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A REVIEW OF THE DESIGN AND CONSTRUCTION OF THE SIZEWELL 'B' PWR REACTOR BUILDING CONTAINMENT STRUCTURES

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ABSTRACT

The design and construction of the Sizewell 'B' primary containment has resulted in a structure different in certain aspects from that envisaged at the time when the decision was made for Sizewell to be based on the SNUPPS design which originated in the USA.

With the construction of the structure essentially complete, it is a suitable time to review the design of the structure and to ascertain whether significant improvements can be gained, in terms of cost and construction programme, for further projects based on the Sizewell design.

1. INTRODUCTION

Sizewell 'B' is the first PWR nuclear power station to be built in the UK. Construction of the station for client Nuclear Electric (NE) is being managed from within NE by the PWR Project Group; Nuclear Design Associates (NDA), are the design consultant responsible for the civil engineering works. The station design is based on the SNUPPS concept with the primary 4-loop circuit being of Westinghouse design with a gross output of 1260 MW. During the period of the design and construction of the power station, there have been a number of changes to the design requirements and to the form of the final containment structure (fig. 1).

Critical loading combinations are discussed and details of two of the significant design changes presented. Reference is made to the effect of construction conditions and programme on the design. Finally, areas where construction costs and programme could be eased for future projects are reviewed.

2. LOADING REQUIREMENTS AND COMBINATIONS

The Sizewell 'B' PWR primary containment was designed using an anglicised code which was based on ASME 1983 (ref. 1) with revisions up to and including summer 1985 Addenda. The load combinations in this code are similar to those in ASME with the addition of a requirement to check the ultimate capacity of the containment to withstand an internal pressure loading at ambient temperature to demonstrate an ultimate minimum safety factor of 2.0.

