Civil engineering in the nuclear industry

Proceedings of the conference organized by the Institution of Civil Engineers and held in Windermere on 20-22 March 1991

Thomas Telford

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Opening address

J. COLLIER, Nuclear Electric plc

This is the first major conference specifically on civil engineering in the nuclear industry for at least 15 years. Given the major changes which have been taking place both institutionally and technically in various countries, such a conference is, I believe, timely if not overdue.

Civil engineering has an important contribution to make at every stage of the nuclear fuel cycle, from the choice of site and conception of the design of a major power station or fuel plan, through modifications during operation, to the final stages of designing and building waste management stores and repositories, and the decommissioning of stations and plants. Since the last conference the technical demands on designers and builders of nuclear plants have steadily increased. The clamour for ever greater safety requires a designer to consider and allow for the most unlikely events: natural events such as earthquakes or violent storms, as well as man-made diasters such as aircraft crashes. These pose fresh challenges for our ingenuity. One consequence of this strive for greater and greater safety and quality has inevitably been increased costs. The industry is responding to this challenge with new methods of construction, giving higher productivity and faster construction times: modular assemblies of re-bar, pre-fabricated culverts, all-weather construction methods, diaphragm walling, improved cranage, to mention but a few.

The challenge for the future must be to maintain the same high standards and quality but to build the stations and plants quicker and cheaper. This should not be a problem for civil engineers with their long history of achievement in providing the infrastructure of our civilization; such challenges are their bread and butter.

My responsibilities lie in power generation. In this area the most important single project in the UK, and the second largest civil engineering project of all after the Channel Tunnel, is the construction of the

PWR station at Sizewell on the Suffolk coast. Civil engineering has dominated the first three years of the project and I am happy to record that progress has been outstanding, if not spectacular. Indeed, construction is presently months ahead of the 72 month commitment programme.

There have been a number of notable achievements along the way. For example, the longest and deepest reinforced concrete diaphragm wall in Europe has been built to allow continuous dewatering on the site. The first immersed-tube pre-fabricated offshore cooling water culverts in England have been installed. At the end of February 1991 more than 75% of the required 408 000 cubic metres of reinforced concrete had been placed, with 78.5% of the required 71 000 tonnes of reinforcing steel.

The project has been planned to overlap civil with mechanical and electrical work, so that equipment and plant can be installed in clean, dry and prepared conditions at the earliest opportunity. This has only been possible through the close co-operation of the civil contractors and the project management team. When I visited the site I could see the gains made from such co-operation, with electrical equipment, pumps and pipework already installed in the lower levels of the buildings.

At Sellafield in Cumbria other major nuclear plants associated with the end of the fuel cycle are being completed and brought into operation. The massive £1.8 billion thermal oxide reprocessing plant (THORP) is now essentially physically complete, and will be commissioned for operation during the next couple of years. The Secretary of State for the Environment Michael Heseltine recently opened the high-level waste vitrification plant and store. Other plants for handling intermediate-level waste and for the treatment of low-level liquid wastes have been brought into commission in the last few years. These plants essentially complete the nuclear fuel cycle in the UK. They are operating here and now, and they give the lie to the jibe 'they don't know what to do with the radioactive waste'. We do know what to do with it, and we have invested no less than £4 billion to implement the strategies we have devised.

As far as the nuclear industry as a whole is concerned, I believe our objectives for the future must be to

(i) regain public confidence in the reliability and safety of our plants

(ii) build and commission future projects such as Sizewell B to tight programmes and cost

(iii) increase the generation from our existing plants, and reduce costs both for our present operations and for our future plants.

All should be aware of some of the contributions which nuclear power already makes to our country's economy and environment. Everyday in the UK nuclear power

(i) generates some 20% of our electricity worth more than £2 million

(ii) avoids the emission of around 100 000 tonnes of CO_2 and 2000 tonnes of airborne pollutants such as SO_2, NO_x, dust and fumes

(iii) saves the equivalent of around 200 000 barrels of oil, worth £2 million, and furthermore the nuclear industry spends £2 million each and every day with British 'high-tech' mechanical and civil engineering companies.

Overall Sizewell B and Sellafield - including THORP - provide a total of around 2000 permanent jobs and associated benefits in remote parts of the country, boosting the local economies around these sites. I am confident that these benefits will come to be recognized generally, and that this will occur sooner rather than later. I foresee a healthy future for civil engineering consultants and companies in the UK's nuclear industry. Specifically, they can look forward to being involved in

(i) planning with Nuclear Electric, up to 1994, for the nuclear power station options after Sizewell 'B'

(ii) the siting, design and construction of waste stores and repositories for low, medium and high level wastes

(iii) work arising from the decommissioning of the Magnox stations (Berkeley was shutdown permanently in 1988/89), requiring some innovative civil engineering solutions.

