safety in such vessels (extra bursting protection), prestressed concrete appears to be advantageous both in terms of economy and safety engineering. Individual specific areas involving lid, inner liner, penetrations, thermal insulation, are, and will be, items of continuous investigation.

As to the development of liners, present studies will be concentrated on the following three types.

Hot liner

The Austrain Co-operative Project,[9] which will investigate a hot liner for prestressed concrete of pressure water reactors with an output of 1500 MWe has been repeatedly described in detail at technical conferences.

Fastened in insulated (lightweight) concrete with short bolts, the hot liner consists of a martensitic steel which has an elevated temperature yield point so high (38.1 t/in^2) that even when the reactor is in operation the liner fully remains within the elastic range.

The disadvantages of this concept seem to be:

Relatively high temperature of 100°C in concrete

Low failure strain of liner steel

High tensile load on hammer bolts at the cooling of the liner (failure of heating system).

Concurrently with model vessel tests (1:3 scale), the suitability of the concept for large construction is investigated by designing a reference vessel for pressure water reactors with an output of 1300-1500 MWe, and by comparing the cost estimates of steel pressure vessels and prestressed concrete reactor pressure vessels.

Under a concept developed by Kraftwerk-Union Erlangen, the inner steel vessel, (unlike other solutions, where the liner serves for sealing purposes only), performs part of the load-carrying function (the 'supporting tank' principle). The supporting tank (length 50 mm), at the beginning of reactor operation, has to absorb the entire internal pressure up to σ_{zul}*. Only with further increase in pressure will the tank, through the 200 mm hinged supports distributed across the circumference at a distance of 70 mm each, rest on the concrete body. Therefore the supports in the assembled condition have a distance (clearance) of 5 to 7 mm from the supporting tank.

The advantage of this system lies in the fact that the supporting tank at a maximum internal pressure of 180 at is subjected to a compressive stress of only 0.25 β_S, and that supporting tank and concrete body can be manufactured separately.

This system is unfavourable because the manufacturing process would seem to be expensive due to the high demand made on accuracy; besides, it is also necessary to study very carefully, in terms of calculation, the load redistributions as a result of the concrete deformation during operation.

Otherwise, the system would form the basis for the ready-to-build design of an integrated prestressed concrete reactor pressure vessel of the 'star-shaped vessel' type for a 1500 MWe pressure water reactor[11].

The central vessel is provided for the reactor core, and the four reactor core, and the four outer satellite vessels accommodate the steam generators of the primary circuit.

For the design of a prestressed concrete reactor pressure vessel for light-water reactors a liner type was chosen which enabled a 300°C coolant to be applied directly to the liner[12].

*the allowed stress

The austenitic steel used for corrosion considerations, which has already been successfully tested in the reactor operation, is systematically plasticised during the reactor operation. With every shut down the liner goes through a reverse plastification.

In tests made on large liner sections, more than 2000 temperature changes have been carried out without damage to the liner. Between the hot and cold liners, which are linked through fastening bolts and at the same time serve as a formwork for concreting, a special ceramic insulation is provided which together with the cooling system protects the prestressed concrete part against excessive temperatures. The advantage of this system lies in the simple structure.

For certain reactor types as well as for the design of concrete containments, liner integrity is a major liner-related problem. In terms of the research projects currently in progress, the basic problem is the following:

The liner is linked by shear connectors and studs, at narrow distances, as well as by friction between the sheet and concrete (increased by high internal pressure). Thus in the main, it follows that the deformations of the prestressed concrete vessel, the concrete deformations, especially on passing from state I to state II (cracking) in local concentration, having to be accomodated by the liner with the degree of safety required for its tightness; disturbances owing to local factors (deformation restraints, material defects, stress peaks) must be taken into account.

In the experimental part of the research work concerned[7], as a first step, the question of slip restraints between the liner sheet and concrete due to cohesion, and of the friction due to internal pressure is investigated on sandwich like specimens. For these investigations, liner sheets are inserted in the mould without shear connectors. Checks are carried out by removing the sheets under specified lateral pressure.

A second test series planned is to determine the ductility of the liner when the concrete cracks. This includes the transfer of tensile forces to the concrete core covered on both sides with a liner.

In a third test, the influence of changing bending moments is also to be studied.

In the theoretical part of the research work on liner integrity[1], existing computing programs are to be modified based on the test results gained so as to sufficiently calculate Liner deformations or liner stress occurring in the region of concrete cracking.

Furthermore, as a part of parametric studies, criteria of appropriate arrangement of bonded reinforcement or the use of steel-fibre concrete on the inner surfaces of the vessel are to be worked out so as to possibly eliminate extreme deformation discontinuities on liner sheets.

Outlook

The results of fundamental research so far carried out on prestressed concrete reactor pressure vessels allow an economical and realistic dimensioning and design, including a strictly specified safety risk, of prestressed concrete reactor pressure vessels of all systems, for any type of load situation. The experience and knowledge gained from this will be of greatest benefit to the future development of the entire reinforced and prestressed concrete engineering field.

As in the next 10-15 years light water reactor nuclear power stations will be still dominating the market, civil engineering, detail questions of relevant types of reactors–calculation and design of liners, their fastening, heat inslulation, top closures, pipe penetrations–and related solutions, feasible both in terms of economy and advanced safety concepts, will be studied in type specific research.

INSTITUTES CARRYING OUT RESEARCH

(1) Bundesanstalt für Materialprüfung–BAM–Berlin-Dahlem.

(2) Friedr. Krupp GmbH, Universalbau, Essen.

(3) Institut für Massivbau der Technischen Universität München.

(4) Institut für konstruktiven Ingenieurbau der Ruhr-Universität Bochum.

(5) Institut für Beton und Stahlbeton der Technischen Universität Karlsruhe.

(6) Institut für Bauphysik, Stuttgart.

(7) Institut für Baustoffkunde und Stahlbetonbau der Technischen Universität Braunschweig.

(8) Institut für Statik und Dynamik der Technischen Universität Stuttgart.

(9) Reaktorbau Forschungs- und Baugesellschaft m.b.H. & Co., Seibersdorf (Austria)/Kraftwerk-Union, Erlangen.

(10) Kraftwerk-Union, Erlangen.

(11) Dyckerhoff & Widmann, München.

(12) Friedr. Krupp, GmbH, Essen.

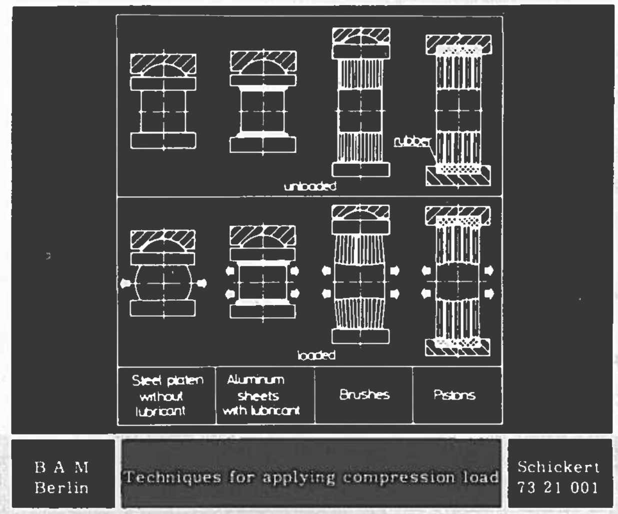


Figure 1. Systems for compressive load transfer.

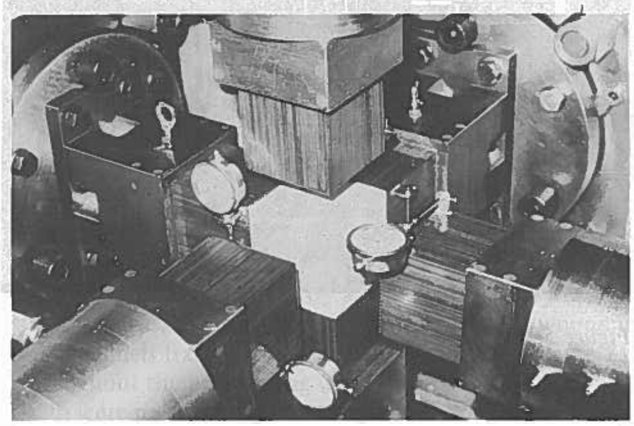


Figure 2. Testing device with load brushes.

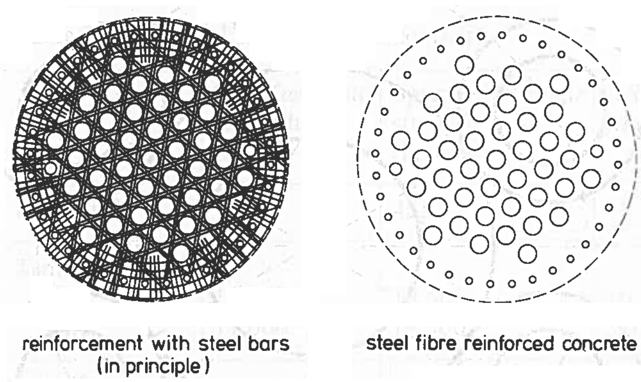


Figure 3. Standpipe area of PCRV structure, showing conventional reinforcement on left and steel fibre reinforcement on right.

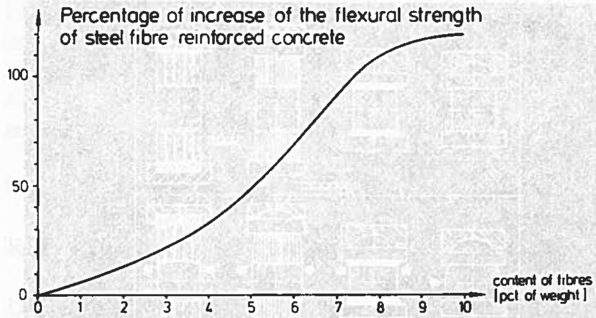


Figure 4. Increase in tensile strength of steel fibre concrete as compared with unreinforced concrete.

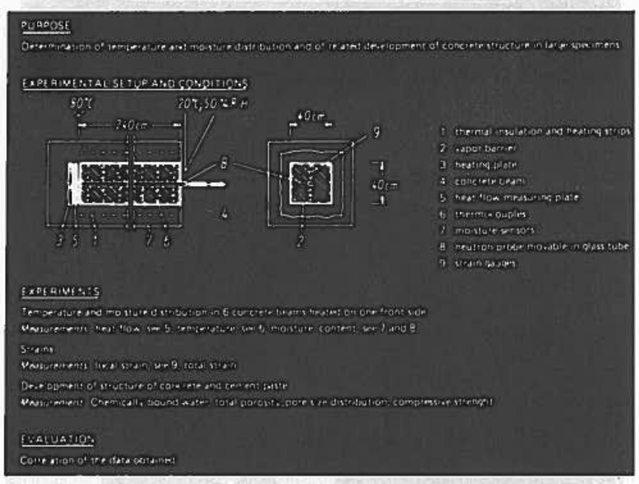


Figure 5. Moisture transfer in concrete.