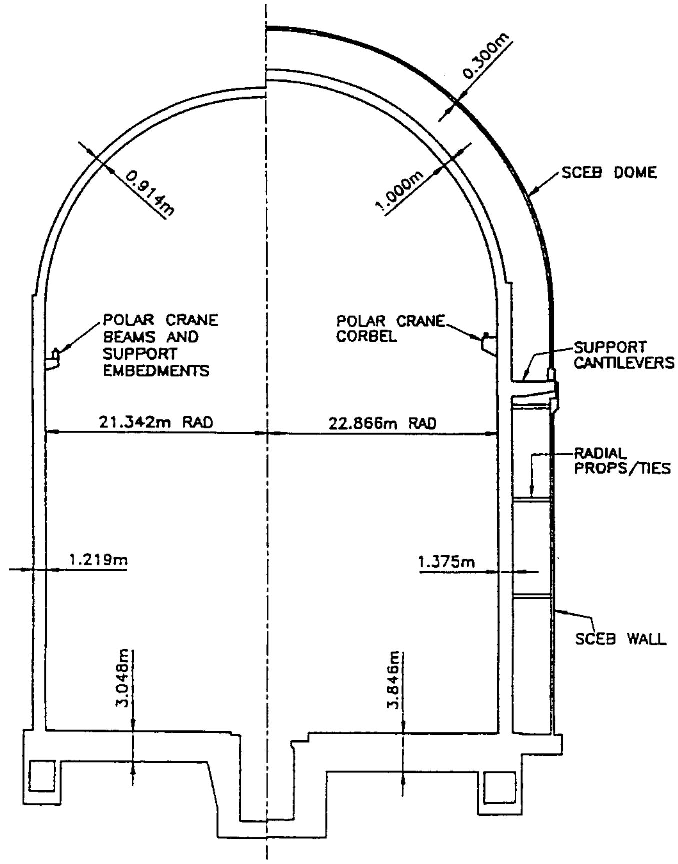

FIGURE 1

SECTION THROUGH CONTAINMENT STRUCTURE

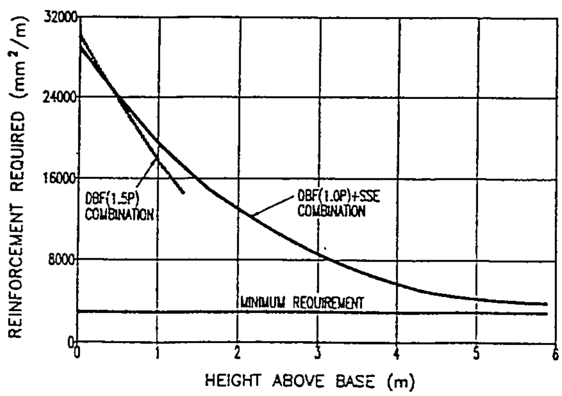

FIGURE 2

VERTICAL REINFORCEMENT REQUIRED IN CONTAINMENT WALL

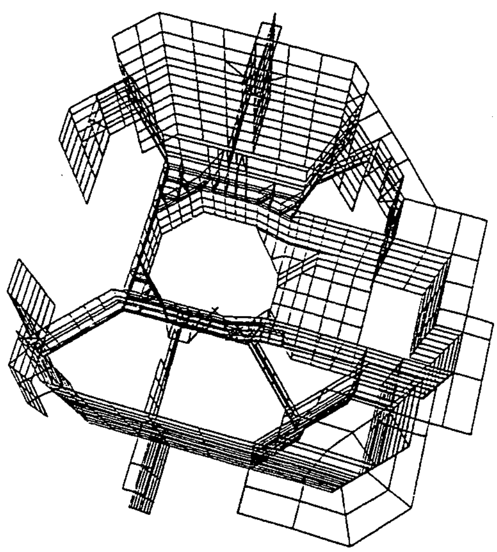

FIGURE 3

FINITE ELEMENT MODEL OF INTERNAL STRUCTURES

During the pre-construction period, when the design code was being developed, there was debate (ref. 2) about the rational of coupling the internal design pressure loading with the Safe Shutdown Earthquake (SSE) load case, which for Sizewell was specified as a maximum horizontal ground acceleration of 0.25g, a maximum vertical acceleration of 2/3 the horizontal, and UK soft site spectra. The Nuclear Installations Inspectorate (NII) the UK licencing authority, were of the opinion that the two load cases should be considered in combination, whereas the position of the Client, at that time called the Central Electricity Generating Board (CEGB), reflected the arguments for the decoupled case presented by Stevenson's paper (ref. 2).

Therefore, in response to a NII requirement to clarify the design feasibility of the reactor building to withstand the combined effects of the Design Basis Fault (DBF) pressure and SSE, an investigation was undertaken to determine the effect of coupling these load cases. This investigation showed that considering these load cases together had significant implications on the design of the containment but that the structure with the provision of additional reinforcement could accommodate the coupled loading condition.

For this investigation and for follow-on design purposes the seismic representation of the reactor building consisted of a beam and lumped mass model. A detailed examination of the results of the seismic analysis showed that fixed base results were higher than the soil structure interaction (SSI) results and that rocking of the containment structure due to the horizontal input dominated the response of the structure. This rocking was found to be sufficient to cause uplift under a portion of the base, which for the combined loading case, made the response of the base to the associated pressure loading more onerous. Design information was generated for the containment wall using a pseudo static analysis with a Fourier representation being used to apply the horizontal loading. The design of each section of the wall required the combined effects of bending, axial load and membrane shear to be examined and additional reinforcement requirements were established.

Finally an agreement was made between the CEGB and the NII to design Sizewell 'B' for the combined loading condition. This decision was made to enable the Sizewell design to proceed to programme and the CEGB reserved their position with respect to future stations. As predicted by the earlier investigation, the combination of these two load cases resulted in an increase in reinforcement quantities throughout the containment. In the barrel, for example, the minimum hoop reinforcement required at each face, 2888 mm^2/m height, was found to be sufficient for all load combinations with the exception of the coupled loading which required 3590 mm^2/m height at each face over the height of the cylindrical barrel. In the lower part of the barrel, the maximum vertical reinforcement at the wall base junction was dictated by the DBF combination with the 1.5 load factor on pressure. However, the curtailment of the reinforcement was controlled by the coupled SSE + LOCA load combination (fig. 2).