Outside the UK, opportunities are opening up in Eastern European countries; already German and French companies have made significant inroads into such markets. The predecessors of today's engineers helped lay the foundations of the industrial strength of this country, in the words of the charter of the Institution of Civil Engineers by 'directing the great sources of power in nature for the use and convenience of man'. I believe that the future of the nuclear industry continues to offer many and challenging opportunities to continue in this great tradition.

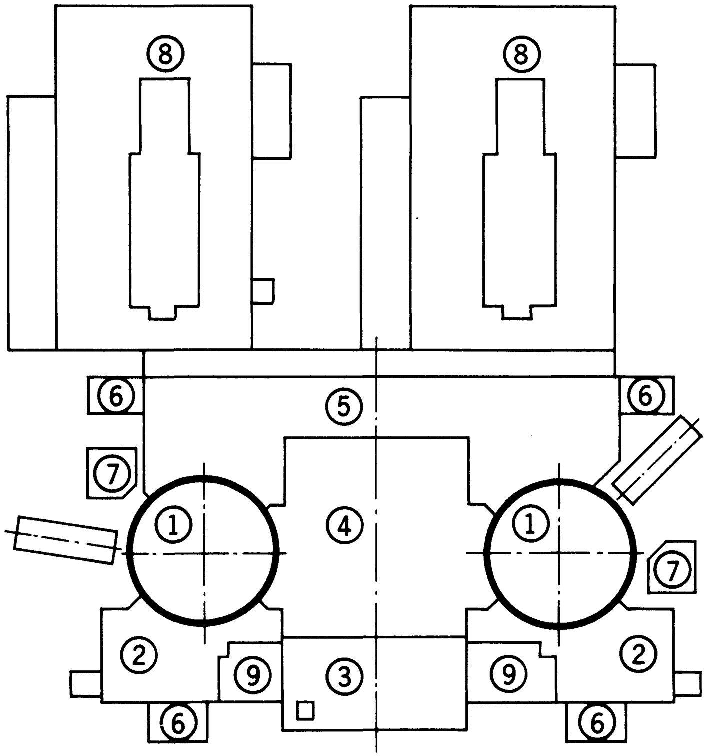
1. Design and construction of the French PWRs

J. L. COSTAZ, Civil Engineering Manager, EDF-Septen

SYNOPSIS. Thirty-four 900 MW PWR have been built in France since 1970 and they are now all in operation. The containment is a single wall prestressed structure with an internal metallic liner. The pressure tests in operation are made at the design pressure during the first refuelling and then every ten years. The first ten year test has been done on the FESSENHEIM and BUGEY containments. No aging effect could be found on the tightness. Shrinkage and creep are quite different from one site to another but the delayed phenomena are now stabilized.
The civil works of twenty-two 1300 MW PWR are now finished but only seventeen are in operation. The containment is a double wall without liner, the leaks through the internal wall being collected in the annular space permanently maintained slightly below atmospheric pressure.
Recent developments have been performed, especially the ways to improve the concrete tightness and the studies related to a high strengh concrete with silica fumes for the containment and cooling towers of the CIVAUX NPP.

1. Civil Engineering takes a very important part in a nuclear plant. In addition to its classical scope in an industrial site, its leading part in the safety gives its nobility title. This needs a high level of quality for the design and the construction that can be shown by the level of prices :
The main walls represent 15 % of the total cost of a PWR plant and their design represent 10 % of the construction price. This shows the large interest of the standardisation, the same design being used for a lot of units.
DIFFERENT STANDARDS FOR THE FRENCH PWRs
<u>900 MW</u>
2. After 2 prototypes in FESSENHEIM and 4 in BUGEY, the CP1 standard covered 18 units
4 in TRICASTIN (Rhone Valley)
6 in GRAVELINES (North Sea)
4 in DAMPIERRE (Loire Valley)
4 in BLAYAIS (Garonne mouth)
and the CP2 standard covered 10 units (Fig.1)
2 in St LAURENT (Loire Valley)
4 in CRUAS (Rhone Valley)
4 in CHINON (Loire Valley)

The main difference between these two standards is relative to the turbine hall and its turbo-generator. The nuclear islands are quite the same and this is especially true for the reactor building also called containment.