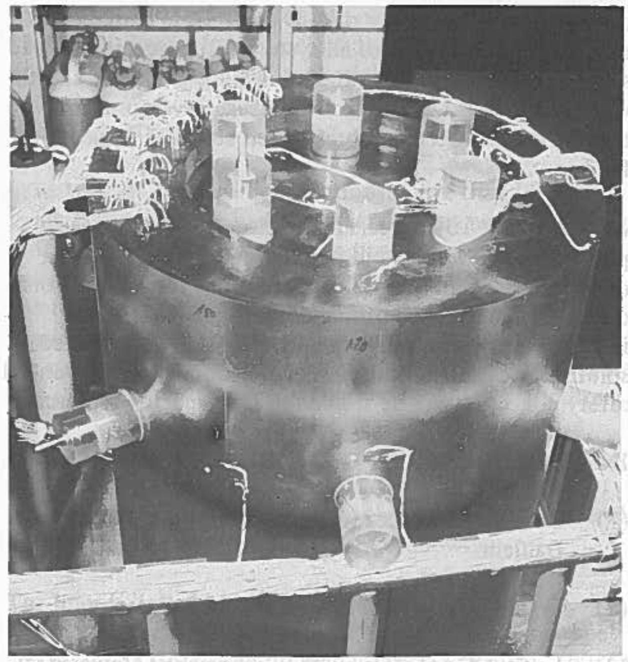


Figure 6. Cast resin model, 1:47 scale.

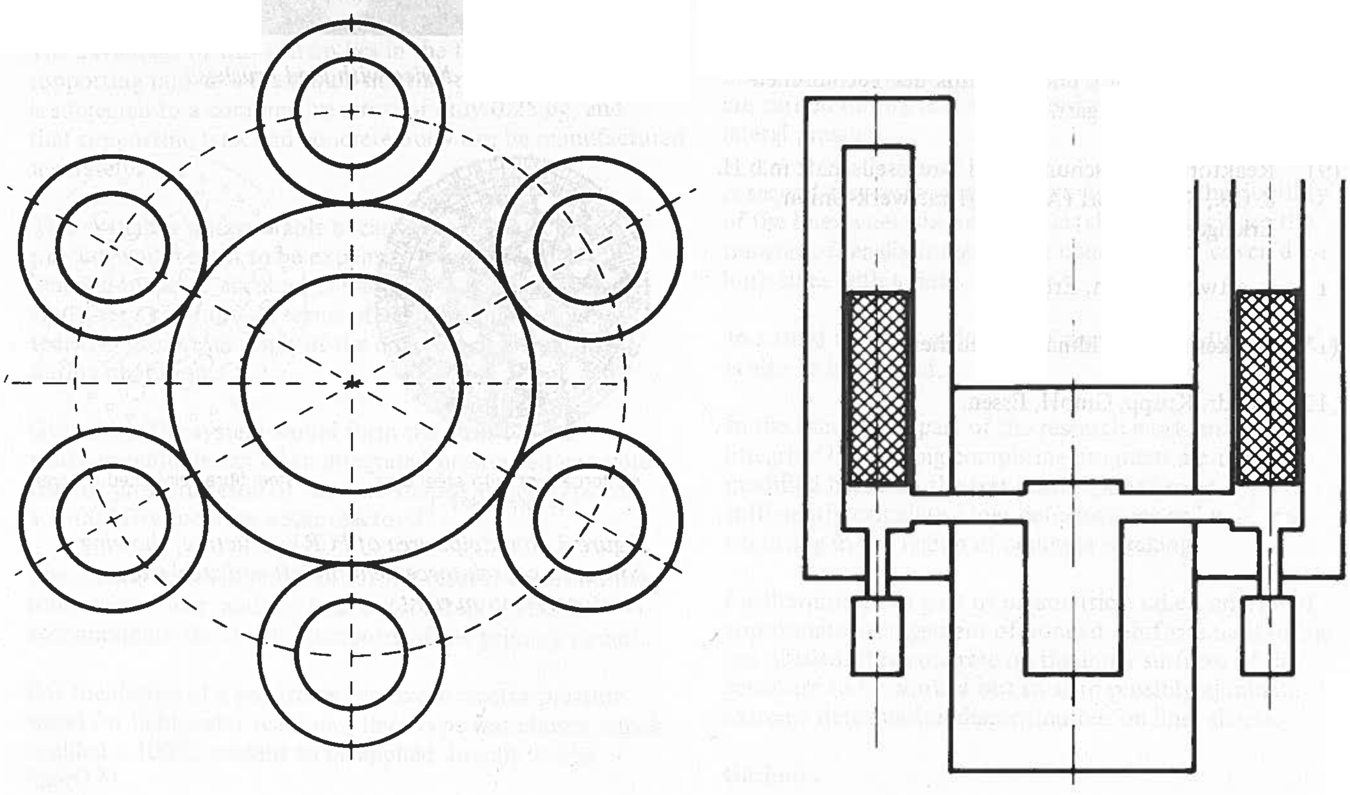


Figure 7. Horizontal and vertical sections of star-shaped vessel.

Pressure tests of PCRV models

T. Takeda, Y. Takemoto, T. Yamaguchi, M. Ito, K. Imoto and T. Tada

INTRODUCTION

During this decade prestressed concrete reactor vessels (PCRV) have been constructed and widely used and have demonstrated their safety.

Some new types of PCRV have now been proposed based on advanced reactor technology. The multi-cavity type PCRV for a high temperature gas-cooled reactor is one of the new types which has been now given attention in the prestressed concrete engineering field. A research programme on the structural behaviour of this type of PCRV is being conducted at the Technical Research Institute of Ohbayashi-gumi Ltd. This programme includes pressure tests of PCRV models, vibration tests of support structures and development of an analysing method.

The results of pressure tests on two 1/20 PCRV models and the comparison between measured and calculated values are reported in this paper.

DESIGN MANUFACTURE AND MATERIAL PROPERTIES OF TEST SPECIMENS

Two prestressed micro-concrete cylindrical models were manufactured as test specimens. One of them (multi-cavity type-M-2) was an accurately scaled down model of the prototype structure designed for a 1000 MWe high temperature gas-cooled reactor. Scale dimensions were 1/20 and the reinforcement ratio and prestress level were almost equivalent to the prototype which had six steam generator cavities around the central core cavity and star-like support structure connected rigidly beneath the bottom slab (Figure 1).

The other (single-cavity-M-1) model was designed to an equal size and strength to M-2 but without any steam generator cavity or support structure.

Both specimens were manufactured using the following procedure:

Arrangement of non-prestressed steel and duct sheathes on the base slab

Positioning of prefabricated liner plates

Setting up of steel formwork segments (Figures 3 and 4)

Placing concrete monolithically (no construction joint)

Curing (40-70 days)

Vertical prestressing

Circumferential prestressing.

Prestress levels of each portion were so designed that the specimens were essentially elastic at the design pressure of 50 kg/cm^2. High strength steel bars, 12 and 16 mm in diameter, were used for the vertical prestressing. Prestressing force were controlled by a load cell and strain gauges were attached on the bars.

In order to obtain a steady circumferential prestressing force, a special wire-winding apparatus was set up which controlled the wire tension by a counterweight. By using this apparatus, 2.9 mm diameter steel wires were wound in steel channels fixed on the outside of cylinders (Figure 2). Throughout the prestressing work, several points of concrete strain were measured by the embeded strain gauges.

The mechanical properties of materials used are shown in Table 1.

Table 1. Mechanical properties of materials

	Specified yield strength (kg/cm^2)	Specified ultimate strength (kg/cm^2)	size (mm)
Concrete*		455	
Tendon			
bar	11,000	12,500	16 12
wire	17,000	19,500	2.9
Reinforcement			
indented wire	17,000	19,500	2.9
deformed bar	3,500		13
Liner			
steel plate		4,100	2.3

*maximum aggregate size of concrete 6 mm

PRESSURE TESTS

Pressure loading was applied hydraulically to the cavities of each specimen by means of a steel liner which also prevented premature leakage of water.

After several repetitions of pressure loading up to the design pressure of $50 kg/cm^2$ some over-pressure loads were applied to the specimens to investigate crack development, final pressurization was then carried out to evaluate the ultimate load capacity and to observe the failure of the PCRV models. In general, one loading step was $5 kg/cm^2$.

About 350 points of measured data for each specimen were recorded and analised automatically using a data processing system which consisted of a small computer and some peripheral instruments. Cantilever-type strain transducers and embeded strain gauges of 30 mm gauge length were adopted for measuring displacements and concrete strains respectively. A few dial-gauges were also used for monitoring displacements at some critical portion of the specimens.

THEORETICAL ANALYSIS

The structural behaviour of each specimen due to both the prestressing force and the internal pressure were analised theoretically.

The finite element method was used for a non-linear analysis of the single cavity type model (M-1).

Assumptions made were as follows:

The model is an axisymmetrical body of revolution

Idealized elements took the form of a triangular linkage for concrete, bar for steel bars in the radial plane, hoop for wound wires, and hoop bars and plate for steel liners

The bond between the steel and concrete was perfect except for the vertical tendon (non grout)

The yield criteria of the concrete follows Prager and Drucker's equation

Unloading on plastic elements was judged by a proportional constant for the uncracked elements and the strain increment for cracked elements.

For the (M-2), multi-cavity type model the elastic responses to the prestressing force and the internal pressure were calculated by the three-dimensional finite element method of arbitrary hexagonal elements. The analytical model was a 30° portion of the specimen, divided into twelve parts. Steam generator cavities, gas ducts and support structure were taken into account, but standpipe openings in the top slab were neglected in the analysis; 166 elements and 1168 nodal points were adopted.

TEST RESULTS AND DISCUSSION

Strain distribution due to prestressing

Figure 5 shows the strain distribution in the concrete for both specimens, immediately after prestressing.

The strain profile due to vertical prestressing is shown in on the far left of Figures 5A and 5B for each specimen. The middle diagrams show the profiles of radial and circumferential strains due to circumferential prestressing. The measured and calculated data agreed within acceptable limits. The effect of support structure on the strain distribution of that portion of the M-2 specimen is clearly shown in Figure 5B.

Structural behaviour during pressure tests

In the following paragraphs the structural behaviour of specimen models during pressure tests are described in terms of deflection, concrete strain, crack pattern and failure mode.

Deflections of models

The relationship between pressure loads and deflections of each model at the centre of top slab and the midheight of cylinder wall, given in Figures 6A and 6B respectively, illustrates a good recovery from the deflections at each peak pressure level, and therefore suggests the ductile behaviour of a PCRV loaded by over-pressure.

Deflection profiles measured along the surface of the top slabs for both specimens show that this portion of a PCRV model mainly behaves as a flexural member. The cylinder wall deflection of the M-2 model which has S/G cavities is rather large in flexure compared with that of the M-1 model.

Concrete strains

Figure 8A shows that strain distribution in concrete of specimen M-1 pressurised up to $120 kg/cm^2$. Measured and calculated results were found to be in good agreement.

The strain distribution of M-2, at a pressure of $100 kg/cm^2$ is given in Figure 9B. The effects of S/G cavities on strain profiles in the cylinder wall sections are apparent from a comparison of both models. Figure 9A shows a typical behaviour of strains at the vicinity of the inside corner and the mid-height of the cylinder walls. Cracking of the inner zone of concrete can be assumed from the strain development.

Tendon strains

The behaviour of tendons during over-pressure conditions would appear to be essential with regard to deformability, ultimate capacity and failure of a PCRV structure. Figure 8 shows the response of typical vertical and circumferential tendons of M-2 to the final pressure loading.

No tendons of specimens M-1 went beyond the elastic limit of the vertical tendons of the inside zone, while all vertical tendons and midheight portion of circumferential tendons of specimen M-2 went beyond the elastic limit in the final state.

Crack patterns and failure modes

Several types of crack propagation in both specimens are illustrated in Figure 11. External surface cracks were recorded by observation and strain gauges attached to the concrete surface, while cracking in the core was measured by means of the embeded gauges.

Development of cracks in both specimens were as follows:

Specimen M-1 (single-cavity type). The specimen showed elastic behaviour within the design pressure 50 kg/cm^2 and all strains and deflection recovered completely at each unloading.