The ultimate load combination introduced for the UK containment requires a demonstration that the primary containment has the capacity to withstand a hypothetical internal pressure loading of at least twice the design pressure. This check was performed using a modified version of the non-linear program ADINA-81; the modifications being made to improve the modelling of concrete behaviour. Similar modifications have since been incorporated in a later version of the program, ADINA-87. As part of the validation of the use of this program for the analysis of the Sizewell 'B' containment, analyses were performed for the NRC commissioned 1/6th scale reinforced concrete containment that was built and tested at Sandia and for the 1/10th scale prestressed concrete model of the Sizewell containment that was built and tested in the UK (ref. 3).

Details of the final analyses and design of the containment are contained in the Design Report which has been issued to the NII. The 11 volumes of this report consists of a total of 52 individual reports addressing specific items, or areas of the containment. It provides a complete review of the comprehensive design exercise carried out on the primary containment and includes supporting reports such as concrete property tests, and civil and liner summary reports.

3. ANALYSIS OF THE PRIMARY CONTAINMENT BUILDING AND INTERNAL STRUCTURES

Three computer codes were used for the Finite Element analysis of the primary containment and internal structures:-

i) PAFEC for linear elastic static analysis
ii) NASTRAN for seismic analysis
and iii) ADINA for non-linear analysis.

Numerous models of various types were generated to investigate the general behaviour of the structure, or the local effects at discontinuities, load points or penetrations. Axisymmetric models were generally used to examine the overall behaviour of the containment structure under the full range of the specified loading conditions. In addition to stress output, for the design of the containment, displacement and strain outputs were obtained for use by the liner designer and by various suppliers of equipment located within the containment.

The final task for NDA on the Sizewell project is to re-analyse the structure, to obtain the predicted strains and deformations for the structural overpressure test, which is programmed for the end of August 1993. Unlike the design analyses, these analyses need to include for the stiffening effects of the liner and associated stiffeners, and the effects of reinforcement and the prestressing tendons. Results from longterm concrete tests will also be reviewed in order to produce properties applicable to the age of the concrete and the duration of the loading.

Figure 3 shows an example of one model used for the analysis of the internal structures. With the growth of computing power that has occurred over the period of the project, larger, more detailed models can now be used. This would enable the analysis to be concentrated on fewer models each providing information for the design of a larger volume of the structure. The internal structures proved to be particularly complex due to the numerous component loads that needed to be taken into account for the specified load combinations. The design code for these structures was an anglicised version of ACI 349.

4. SECONDARY CONTAINMENT ENCLOSURE BUILDING

The secondary containment enclosure building (SCEB) envelopes the parts of the primary containment that would otherwise be exposed to the environment. Together with the adjacent fuel and auxiliary buildings it forms the secondary containment which has the function of providing an additional barrier to any leakage originating from the primary containment and its penetrations. The incorporation of the SCEB was an enhancement to the original SNUPPS design.

The secondary containment is formed of four parts:-

i) a lightweight concrete dome supported by reinforced concrete slabs (figs. 4, 5 & 6) cantilevering out from the near top of the primary containment barrel
ii) a vertical reinforced concrete wall over approximately 1/3 of the circumference of the primary containment barrel
iii) reinforced concrete upstand walls on the roofs of the adjacent buildings
iv) enveloping buildings which are outside the structural description provided in this paper.

It was originally proposed that the SCEB would be constructed of a double layer of steel panels, the inner being the pressure seal and the outer providing weather protection and an architectural finish, supported off the surface of the primary containment by a structural steel grillage. However, the Contractor offered a saving on the tender price if the structure was changed to concrete and following a preliminary design exercise the structure in it's final form was shown to be feasible and cost beneficial to the project.

It was considered beneficial to keep the effects of the change on the primary containment to the minimum and hence the cantilever was divided into a number of individual slabs in order to reduce their hoop stiffening effect on the containment shell. Large gaps in the cantilever were also provided at each of the three buttresses to allow the permanent prestressing platforms to be used to provide access to hoop tendon anchorages.