1 Reactor building

2 Fuel building

3 Cooling rooms

4 Nuclear auxiliary building

5 Electrical rooms

6 Diesel generator sets

7 Steam generator auxiliary feedwater storage tank

8 Turbine building

9 Refueling water storage tank

Figure 1. CP2 plot plan, 2 x 900 MWe

3. The containment structure is the most important one in a PWR plant. It is the third and last barrier to prevent the escape of radioactive products into the environment in the event of a major accident such as LOCA (loss of primary coolant). It also protects the nuclear sytem from external hazards such as aircraft impact, missile impact, chemical explosion or fire.
All the 900 MWe PWR containments consist of a single wall prestressed concrete structure with an internal metallic liner of 6 mm anchored in the concrete (Fig.3). Its design pressure is 0.4 MPa gauge and its tightness is given by its leak-rate : 0.3 % of the internal mass of air and steam during 24 hours.

4. The other nuclear island buildings surround the reactor building. (Fig.1)
The main buildings are :
- the fuel building,
- the nuclear auxiliaries building,
- the electrical and safeguard auxiliaries building.

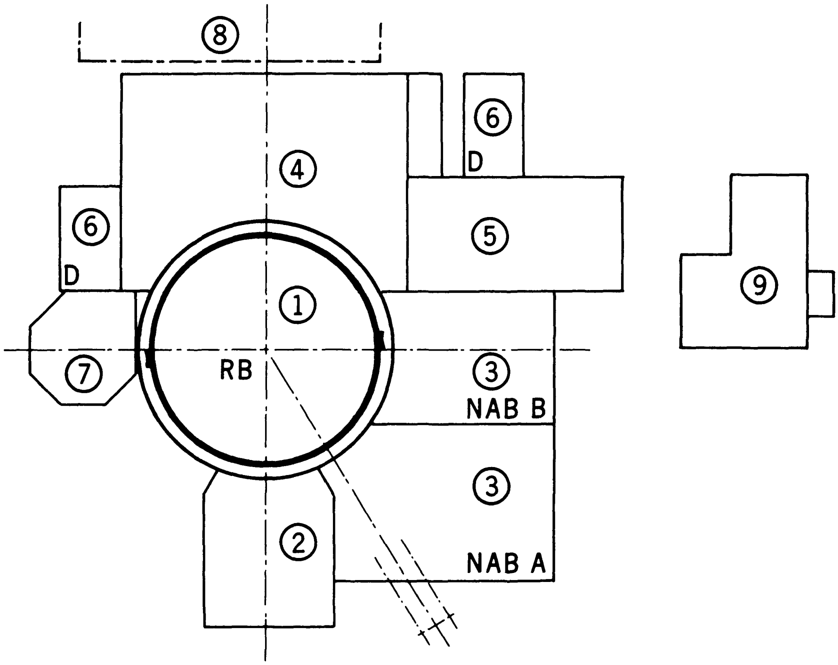
They do not have to withstand internal pressure but are subject to externally-generated hazards (plane crashes, explosions) and earthquakes. Hence these reinforced concrete buildings are stiffened by a large number of vertical walls which, with the base mat and floors, form highly rigid structures.
The turbine hall is not protected from externally-generated hazards. Assurance only has to be provided that it will not collapse in the event of an earthquake. This means it is a standard industrial building.
It houses a special structure : the turbo-generator set pedestal which must exhibit a very high rigidity in spite of its plane dimensions. For the CP2 standard, the table is put on springs in order to improve the control of vibrations.

<u>1300 and 1400 MWe</u>

5. The large number of units in a same 900 MW standard showed the advantages and disadvantages of this situation, the major advantage being relative to the design cost, the major disadvantage being not to benefit of the experience of the first plants.
For the next series of 1300-1400 MW plants, there can be found 3 standards.
- the P4 standard : (1300 MWe)
    - 4 units in PALUEL (Channel)
    - 2 units in St ALBAN (Rhone Valley)
    - 2 units in FLAMANVILLE (Channel)
- the P'4 standard (1300 MWe)
    - 4 units in CATTENOM (Moselle valley)
    - 2 units in NOGENT (Seine valley)
    - 2 units in BELLEVILLE (Loire valley)
    - 2 units in PENLY (Channel)
    - 2 units in GOLFECH (Garonne valley)
- the N4 standard (1400 MWe) (Fig.2)
    - 2 units in CHOOZ (Meuse valley)
    - 2 units in CIVAUX (Vienne valley)

The P'4 standard was decided to lower the cost of the P4 one by lowering the size of buildings and taking into account the actual earthquake level (0.1 or 0.15 g).
The N4 standard was required due to changes in the nuclear system.

6. Since PALUEL, the construction of which began in 1977, the containment consists of a double walled concrete structure (Fig.3)
- the inner prestressed wall without metallic liner provides pressure strength and tightness. The design pressure is 0.38 MPa gauge for P4, 0.42 MPa gauge for P'4 and 0.43 MPa gauge for N4,



1 Reactor building
2 Fuel building
3 Nuclear auxiliary building
4 Safeguard & switchgear building
5 Access building
6 Diesels
7 Refueling water storage tank
8 Turbine hall
9 Waste auxiliary building

Figure 2. N4 nuclear island plot plan, 1400 MWe

- the outer reinforced wall provides protection against external missiles and hazards and creates an annular space permanently maintained slightly below the atmospheric pressure in order to collect the leaks through the inner wall in case of LOCA.