Initial cracking was observed at pressure of 75-80 kg/cm^2 on the central portion of top slab in the radial direction (see Figure 9(1)). Circumferential cracks on the upper and lower corner of the core cavity were presumed to have begun at 80-90 kg/cm^2 (2).

Initiation of the vertical cracks at the midheight of the core cavity was assumed to be at 150-180 kg/cm^2 (3). Vertical cracking on the outside of the cylinder was from about 190 kg/cm^2 (4).

Horizontal cracks at the midheight of the cylinder and shear-type circumferential cracks in the vicinity of the tendon bearing plates on the top slab were observed long after the pressure had reached 200 kg/cm^2 (5,6). When the hydraulic pressure reached 238 kg/cm^2, water leakage occurred in the cylinder near top slab.

Specimen M-2 (multi-cavity type). Elastic behaviour was also shown within the design pressure 50 kg/cm^2. Circumferential cracks at the upper corner, and vertical cracks at the midheight of the core cavity seemed to have occured first and, were followed by lower corner cracks at pressure of 90-100 kg/cm^2 (1,2). Horizontal flexural cracks on the midheight of the cylinder, vertical cracks on the cylinder just above the S/G cavities, and radial flexural cracks on the centre of the top slab were observed at about 120 kg/cm^2 (3).

Water leakage of specimen M-2 occurred at the same pressure 238 kg/cm^2 as M-1 by chance. The final crack paterns of M-2 are shown in Figure 12.

By the fact that the radial flexural cracks on the top slab are remarkable and the cracks on cylinder surface were found to be less severe for specimen M-1, it can be considered that the principal failure mode of this specimen was the flexural failure of the top slab.

For specimen M-2, every radial flexural crack on the top slab, horizontal flexural crack on the cylinder surface and vertical crack over the S/G cavities were also found to be equally remarkable.

It can be concluded therefore that the failure mode of this multi-cavity specimen was a complex type of flexural slab failure, wall failure by flexure and vertical cracking.

CONCLUSION

(1) Pressure tests on the models showed sufficient strength of about five times the design load with gradually progressing failure.

(2) The effects of the existence of S/G cavities, and a rigidly connected support structure were clarified experimentally by a comparison being made between both test results.

(3) The nonliner FEM analysis applied to specimen M-1 and the three-dimensional elastic analysis applied to M-2 showed their acceptability to predict the structural behaviour of PCRVs.

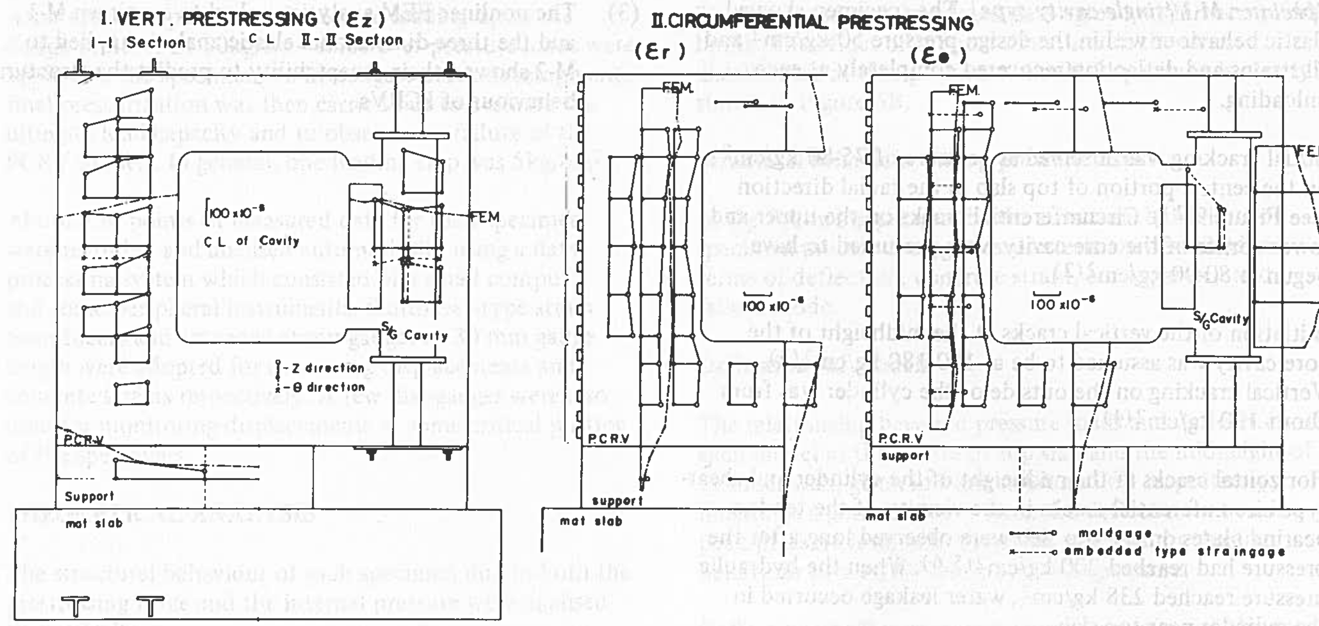
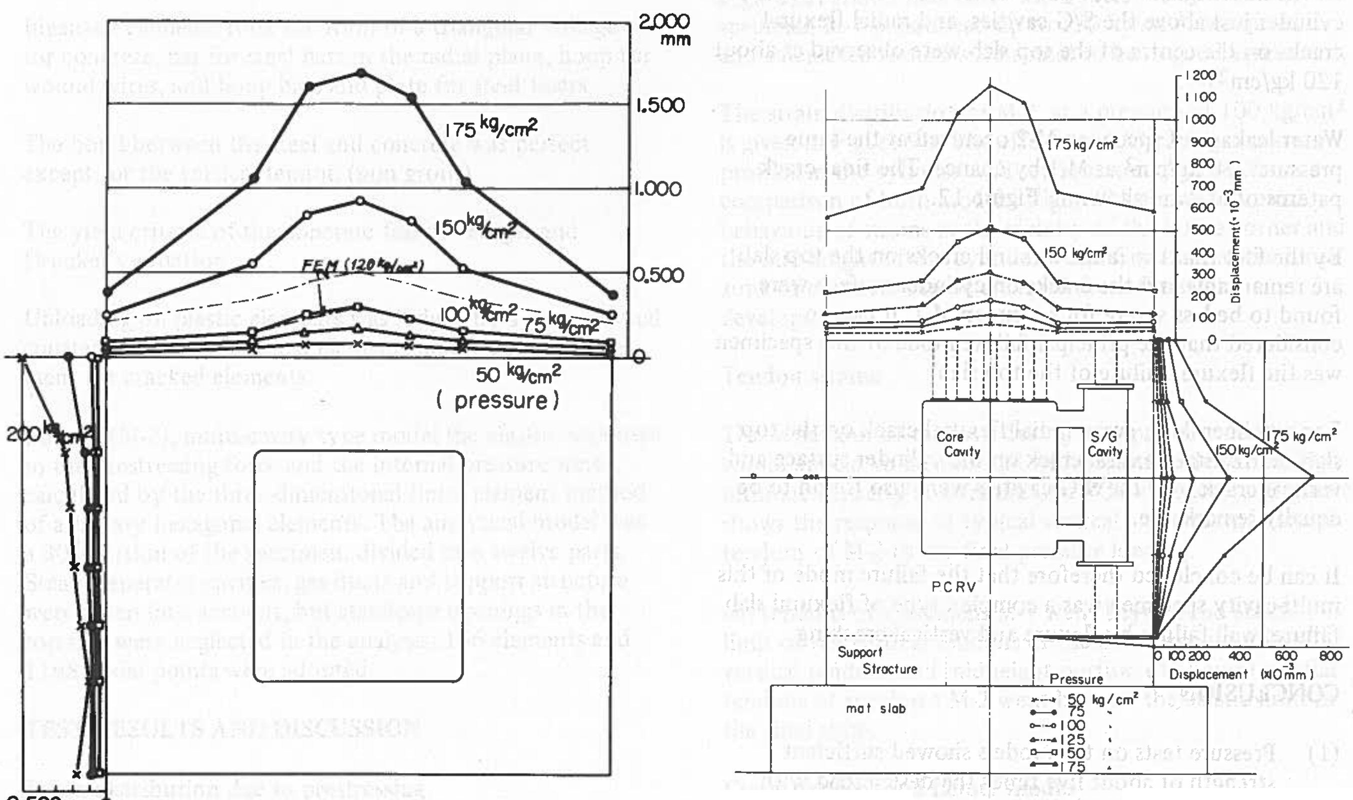


Figure 5. Concrete strain distributions after prestressing. (B) M-2 model



(A) M-1 model

(B) M-2 model

Figure 7. Deflection profiles of PCRV surface due to pressure

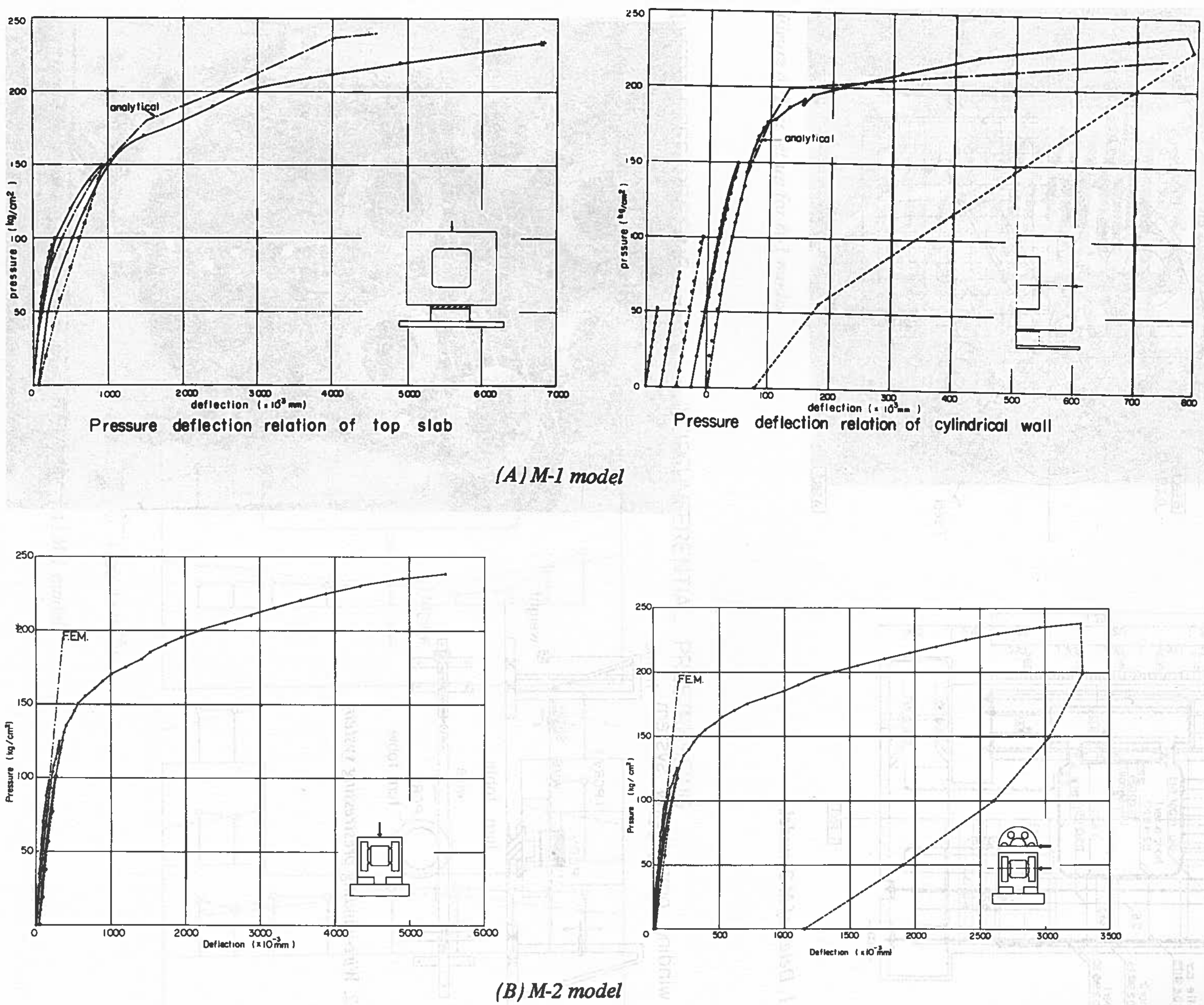


Figure 6. Pressure v. deflection relationship of top slab centre and cylindrical wall midheight.