The design of the SCEB was governed by the SSE load case. The loading on the structure was determined by performing a NASTRAN analysis using the model in figure 7. It was found that the loading was sufficient to cause uplift on the bearings between the cantilevers and the circumferential ringbeam at the bottom of the dome. Vertical prestressing bars, passing through the cantilevers and into the ringbeam, were incorporated in the design to provide an increase in the vertical reaction on the bearings and to ensure that uplift was prevented under seismic conditions.

The support conditions of the SCEB on the cantilevers were examined in detail to arrive at the most beneficial arrangements, with pinned, fixed and partially fixed being considered with respect to normal loadings and seismic response effects. The final result was the choice of a pinned arrangement utilizing prestressing bars and 'pot' bridge type bearings, the applied rotation between the top and bottom halves of each bearing being accommodated by a neoprene disk.

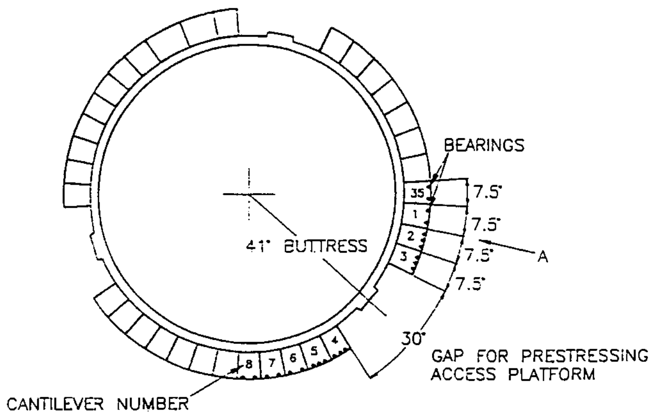

FIGURE 4

PLAN VIEW AT SCEB SUPPORT CANTILEVERS

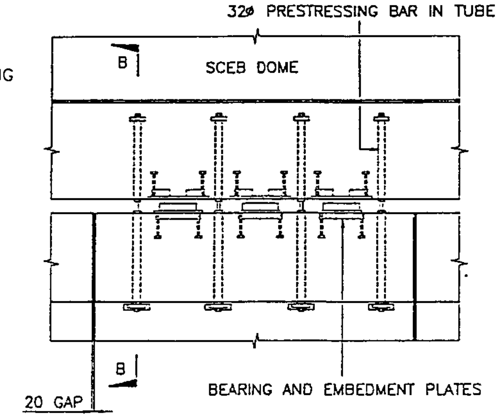

FIGURE 5

VIEW ON ARROW A SHOWING ARRANGEMENT OF BEARINGS AND PRESTRESSING BARS

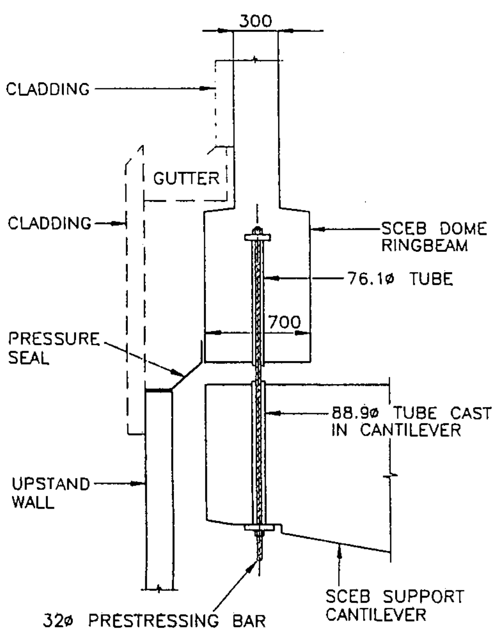

FIGURE 6

SECTION B–B

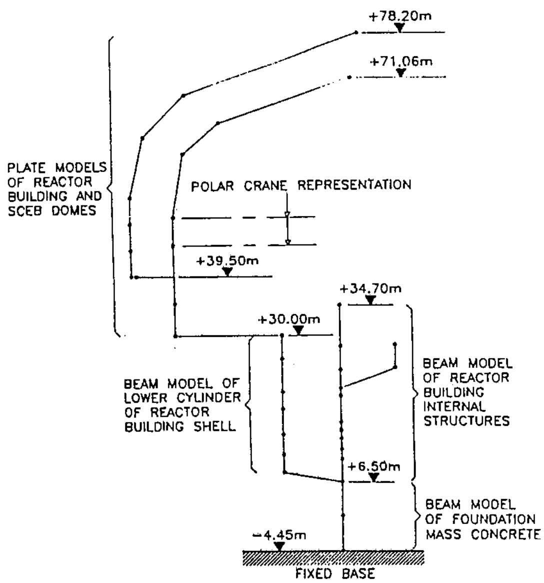

FIGURE 7

DIAGRAMATIC REPRESENTATION OF REACTOR BUILDING SEISMIC MODEL

The wall forming the lower part of the SCEB is restrained horizontally by a number of permanent radial props/ties spanning between the wall and the primary containment. It was divided into three vertical panels in order to reduce the hoop stiffness of the walls and the prop loads due to radial movements of the wall and the primary containment. Dividing the wall in this manner also assisted construction on site as the outer panels could be constructed prior to the closure of the construction opening in the primary containment wall. Flexible pressure seals were installed in the vertical gaps between the panels, and between the panels and the adjacent buildings.

A detailed NASTRAN model of the containments was produced to perform the seismic analysis. This model included a representation of each individual support cantilever with each bearing being represented by springs between the cantilevers and the SCEB dome. The loading on each component was therefore directly available from the analysis. For the design of the main reinforcement in the cantilevers the radial load, and the bending moments about horizontal and vertical axes all required consideration. The complete quadratic combination (CQC) method was used to combine the modal results rather than the square root sum of squares (SRSS) method, the SRSS method being unable to satisfactorily combine the modes with close frequencies.