The containments of 900 and 1300 MW are quite different. Although the strengh is provided by prestress in both cases, the tightness is provided by a steel liner for 900 MW and by drainage for 1300 MW. Each of these two containments meets fully with the requirements of the safety authorities but the second type is better against external missiles and hazards.

Their prices are quite the same if calculated for the same internal volume and pressure.

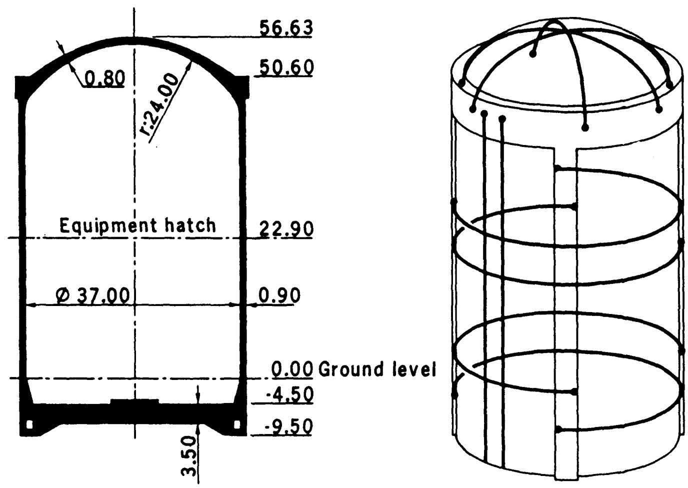
Table 1 - Main characteristics of containments

	900 MW PWR CP1 - CP2	1300 MW PWR P4		1300 MW PWR P'4	1400 MW PWR N4
	TRICASTIN Type	PALUEL Type		CATTENOM Type	CHOOZ Type
		inner wall	outer wall	inner wall	inner wall
Inside diameter (m)	37	45	50.85	43.80	43.8
Total internal height (m)	60.38	65.95	71.95	61.485	62.885
Wall thickness (m)					
- raft	3.5	3.0	-	2.8	3.0
- cylinder	0.9	0.9	0.55	1.20	1.20
- dome	0.8	0.95	0.40	0.90	0.9
Steel liner (mm)	6	-	-	-	-
Total internal vol. (m3)	63,000	106,000		85,216	87,325
Accident gauge pressure (MPa gauge)	0.4	0.38		0.42	0.43

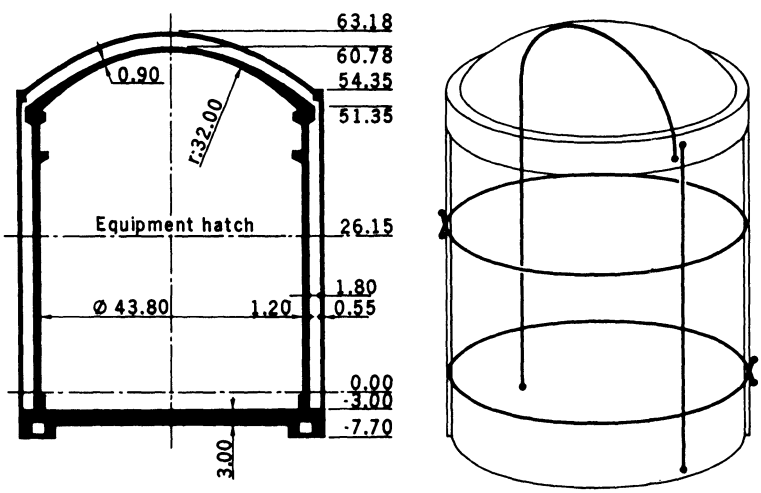
7. In summary, the main construction work of a 1400 MW unit is characterized by the following quantities :
- 200,000 m^3 of concrete
- 300,000 m^2 of formwork
- 15,000 t of reinforcing bars
- 1,500 t of prestressing tendons
- 1,000 t of embedded parts
- 6,000 t of structural steelwork

PRESTRESSING OF CONTAINMENTS

8. In all units the raft is reinforced concrete (with some cables to prevent cracks developping in P4 and P'4) and the containment shell and dome is prestressed. The main prestressing characteristics for 4 typical units, TRICASTIN, PALUEL, CATTENOM and CHOOZ are given below (table 2).