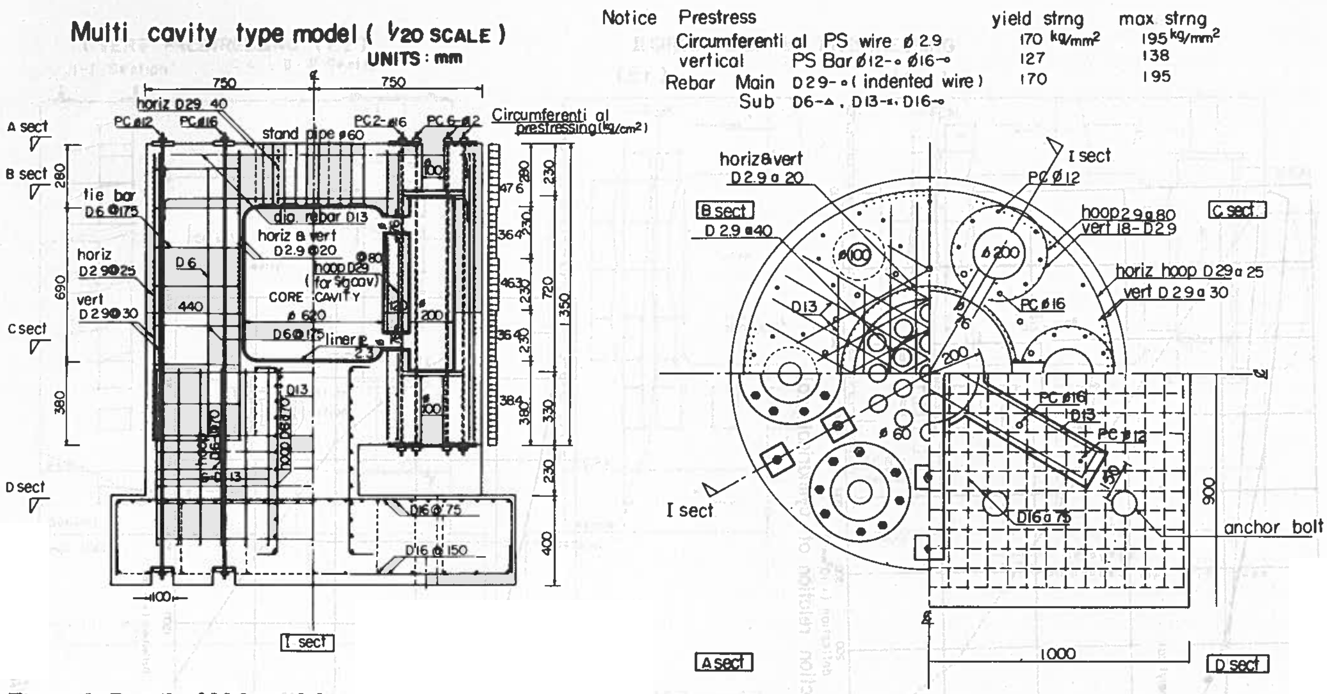


Figure 1. Detail of M-2 model.

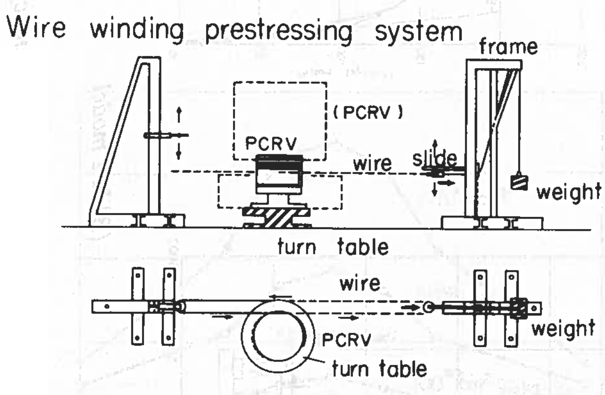


Figure 2. Wire-winding prestressing system.

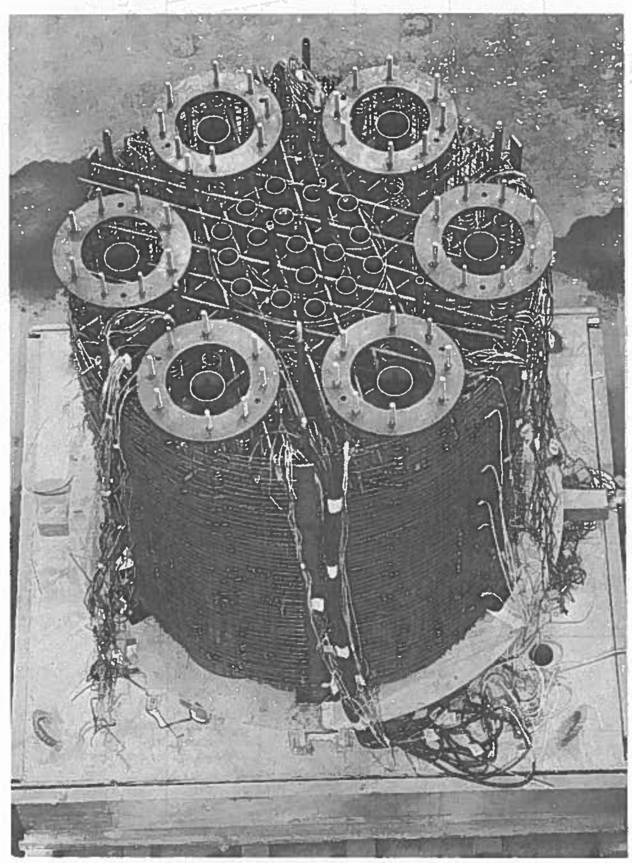


Figure 3. Total reinforcement of M-2 model before casting.

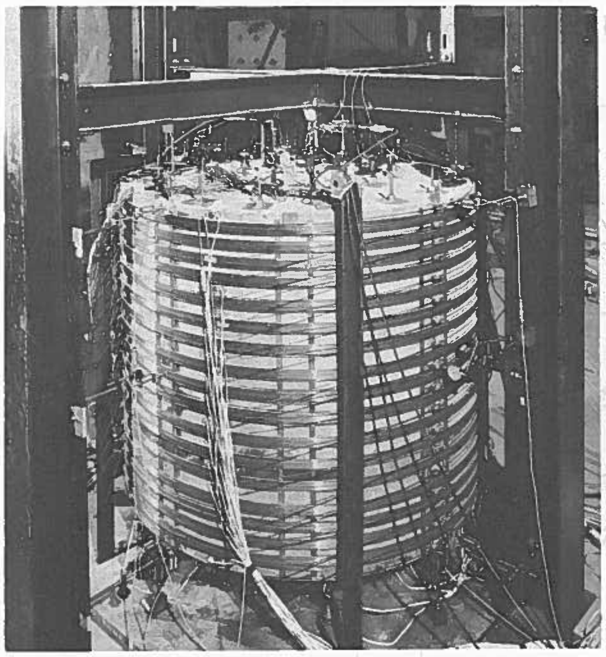


Figure 4. Test set up for M-1 model.