Due to the programme requirement for dome construction and prestressing operations to occur in parallel, the dome could not be concreted in complete rings. Construction of the three dome areas adjacent to the buttresses could only be started once the stressing operations on the dome hoop tendons were completed as temporary works to these areas would prevent access to the hoop prestressing anchorages. Therefore the three parts of the dome above the cantilevers were constructed independently in advance of the infill panels, as was the top cap, which was the first part of the dome to be constructed. Analyses were performed on the part completed dome in order to determine built-in stresses and to identify potential problems with the proposed concreting sequence.

5. POLAR CRANE CORBELS

As SNUPPS it was originally intended to support the polar crane rail on thirty two structural steel beams supported by steel liner embedments. However, design changes were found necessary to the support arrangements due to the thermal and seismic loadings on these steel items. As a result the size and the weight of the liner embedments increased and problems were anticipated with their fabrication and installation at site. NDA proposed an alternative design for a continuous reinforced concrete corbel using through liner couplers to allow the reinforcing bars to be continued into the containment wall (fig. 8). The corbel was divided into a total of forty eight individual units in order to reduce the local increase in hoop stiffness in the containment wall, and the associated bending.

This change to the design required a number of unconventional design features to be addressed. Although the design of the corbel reinforcement was based on established principles the means of anchoring the reinforcement through the liner and the effect of lack of bond at the liner concrete interface required investigation. The reinforcement was anchored using the through liner couplers originally developed for connections through the containment bottom plate, however appropriate fatigue testing was carried out for the corbel application.