PWR 900 MWe



PWR 1400 MWe

Figure 3. Cross-section and prestressing layout

The strand protection is always cement grouting and the cable type changes from 19 T 15 in TRICASTIN to 37 T 15 in PALUEL. Vertical and dome cables are tubes whereas horizontal are semi-rigid ducts.
The dome prestressing changes from a triangular grid (900 MW and P4) to a square grid (P'4 and N4) thus enabling a reduction of the dome thickness (seismic improvement) and also by means of turning 2/3 of the vertical cables into the dome a reduction of the number of dome anchors, which always prove difficult to cope with.

Table 2 - Prestressing characteristics

	TRICASTIN CP1 PW 900	PALUEL P4 PW 1300	CATTENOM P'4 PW 1300	CHOOZ N4 PW 1300
Type of cables	19 T 15 FREYSSINET	37 T 15 FREYSSINET	37 T 15 FREYSSINET	37 T 16 FREYSSINET
Number of ribs	4	2	2	2
Number of horizontal cables	274 (3/4 turn)	133 (full turn)	118 (full turn)	136 (full turn)
Number of vertical cables	212	180 (120 turned in dome)	155 (99 turned in dome)	172 (116 turned in dome)
Number of dome cables	162 (triangular grid)	120 (triangular grid)	116 (square grid)	116 (square grid)

PWR CONTAINMENT DESIGN

General

9. The design of structures for the earliest PWR has been performed with French standard rules for reinforced concrete (CCBA68) or prestressed concrete (IP65) : these rules were based on the allowable stresses method.
The design of structures for the last PWR has been performed with new French standard rules for reinforced concrete (BAEL) or prestressed concrete (BPEL). These rules are based on the limit state methods.
All specific values and load combinations are defined in the EDF document known as RCCG (Rules for design, construction and testing of the civil works for the French PWR power plants). These documents have been revised to take into account the more recent developments.
The general dimensionning is not fundamentally different in either method, but the recent regulations give a better comprehension of the structure behaviour and a better estimate of the security factor and are homogeneous with the future European rules for concrete structures. (Eurocode 2)

Combination of the main actions

10. The main elementary actions have been taken into account in the design :
- the permanent actions :
  . dead load, creep, shrinkage, permanent thermal effect (G)
  . prestressing (F).
- the variable actions :
  . loads during construction (QC),
  . maximum service load (Q),
  . mean service load (Q'),
  . pressure test (Qpe),
  . wind (W),
  . snow (S),
  . variable thermal effect (T),
  . earthquake 1/2 sse (0.5 E).
- the accidental actions :
  . loss of coolant accident (Fp),
  . earthquake SSE (E).

11. Three types of combination of actions are to be taken into account :
- the basic combinations for the check of limit state resistance and shape stability with normal security factors,
- the accidental combinations for the check of limit state resistance and shape stability with reduced security factors,
- the combinations of service limit state for the check of the compression in concrete and the crack opening.

An example is given hereafter for the 1400 MW N4 containment.

12. The following basic combinations are checked with the security factors 1.5 for the concrete and 1.15 for the reinforcement :

$$(1.35 \text{ or } 1)\, G + P + (1.5 \text{ or } 1.35)\, Q1 + \Sigma 1.3\, \Psi_{0i}\, Qi$$

For the basic action (Q1), one of the following :
- Qc load during the construction,
- Q maximum service load,
- Qpe pressure test
-0.5 E earthquake 1/2 SSE

with the other actions (Qi) (climatic actions, mean service load, temperature).

13. The following service limit state combinations are checked for the concrete compression and for the crack opening :

$$G + P + F + \Sigma\, \Psi_i\, Qi$$

For the basic accidental action (F), one of the following :
- Fp LOCA,
- E Earthquake SSE,
- E + Fp

with the other actions (Qi) (climatic actions, mean service load, temperature).

14. The following service limit state combinations are checked for the concrete compression and for the crack opening :

$$G + P + Q1 + \Sigma \Psi_{0i}\, Qi$$

For the basic action (Q1), one of the following :
- Qc loads during the construction,
- G maximum service load,
- 0.5 E earthquake 1/2 SSE,
- Qpe pressure test,

with the other actions (Qi) (climatic actions, mean service load, temperature). The cracks are considered as prejudiciable for the inner face of the containment and it is verified that during LOCA. pressure test or 1/2 SSE average minimum concrete stress is a compression higher that 1 MPa and that in case LOCA + SSE, average minimum concrete stress is not a tension through any standard section.

Main computer calculation

15. The main dimensions of the containment are analysed in the preliminary design. At this stage, the designer makes a set of choices as to the prestressing cable type and general lay-out.

Several finite element models are used to check the design and to determine the reinforcement in all particular areas.