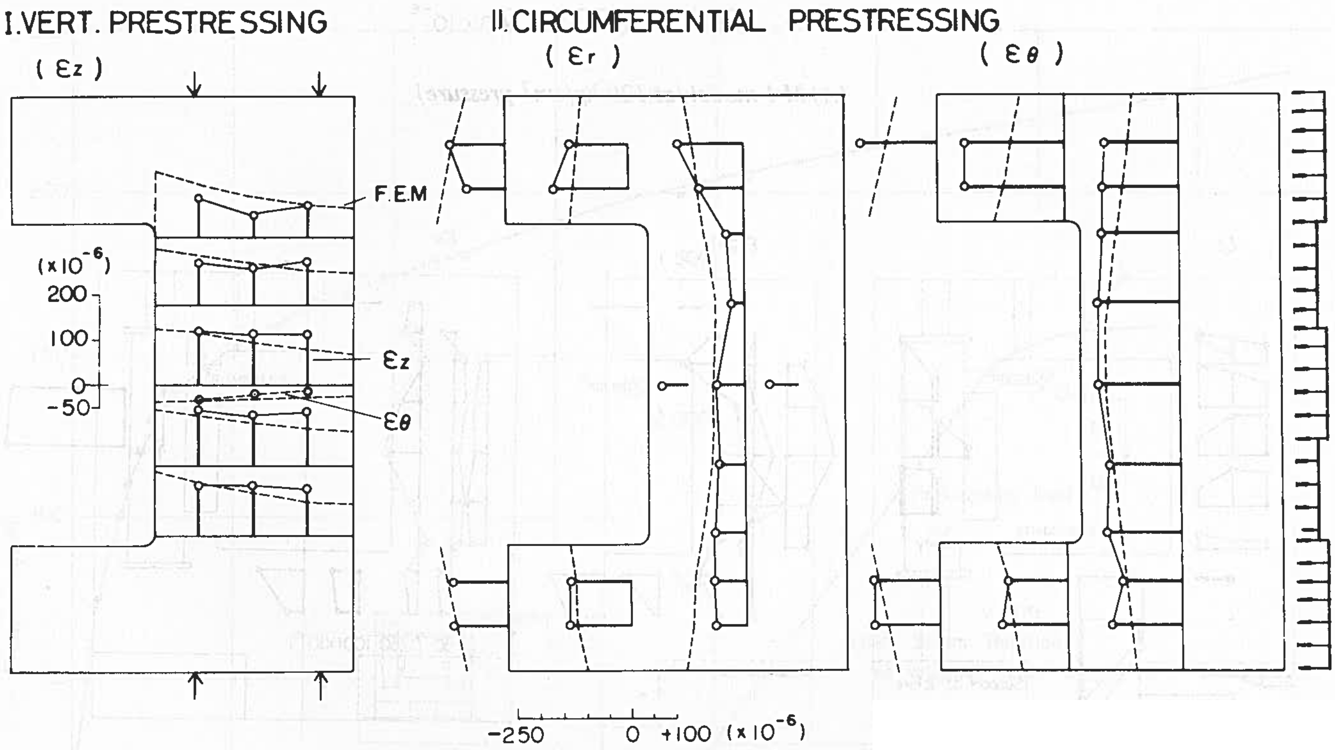
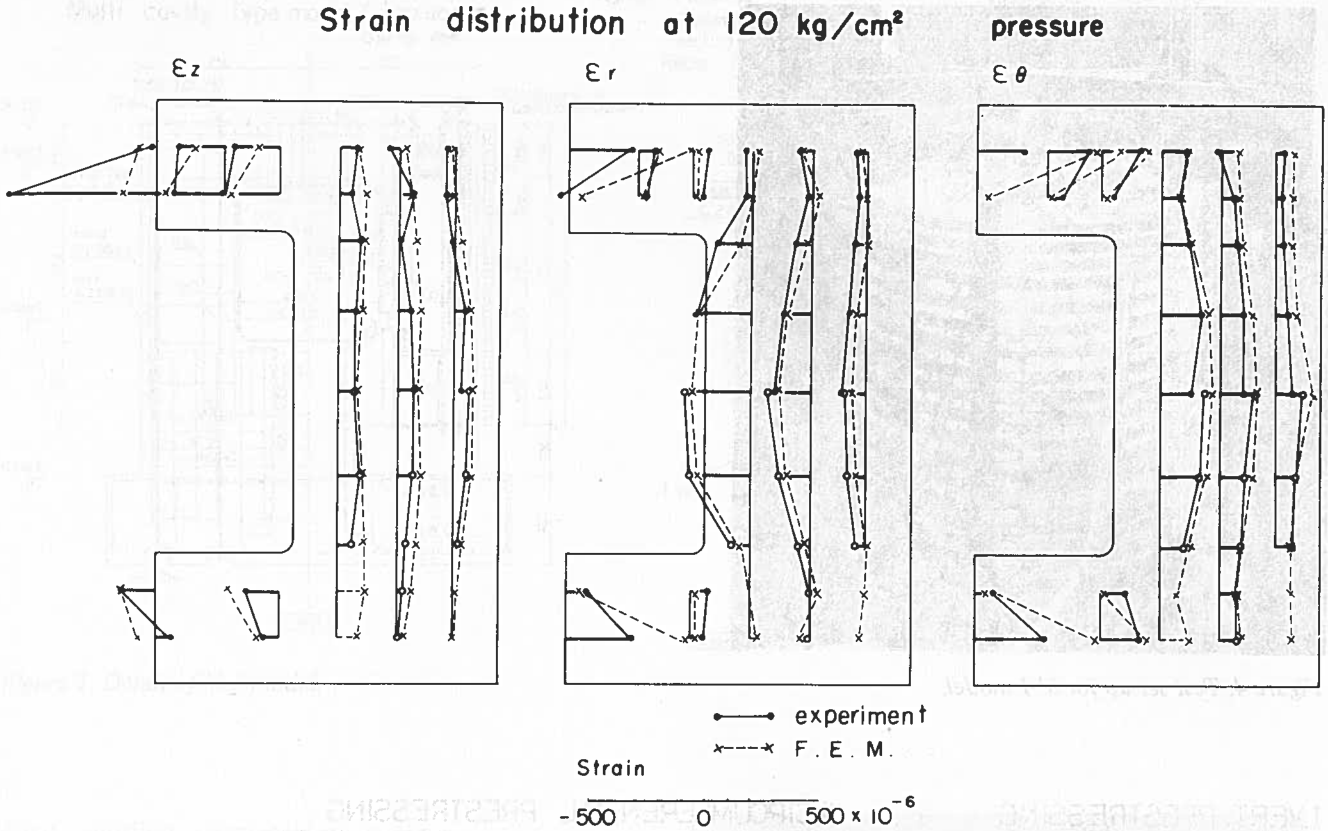
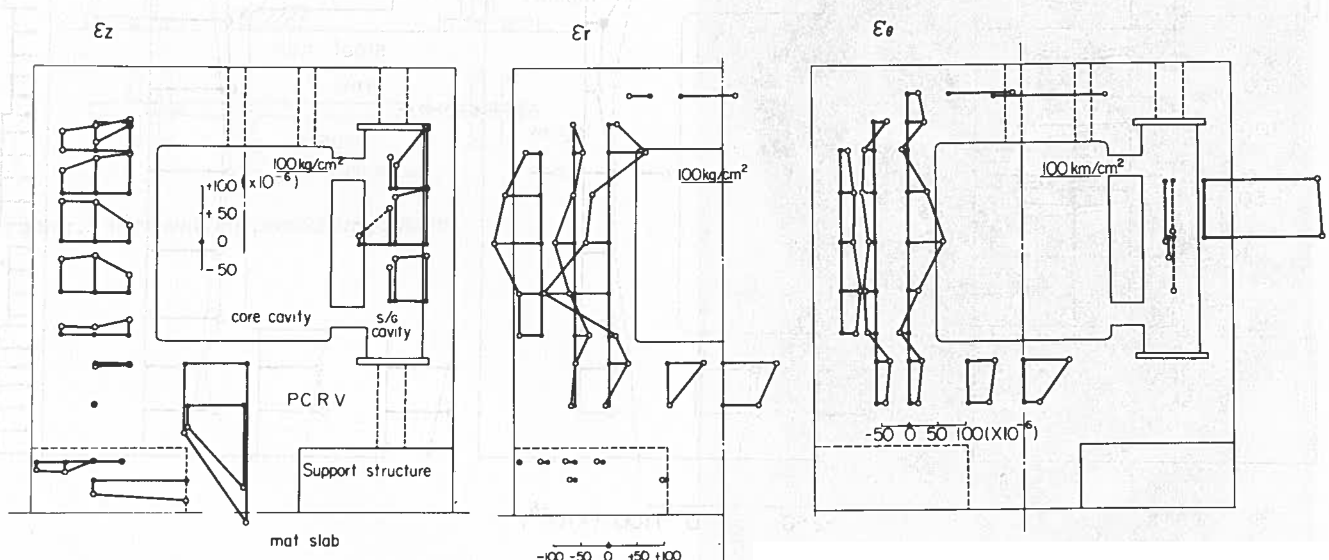


Figure 5. Concrete strain distributions after prestressing. (A) M-1 model



(A) M-1 model (at 120 kg/cm² pressure)



(B) M-2 model (at 100 kg/cm² pressure)

Figure 8. Concrete strain distributions due to pressure.

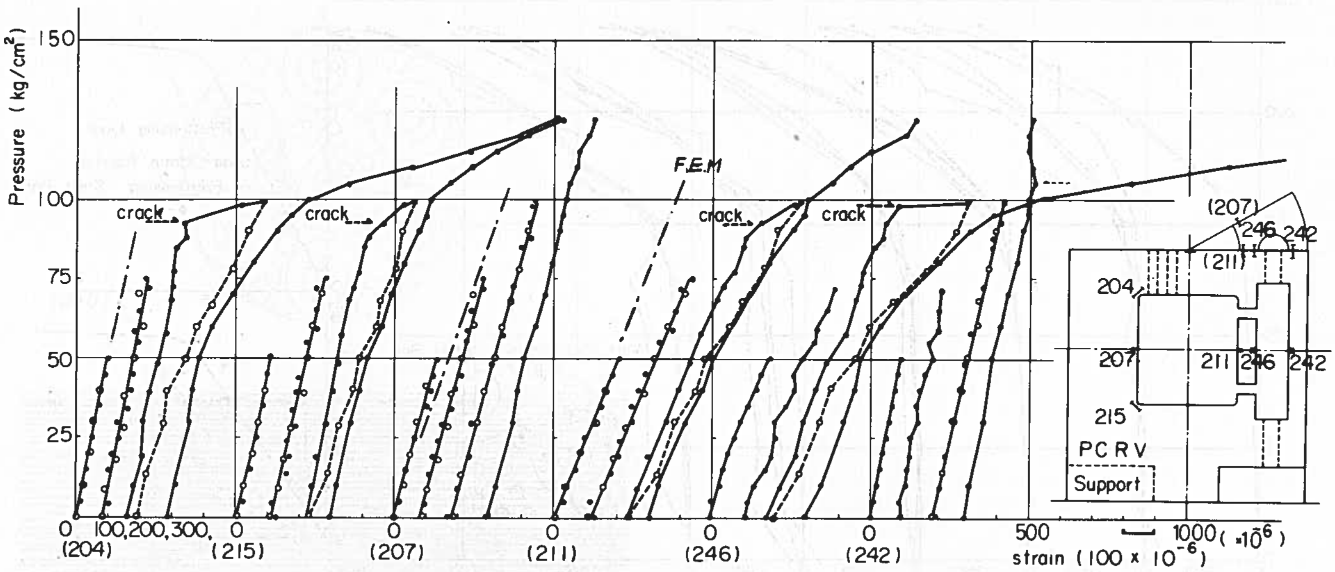


Figure 9. Typical concrete strains V. pressure in M-2 model

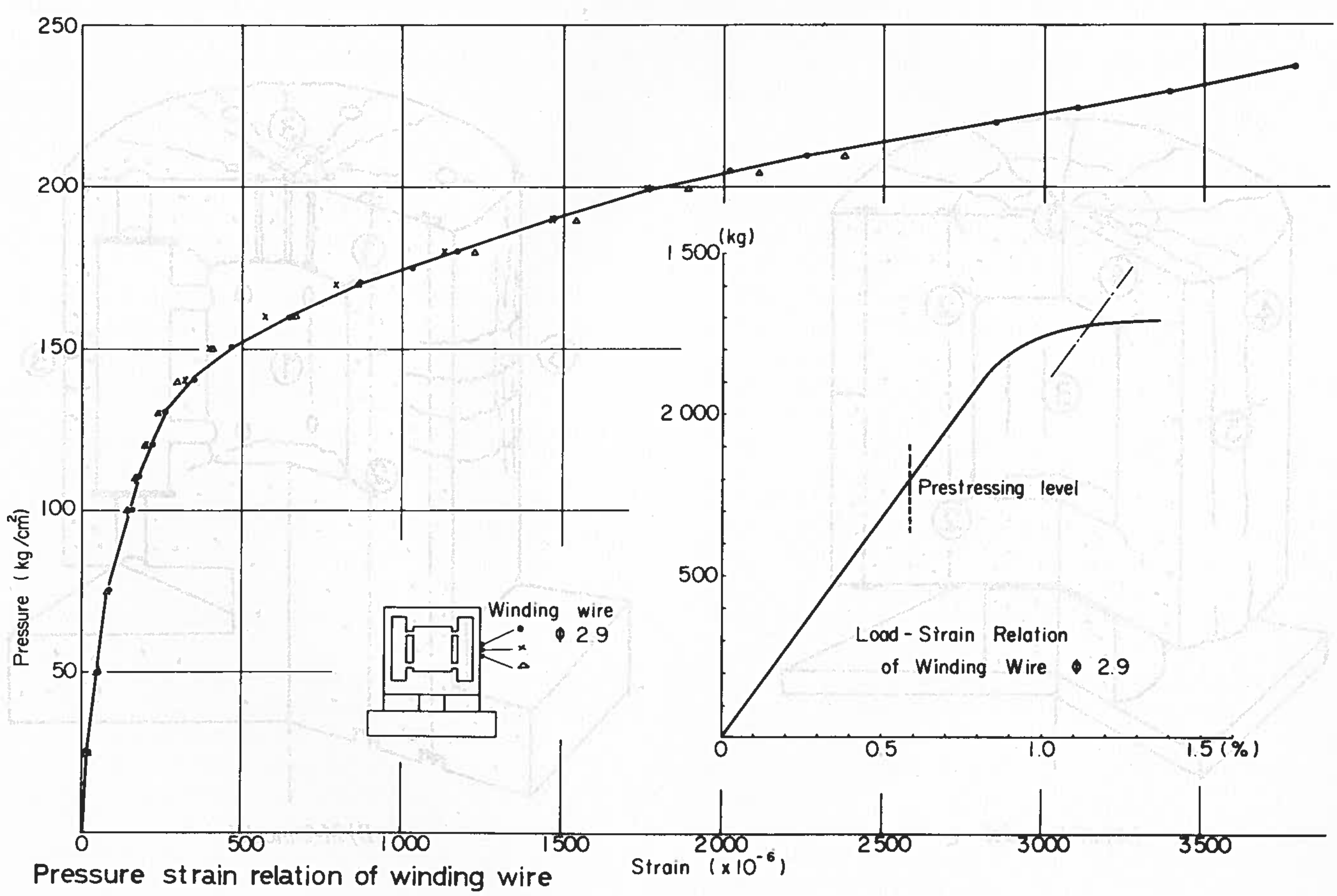


Figure 10a. Behavior of prestressing tendons of M-2 due to pressure.