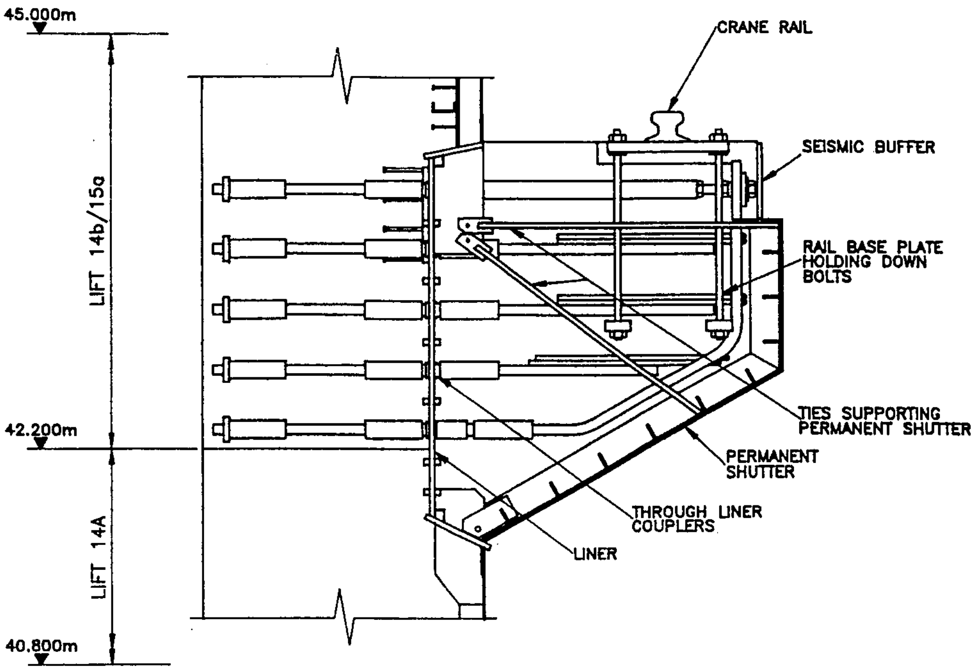

FIGURE 8

SECTION THROUGH POLAR CRANE CORBEL

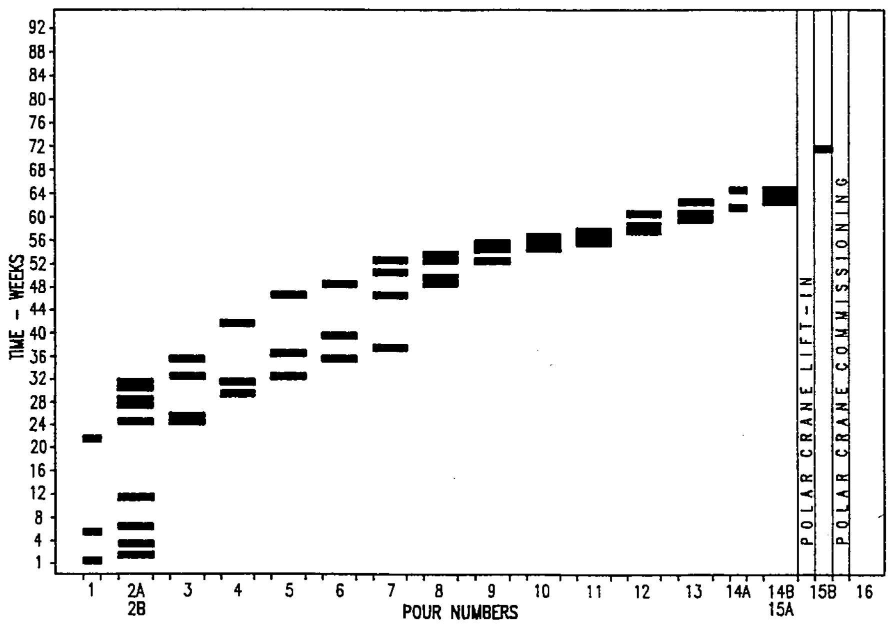

FIGURE 9

PROGRESS OF POURS IN CONTAINMENT BARREL

Permanent shutters were detailed for the corbels so that the need to erect falsework on the inside of the liner could be avoided, support being provided by the concrete of the containment wall. However in the event, falsework was used to allow the concreting of the corbels to take place prior to the concreting of the associated containment wall pours. Work to the inside of the liner, to install the crane rail, could then proceed in parallel with the work on the outside to complete the wall pours.

The details of the design and analysis of the corbels has been previously reported (ref. 4). In addition analyses were performed to determine the effects of the corbels on the general behaviour of the containment. A mixture of axisymmetric, Fourier and 3D analyses were performed to investigate a number of proposals to speed the construction of the corbels and the installation and commissioning of the polar crane. These analyses were complicated by the manner in which the concreting operations were being completed around the circumference of the wall, with some segments being completed in advance of others.