16. The main computer calculations are the following :
- seismic analysis, including soil-structure interaction,
- raft calculation using different soil moduli values for the determination of envelope forces,
- general calculation of cylinder including, precise prestressing forces, ribs and main penetration,
- calculation of dome including different construction and prestressing phases.

Numerous local calculations are also performed.

PWR CONTAINMENT INSTRUMENTATION

Description of monitoring systems

17. The containment structures are instrumented to check their behaviour during construction (especially during cable tensioning), during pressure tests (after the construction and during the first refuelling and then every ten years) and for the whole life of the power station.

Surveillance devices have been designed to check local strains and global displacements of the containment. They consist essentially of :
- vibrating wire strain-gauges which measure local strains in the raft, the cylinder and the dome,
- thermocouples combined with strain-gauges for thermal correction (one thermocouple for two strain-gauges),
- pendulums which measure horizontal displacements according to four generating lines at three different levels,
- topographic levelling in the foundation slab gallery,
- dynamometers on four vertical tendons.

The strain-gauges are set in pairs, near outer and inner surfaces in current sections of the containment. The number of strain gauges is small in the containment of a standard unit (24 strain-gauges), but their implantation is the same for all the containments of a given series. We can thus compare the values obtained on units belonging to different sites. Each first unit of a site is fitted with additional strain-gauges in the raft and in the gusset. Each first unit on a series is fitted with additional strain-gauges in non standard sections as main hatches, prestressing ribs, dome bolt.

Analysis of measurement

18. The measurements are given for the earliest units of 900 MW PWR for more than ten years and the first ten year test has been done on the Fessenheim and Bugey containments.

For the different units with the same geometrical and stress characteristics, the results obtained differ only from the concrete characteristics. In particular, the Young's modulus, the shrinkage and the creep are quite different from one site to another.

An example of a set of data obtained on a pair of horizontal strain-gauges, in typical section of containment is given hereafter.

The results during cable tensionning (from two to four months) give rapid strain variation (400 microns/m for the horizontal strain) in which the elastic and delayed phenomena are combined.

After the tensionning and before the first pressure test (500 days), the strain variations (100 microns/m) include only delayed phenomena (shrinkage and creep).

During the first pressure test, it is check that the strain variation according to pressure is linear and reversible. This elastic modulus without delayed phenomena.

After the first pressure test, the strain variation (200 microns/m) continues with only delayed phenomena during six years (2200 days). After the delayed phenomena are practically stabilised.

The pressure tests during the first refuelling and after ten years give no significant difference with the first test, the elastic modulus is the same.

The elastic moduli have been determined for each containment from the results of pressure tests and range from 28800 MPa (Dampierre 1) to 36000 MPa (Tricastin 1).

Comparison with the theoretical values

19. The design of containment has been performed with standard rules for prestressed concrete (IP65 for the earliest 900 MW PWR and BPEL83 for the last 1400 MW PWR).

In the design one value of elastic modulus and one estimation of strains due to shrinkage and creep at 40 years was used.

The estimated Young's modulus used in the design calculation in agreement with regulations is 40000 MPa and differs from the modulus which has been provided by measurements during the pressure test.

If we use standard French BPEL83 rules for prestressed concrete design in which the variation of delayed phenomena acccording the time are estimated by analytic formulas, measured strains values exceed theoretical values.

If we compare the values which are given by measurements for cumulated effect of shrinkage and creep to the estimates of these delayed phenomena taken into account when these containments have been designed, we find that the ratio between measured strain variations at 10 years and design strain values at 40 years is about 75 % to 95 % instead of 80 % with the analytic formulas.

Depending on the effective rate of creep and shrinkage from now to the 40 years life of the plant, the effective strains should be at least the design values and perhaps higher in the range of 10 % to 15 %.

Prudence would lead us in future evaluations for new containment to sligthly underestimate the theoretical Young's modulus which is the major cause of difference between measurements and regulations.

It must also be kept in mind that other margins included in containment design are not taken into account in our comparisons and as the curves indicate that delayed strains are almost stabilised, we can conclude that the measurements are on the whole in good agreement with the design estimates.

COOLING TOWERS

20. Atmospheric cooling towers are generally shells similar to a hyperbolic torus, resting on a large number of diagonals linked by an annular footing which transmits loads to the ground. However the last shells at GOLFECH, CHOOZ and TIHANGE 1 (Belgium) are supported by radial columns. This allows a larger area for the air flow.

These structures, with a height reaching 180 m and a diameter at the base of about 140 m have a thickness of only 20 cm in the straight section of the shell which is, relatively speaking, much thinner than an egg-shell. (Fig.4).