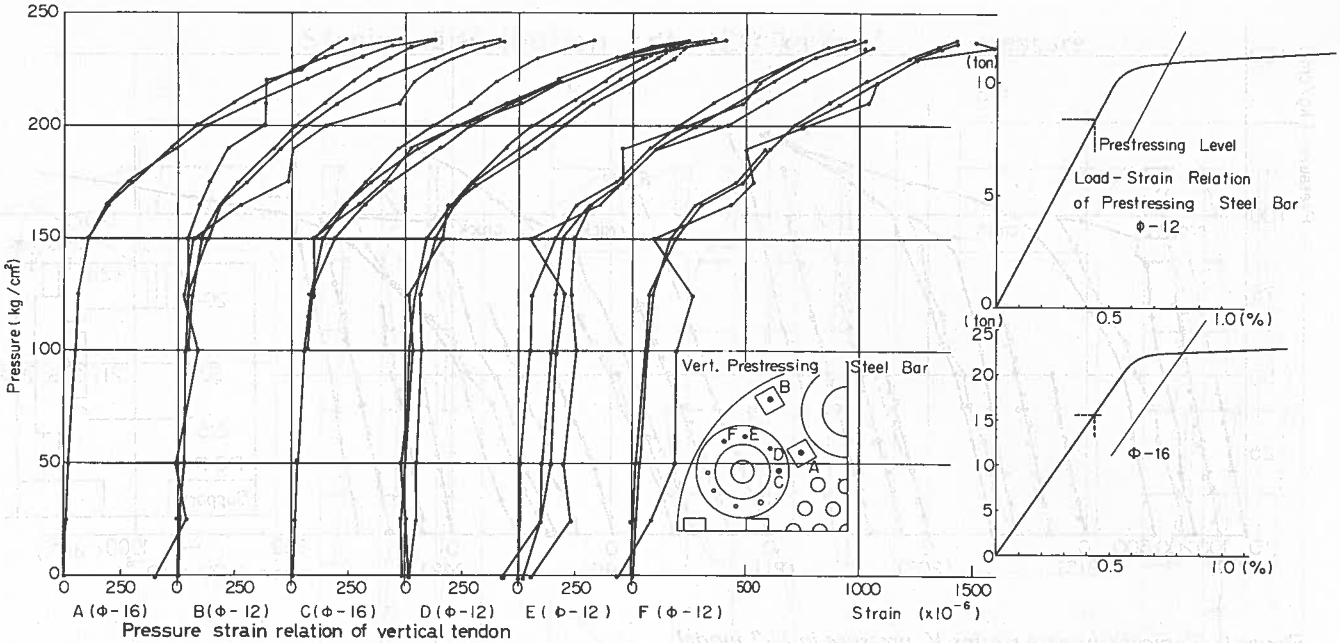


Figure 10b. Behavior of prestressing tendons of M-2 due to pressure.

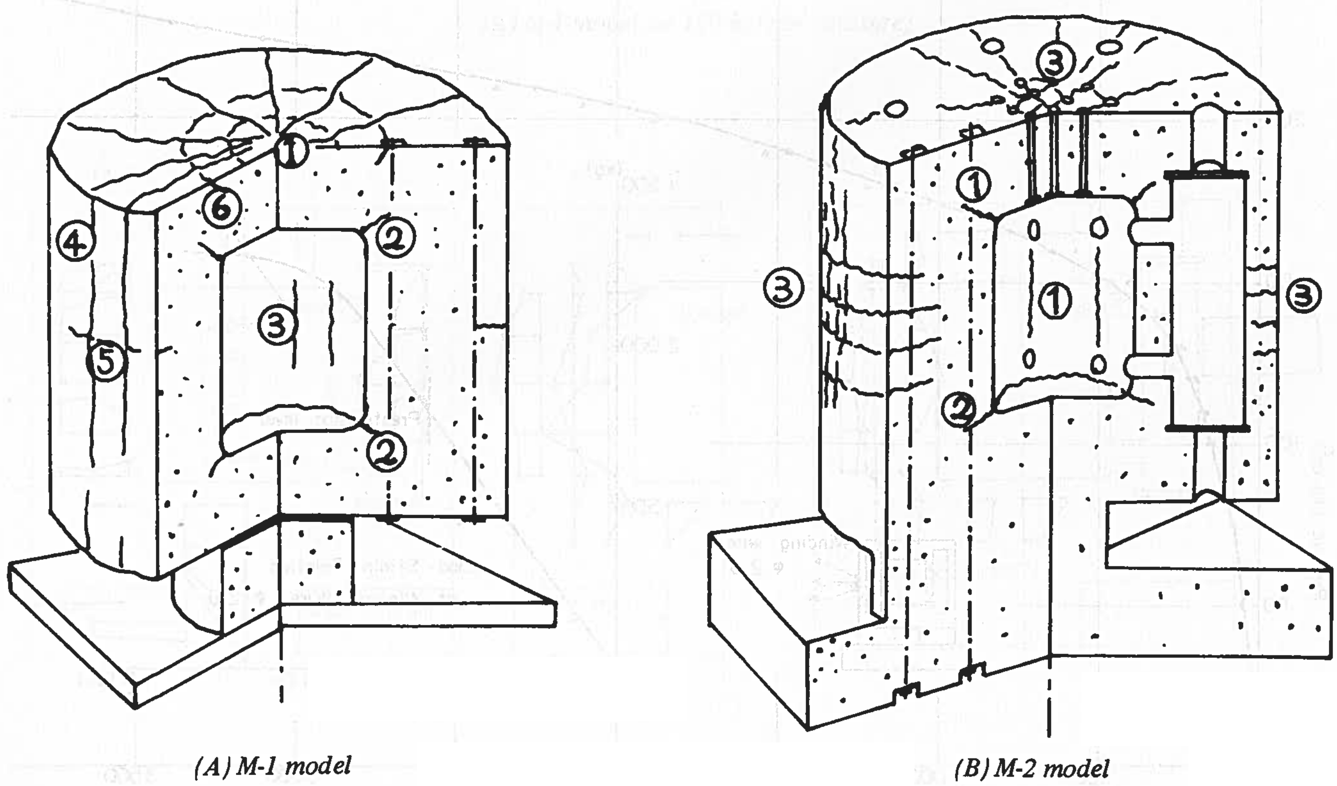


Figure 11. Schematic drawings of different type cracks in PCRV.

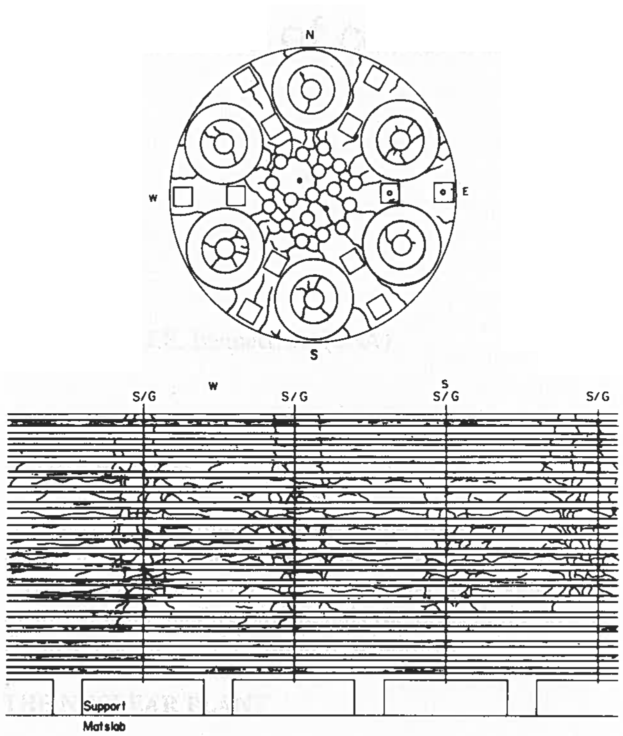


Figure 12. Final crack patterns of M-2 model.

Manufacture of concrete structures for a floating nuclear plant

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INTRODUCTION

A facility is being built in Jacksonville, Florida to manufacture floating nuclear power plants. This paper provides a brief description of the power plant and manufacturing facility, and details the process proposed for manufacture of the concrete structures.

THE NUCLEAR PLANT

The floating nuclear plant is a totally integrated nuclear power station mounted on a floating platform. The power plant is a conventional pressurized water reactor with ice condenser containment and a turbine-generator system with a net output of 1150 MWe. The pressurized water reactor, nuclear steam supply system, turbine generator, auxiliary systems and safeguards systems are based on existing land based technology. Figure 1 shows an isometric projection of the plant. The basic design approach has been to employ proven, accepted and licensed systems and apply them to the floating plant concept.

Concrete structures are provided in those areas where biological shielding is required. These include the containment structures and auxiliary areas housing potentially radioactive equipment (Figure 2).

The steel platform is approximately 400 ft long, 378 ft wide and 44 ft deep with a plant displacement of about 150 000 t. Full depth bulkheads run in both longitudinal and transverse directions extending the length and width of the platform to form a grid. The platform structure is basically a large rectangular steel box grid utilizing a plate and stiffener framing system of all-welded steel construction (Figure 3).

The floating nuclear plant manufacturing facility will comprise a substantially self-sufficient facility in Jacksonville, Florida, with deep water access to the Atlantic Ocean. The facility will be capital intensive rather than labour intensive within the framework of the present state of the art.

The facility is subdivided into three major areas: production shops to assemble and manufacture components and subassemblies, a graving dock and wet slip for final assembly of the floating nuclear plant, and outfitting and testing areas to complete non-nuclear testing of the plant. Support facilities, utilities and maintenance are provided throughout the facility. The facility layout presented in Figure 4 is a conceptual arrangement in which these elements are shown. Detailed engineering is now being performed to produce an optimal design of layout and processes.

The layout will encourage a production routine whereby the same job will be carried out in the same location on consecutive floating nuclear plants. The production routine is not rigid, however, and retains the flexibility needed to accommodate a job which may have fallen behind schedule with a minimum of confusion and redundant equipment and facilities. This situation represents a significant improvement over conventional land-sited nuclear plants from a standpoint of schedule and quality and management controls. Job repetition will facilitate the use of detailed process instructions geared to build quality into the manufacturing processes.

The production shops are designed so that raw material enters at one end and finished products exit at the other. Their location relative to the overall layout will permit finished products to be easily transported to the erection slip. Each shop will include areas and equipment necessary to ensure the effective administration of quality assurance and quality control programmes.

Final outfitting will be completed in a wet slip about 400 ft wide. The graving dock and mooring positions in the slip for the floating nuclear plant being assembled are divided into various stages. As a new hull is launched from the graving dock at one end of the slip, each floating nuclear plant in the slip advances one position. When a floating nuclear plant has gone through the stages of steel erection and major outfitting, it is moved to a vacant position along the seawall where it remains for the period of final outfitting and testing. There is adequate deep water clearance to manoeuver one floating nuclear plant around another moored at the seawall. An additional mooring area will be available along the seawall as an emergency hold position in the event that a unit cannot be immediately towed to the purchaser's site.