The unique nature of the concrete corbel prompted some concern from the licencing authorities who requested confirmation that the design was adequate and that the liner and containment were not jeopardised. Design reports were submitted and the acceptable behaviour of the corbels was confirmed as predicted by comparing theoretical movement and strain results with readings taken during the commissioning overload tests for the crane. Survey points were located above the corbel so that movements could be directly measured during the test. In addition strain readings were taken from instrumentation embedded in the containment wall in the vicinity of the corbel; this instrumentation being provided for the structural over pressure test of the containment. Both sets of readings confirmed that the behaviour of the corbel under polar crane loading was acceptable.

6. CONSTRUCTION PROGRAMME

At an early stage in the design process the importance of the interfaces between the civil and liner contractors was recognised and steps were taken to minimise potential problems. In the barrel it was recognised that the liner contractors work platforms could cause problems with the steel fixing operations below, and the decision was taken to detail the reinforcement in bands of 5.6m height (two lifts), rather than the maximum bar length of 12.0m. This eased reinforcement handling and allowed the fixing operations to follow as close to the liner works as possible. All joints between reinforcing bars in the containment wall and dome were made with CCL swaged couplers.

The progress of the containment concreting operations on site at Sizewell (fig. 9) clearly indicates the acceleration of the barrel construction with increasing height. This progress was dictated by the speed at which the reinforcement could be fixed in each pour. In the lower pours, where a high density of reinforcement was required, it was recognised that further delays could occur due to liner works in complex areas preventing the complete circumference of the wall being released to the civil contractor. The reinforcement was detailed to allow flexibility of working with some sectors being fixed in advance of others. This flexibility was extended to allow the contractor to pour concrete in a similar manner; the initial pours were inspected for signs of cracking near the vertical construction joints and any cracks found were monitored and were in fact only found to be thin and shallow. This uneven progress continued right to the top of the barrel after which concreting returned to complete rings.

Considerable care was also taken with the reinforcement detailing in the complex and congested primary containment barrel at the height of the SCEB support cantilevers and adjacent polar crane corbels. This effort minimised the reinforcement fixing problems and the programme date for the lift-in of the polar crane was successfully achieved.

7. DESIGN REVIEW FOR FUTURE PROJECTS

With the experience gained from the construction of Sizewell 'B' a number of areas can be identified where it is considered possible to achieve significant improvements in construction time and cost by making relatively minor changes to the geometry of the structure. For the civils work such savings have generally been identified with reductions in the areas of the main reinforcement mats and with the reduction, or removal, of the required shear reinforcement.

We are now investigating the possibility of making reductions in the reinforcement requirements in the following areas, which have been identified as being critical to speeding construction on future projects:-

i) the base including the reactor cavity and instrumentation tunnel
ii) the wall base junction
iii) the equipment access penetration
iv) the SCEB cantilevers
and v) the polar crane corbels.

As an example of reducing reinforcement complexity the base thickness is being examined in order to establish its effect on the requirements in the base, reactor cavity and instrumentation tunnel, stressing gallery and wall base junction. The examination includes taking account of any reductions in the quantities of main reinforcement and shear reinforcement, it also takes account of any detailing changes that can be made to ease the fixing of reinforcement on site.

The details of the polar crane corbel, and associated liner panels, are also being examined to assess the effect of making the corbels continuous. This could result in a reduction in the size and number of the stiffeners on the liner, the reinforcement in the corbel and the congestion in the containment wall caused by the radial reinforcing bars. These savings must be balanced against any increases in the reinforcement required in the wall caused by the additional bending due to the hoop stiffness of the corbel. Similar consideration will be given to the structural form of the SCEB cantilevers which it is recognised require optimisation for potential changes of loading criteria.

Based on the experience gained with the construction of Sizewell 'B' power station, NE is optimistic that the construction programme for future contracts could be shortened from the one adopted for Sizewell 'B'. Although the full effect of reducing the construction period for a particular activity may not be fully gained on site, due to other activities and interfaces becoming critical, such savings should be pursued as they will add confidence to the ability of maintaining the construction programme.

8. CONCLUSION

This paper provides a summary of the civil design features of the Sizewell 'B' containment. The success of this Project is essential to the continued development of the nuclear industry in the UK and it is believed that the Sizewell 'B' design has provided a proven model on which to base further station designs.

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