They are subjected to a variety of loadings :

- weight
- differential settlements
- wind
- thermal effects :
  . gradient across concrete thickness due to inner air/outlet air difference,
  . seasonal temperature variations,
  . sunshine (facing shadow/facing sun),
  . concrete shrinkage
- physical/chemical and biochemical hazards (ice, run-off water, algae...).

Shell distortion is regularly monitored, by topographic monitoring and visual inspection, for any variation with time. Monitoring of the 20 shells already built for NPP's has shown that they have all exhibited some defects, these defects (mainly cracks) are preferentially located in the top and bottom sectors of the shell (i.e in the thickest parts).

RECENT DEVELOPMENTS

21. High strengh concrete (HSC)

High strengh concrete has not yet been used in any NPP in the world. The major reasons are probably the reduction of all the nuclear programs and the need of experience of this new material in non nuclear structures.

Nevertheless, we think HSC ought to be used in the future nuclear plants because of its following main qualities :

- water, air and steam tightness
- high resistance
- good beahaviour with corrosive water
- low creep and shrinkage
- acceptable price.

This new material with a water/cement ratio of about 0.4 and admixtures as silica fumes could be used principally for containments and cooling towers. A study is being done for the CIVAUX NPP.

22. Container for low and medium level radwastes (Fig.5)

A new type of container has been designed to replace the old ones with lead protection. It uses heavy concrete (hematite) with different thicknesses. The plug is also made of heavy concrete. A steel liner makes the tightness of this container sure.

23. Fire doors

Civil Engineering is not only limited to the concrete structures. For instance, fire doors are a very important equipment for safety. Up to CHOOZ NPP, french PWR plants used current industrial fire doors but feed-back experience showed