All major construction activities and heavy installations will be completed while the plant is in the wet slip. Crane service is provided by a heavy lift gantry crane. Whirler cranes will supplement the gantry. A large laydown area will be available on each side of the slip, under the gantry crane.

As a floating nuclear plant nears completion, it will move out of the slip, and be moored at the final outfit and test area. This position will provide an unhampered flow of circulating water, shore steam, condensate, and electrical power during all phases of functional testing. After final outfitting and the satisfactory completion of the functional testing, the floating nuclear plant will be towed to the site.

The concrete process flow scheme is designed to minimize interference with the many other operations underway alongside and aboard the platform. This includes reducing the number of men required aboard as well as the length of time needed to perform the particular concrete operation at that location. These objectives are achieved through the extensive use of form assemblies, use of self raising forms on the containment walls, precasting of concrete and conveying and placing concrete by concrete pumps.

In general a FNP will contain:

4000 yd^3 of concrete
580 000 ft^2 of form surface
5000 tons of reinforcing steel

75% of the concrete will be placed in the first three stages:

Stage 1	29.6%
Stage 2	27.6%
Stage 3	18.2%

In a steady state production rate of four FNP per year, this will require an average concrete production and placement of 80 yd^3 per hour for a 40 hour week.

The material yard will handle and spot over 100, 55-ton rail cars per week. Transit mix trucks will deliver concrete from the batch plant to the casting beds, assembly yard and to the concrete pumps along the slip.

Coarse aggregates will be received in rail cars carrying from 55 to 100 net tons. The rail cars will be unloaded by bottom dump into a below grade hopper. The unloading arrangement will have the capacity of unloading a car without moving it once it has been spotted. The aggregates will be washed, dewatered, and separated into four sizes and stockpiled into above ground silos by loading in the top by conveyor belt. The batch plant silo will be filled by drawing aggregates from the bottom of the silos and transporting by belt to the plant. The batch plant will be capable of rescreening the aggregates. No handling and mixing facility for heavy weight concrete is planned, as all concrete will be normal weight.

Fine aggregates will be handled the same as coarse aggregates except the material will not be screened or washed. Car shakers will be required. Fine aggregate may be received in two sizes, that is fine and coarse.

Cement will be received by either rail or barge. Barge shipment would be a partial shipment. Both the rail and/or barge containers will be unloaded by air using an air system and in the movement from the main silo to the plant silo the air system would be used to transport the cement. The cement will be transported in a 12 to 18 in. pipe.

Fly ash will be received either by rail or in 25 ton trucks. The unloading system will be similar to cement but a separate system.

Flake ice will be used in place of the mixing water in order for the batch plant to supply cool concrete to the forms in the summer time. The ice plant will be located in a separate building, adjacent to the batch plant. An ice storage room and a machinery room will be required to house the compressors. The storage of the ice is automatic with the ice makers dropping the ice into the ice storage bin. Automatic ice rakes in the ice bin level the ice and discharge the ice on demand into a screw conveyor which automatically feeds the ice batcher on the batching level. Ice will be batched in the normal manner as the other aggregates with the weight of the ice subtracted from the total mixing water. The major advantages of batching ice is that one pound of ice yields 150 times the refrigeration effects as one pound of water.

Concrete will be batched and mixed in a central batch plant. A wet holding hopper with two to four compartments will receive the concrete from the mixers. The concrete will be transported from the batch plant to the concrete pump by transit mix trucks. Concrete pumps will transport the concrete from the pump to the concrete placement onboard.

The batch plant will have the normal automatic recording controls. Communication facilities will be maintained between the discharging of the pump, the pump, and the batch plant.

Pump lines aboard the FNP will be cleaned out by blowing air and water backwards from the discharge end to the pump. Each pipe coupling will have a catch pan and the concrete pumps will be set in a catch pan arrangement. To clean the line, the pump end will be unloaded in a disposal or transit mix truck.

Cleanup water from concrete mixers both in the batch plant and the transit mix trucks will be dumped into a disposal area where the coarse aggregates will be reclaimed and dumped back into the aggregate system. The fines in settling bins will be reclaimed and used as needed in fill areas. The water will be treated sufficiently to reintroduce it into the water system of the facilities.

Reinforcing bars will be received either on barge or by rail. In either case the rebars will come in 60 ft lengths of 5 a ton bundle. The reinforcing bars will be off-loaded and separated by sizes in the storage areas in front of reinforcing shop. At present, #11 bars will be the largest rebar used in the construction design of the FNP. A 15 ton crane will be used to handle the rebar in the yard and shop. A dual line of shears and benders will be set up to fabricate the various sections. The reinforcing bars will be assembled in cages and lifted in the various preassembled forms, or individually placed in the form.

The form shop will subassemble the form sections for the concrete placements. The face of the panels will either be of plywood or steel depending upon the amount of blockouts, cut areas, and reuse. Some form panels may have many uses while other panels would be used only once or twice a FNP. Special forms such as self raising forms will be used on the containment and crane walls. The panel shop will be responsible for repairing, cleaning, maintaining and remodeling the forms. Manual handling will be held to a minimum.

The overhead crane will aid in the assembly of the various forms. Plywood panels will be used in areas of heavy penetration. The form surface will be treated to assure clean stripping. The form sections will be brought back to the cleanup area for cleaning and placed in storage. Unusual form assemblies of certain configurations will stay assembled and not be broken down into the various sections.

The carpenter shop will be a typical finish carpentry shop and no heavy equipment will be required in the shop. The carpentry shop will fabricate the makeup sections, replace the plywood panel faces, aid in the assembly of the form, aid in the setting of the form section units and in the locating and placing of the embedded parts. The embedments will be installed during the installation of the reinforcing or the assembly of the forms.

The work area for the power plant is extremely small (400 ft x 378 ft) consequently congestion of trade disciplines will become a major problem. The labor level involved in concrete can be substantially lowered by precasting and preassembling in the assembly yard. The crane capacity is 900 tons and whenever possible, precasting and pre-assembly of precast concrete units will be utilized in lieu of cast-in-place concrete.

In general, precasting affects a substantial savings in form-work cost, however, this method results in additional cost normally not associated with cast-in-place concrete. In many cases the additional cost offsets the savings resulting from precasting. Precast elements allow a more efficient section design and usually the quantity of concrete per section is less. In this case the purpose of the concrete is more for shielding rather than a structural requirement and thus the sections are large and any advantages of thinner units and more efficient sections are lost.

Concrete walls are labour-intensive components and as many walls as possible will be precast. Generally speaking, wall elements weighting up to the available transporting equipment capacity (100 tons) will be cast in the yard located immediately to the south of the construction site. A 100 ton straddle hoist will be used to transport the elements from the storage to under the 900 ton crane. The crane will be utilized to its full capacity by integrating individual precast wall elements into concrete assemblies. These assemblies will be fabricated in the concrete assembly area under the gantry crane and moved onboard. An example of a typical concrete assembly is the cellular structure forming the Waste Gas Decay Tanks. In this example, the walls will be cast flat in a casting area and transported to storage for 10 weeks for curing and to reduce shrinkage problems. At the end of this period, the units will be removed from storage and transported to the assembly platen and erected in an upright position; the connecting crosswalls will then be formed and cast as in a normal cast-in-place operation. The completed assembly, weighting up to 700-850 tons will be lifted onboard and placed by the gantry crane in one operation.

Concrete walls against steel bulkheads are integrated with the bulkhead to provide corrosion protection to the steel. This will be done by welding mechanical fasteners (Nelson studs or equivalent) to the steel bulkhead and precasting a portion of the wall thickness. The void between the steel wall and the precast wall will be filled with cast-in-place concrete. To ensure bond at the steel-concrete interface, the steel surface will be clean; at the precast/cast-in-place interface the precast surface will be roughened.

There are two categories of slabs; slabs resting directly on steel plates and elevated slabs supported by walls and beams.

Slabs on steel plates will be cast-in-place and made composite with the plate floor by the use of weld studs or equivalent. Thus, corrosion protection of the steel plate is ensured and the leakage of radioactive materials into the concrete-steel interface is eliminated.

Elevated slabs will be self-forming by the use of permanent steel or precast concrete forms. Total slab thicknesses will be achieved by pumping concrete in place on the permanent form. In some cases, the slab will be precast the total thickness, but notched at the supports to allow for a concrete strip to provide continuity. The lower 9 in. of the concrete containment dome will be precast and lifted onto the containment walls to provide a permanent form for the remaining concrete to be placed on the dome to bring it to the required thickness. This lift is around 900 tons.

CONCLUSION

The concrete operation has been planned to permit concrete structures to be built in a manufacturing environment. Special features are being incorporated in the design to simplify manufacture and permit the integration of the concrete with the other disciplines required in the manufacture of a nuclear power plant. These provisions will permit manufacture of a complete floating nuclear plant within the schedule of 17 months from keel laying to customer delivery.

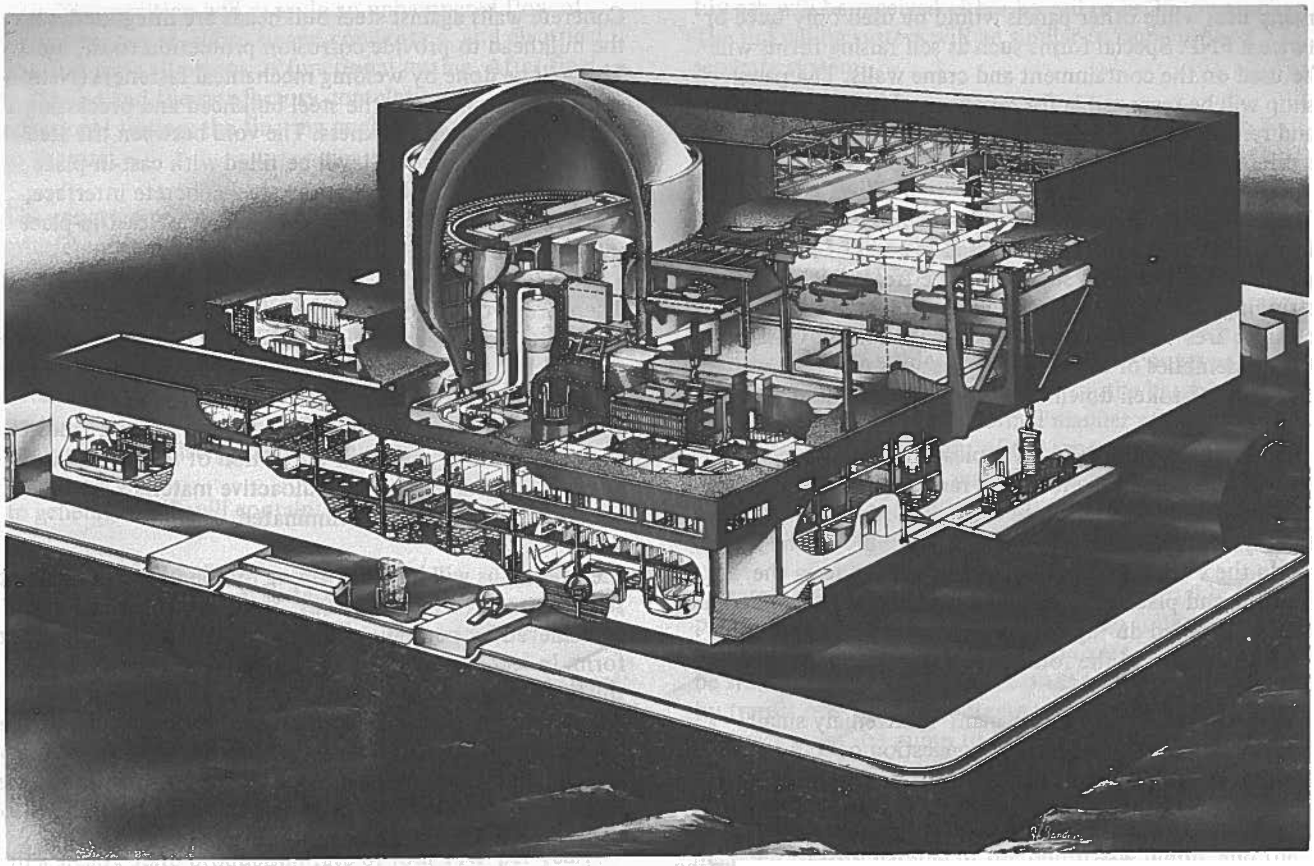

Figure 1. Isometric projection of the plant.

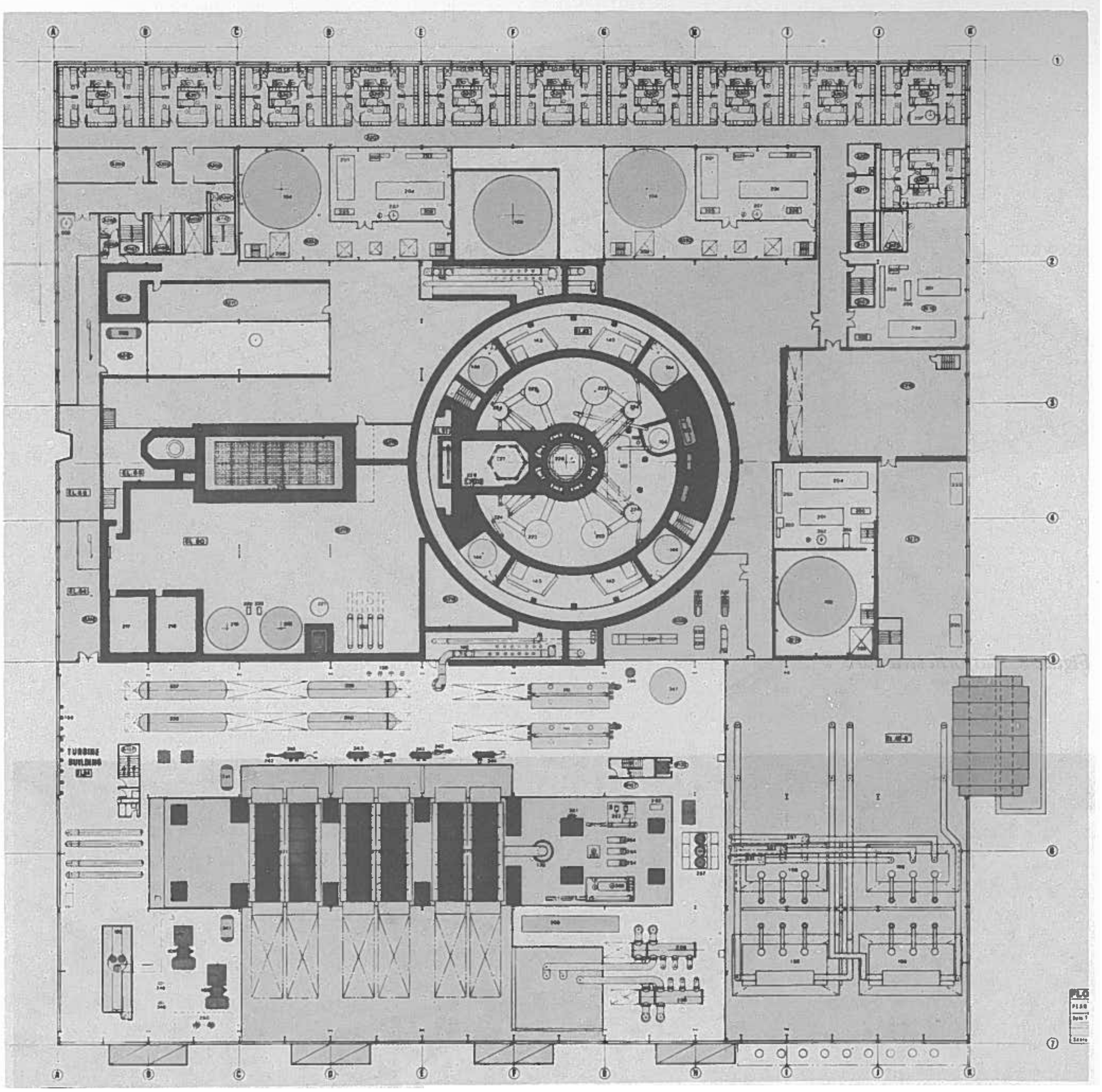

Figure 2. Plan view of the plant arrangement.

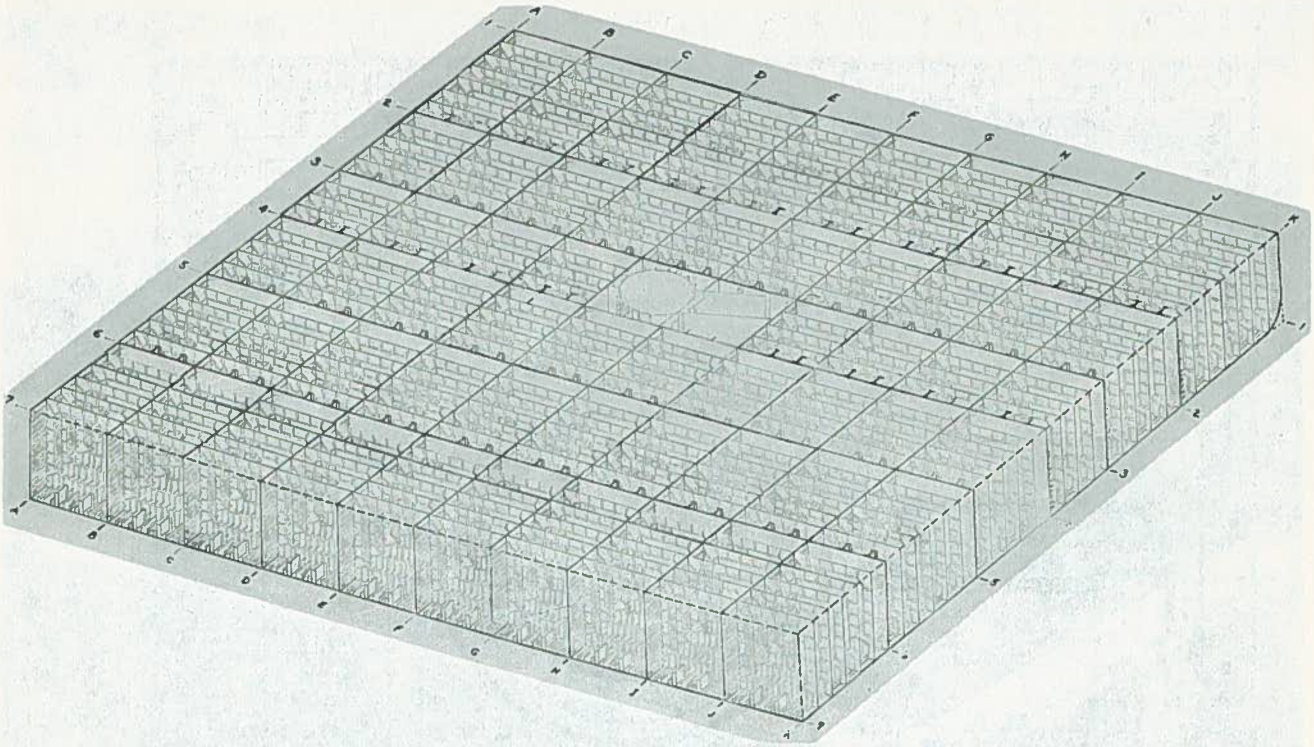

Figure 3. Platform structure.

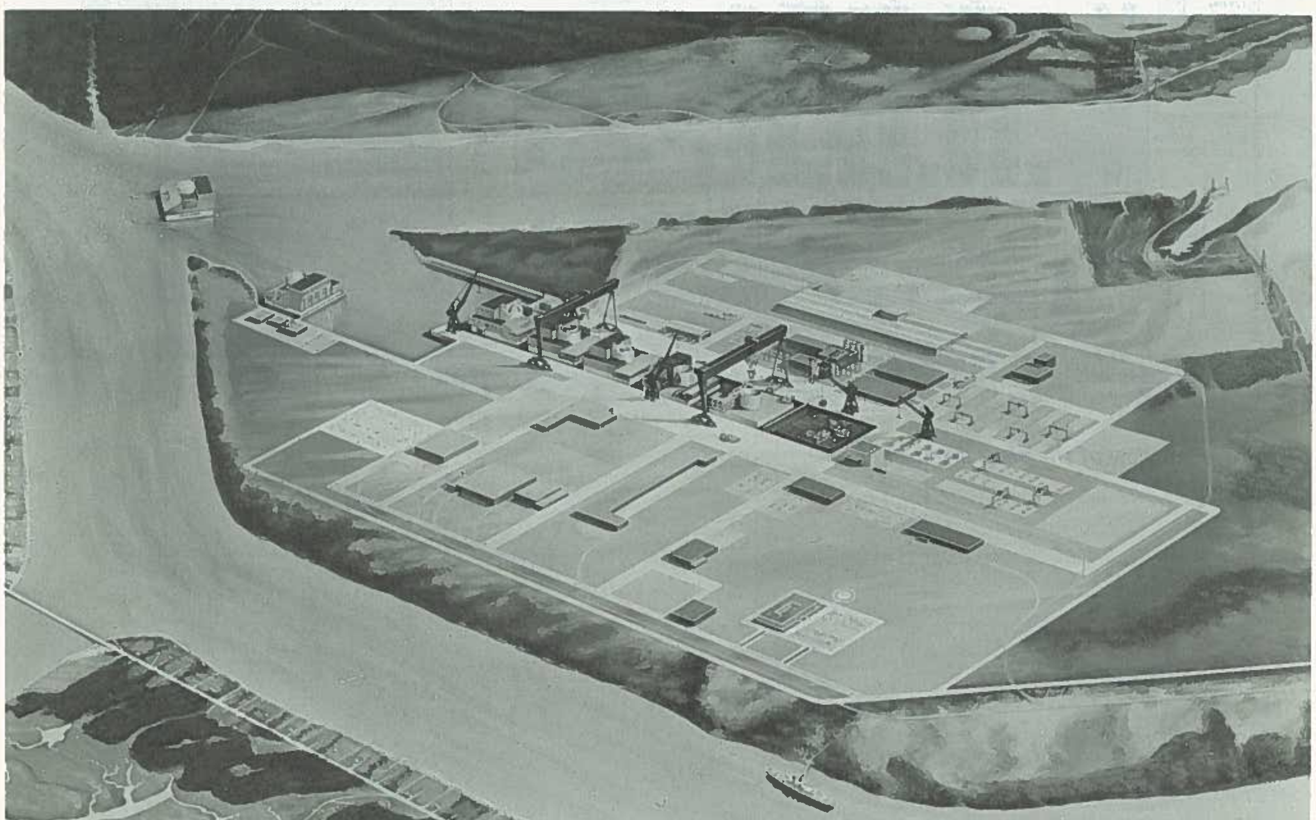

Figure 4. Facility layout.