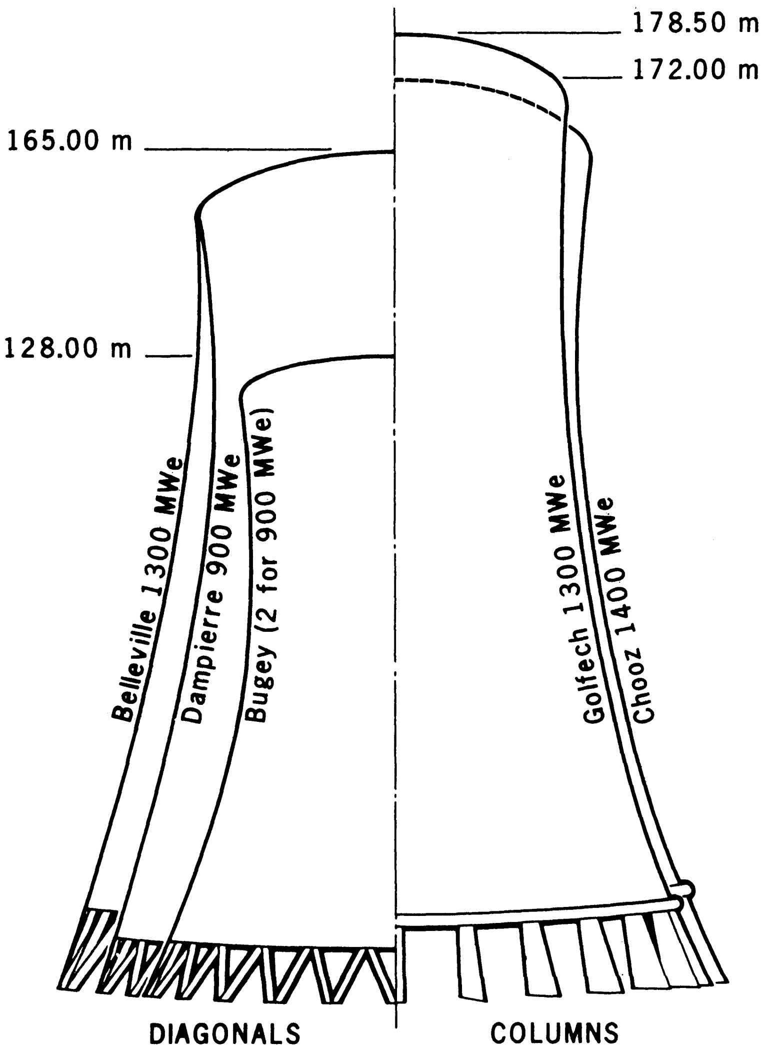


Figure 4. Cooling towers evolution

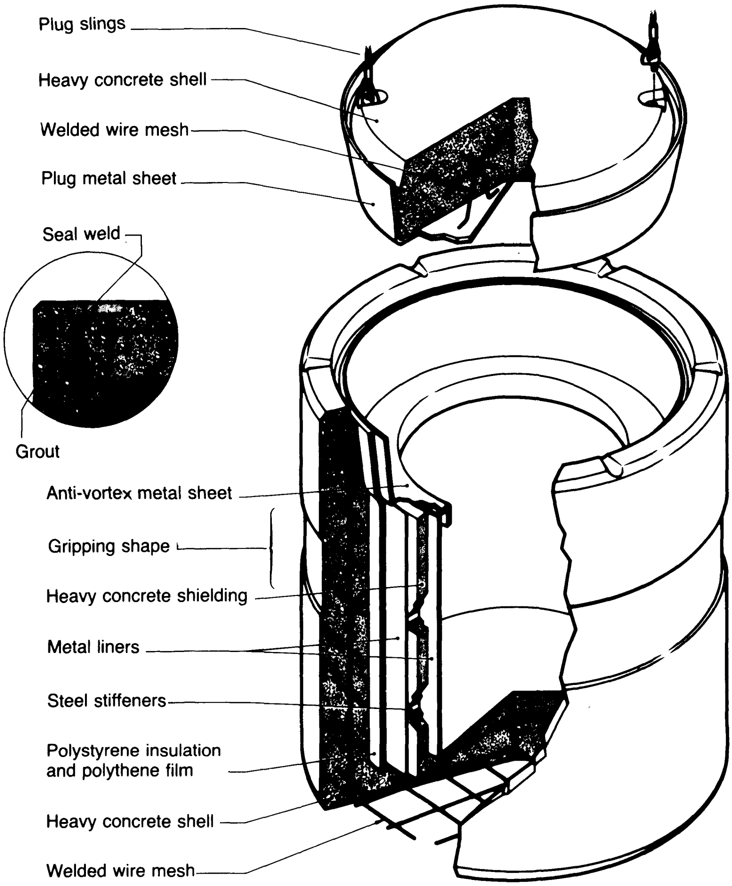


Figure 5. Container for low and medium level radwastes

they were too brittle. New fire doors have been designed by EDF using DURASTEEL fire resistant sheet and they have been chosen for the new N4 standard.
A new type with composite material is also tested and could be a good competitor.

CONCLUSION

24. If we consider the whole cost of a N4 unit close to 8 10^9F (without the first fuel), the civil engineering part is about 25 to 30 % and the main walls close to 15 %.
On the 900 MW standards in which four units were made with very short delays, a total of 4 000 workers were on the site at the same time.
Nowadays for the N4 1400 standard, the delay is longer and a maximum of 950 workers has been seen in CHOOZ for two units.
Activities are very diversified, complex or rustic, prototype or standard.
200 contractors and sub-contractors are on the site. A high level of coordination is necessary to manage the technical cohesion without forgetting human problems.

3. Repository design strategy

P. ISGAR, EurIng, BSc, FIStructE, MICE, MIHT, and A. J. HOOPER, BSc, UK Nirex Ltd

SYNOPSIS. United Kingdom Nirex Ltd is responsible for developing a national repository for the disposal, deep underground, of low and intermediate level radioactive waste. Conceptual design studies have been undertaken in the period from 1987 to present. These have examined the various geological environments and constraints for the underground works, along with generic site layouts for the surface facilities. More recently the site selection process has led the Company to examine possible sites at Sellafield and Dounreay. Repository design during 1990 and 1991 is addressing site specific issues which are fundamental to the legislation and regulatory requirements relevant to the development of a repository at one of these sites.

Introduction

1. United Kingdom Nirex Ltd is responsible for developing a national repository for disposal, deep underground, of low and intermediate level radioactive waste.

2. Conceptual design studies were undertaken, during the period from 1987/1989, that examined the various geological environments and constraints for the underground works, along with generic site layouts for the surface facilities.

3. More recently, the site selection process (Refs 1,2) has enabled the Company to look in detail at possible sites at Sellafield and Dounreay.

4. This paper will describe the following:-

- Nirex organisation and interfaces with advisory bodies.
- An overview of site selection and site investigation.
- The waste expected at the repository.
- The current design strategy for the sites at Sellafield and Dounreay.
- The relationship between Design and Operational Safety.
- How cognizance of extreme environmental hazards and abnormal loading are addressed looking specifically at seismic, extremes of wind and temperature, and

potential impact within the repository.

- Describe how Quality Assurance in design, construction and operation is of paramount importance.

United Kingdom Nirex Ltd Organisational Aims and Interfaces with Advisory Bodies

5. United Kingdom Nirex Ltd - The Nuclear Industry Radioactive Waste Executive was set up with the Government's agreement in 1982 by four organisations, working in partnership:

British Nuclear Fuels plc
Central Electricity Generating Board (Nuclear Electric)
South of Scotland Electricity Board (Scottish Nuclear)
UK Atomic Energy Authority (AEA Technology)

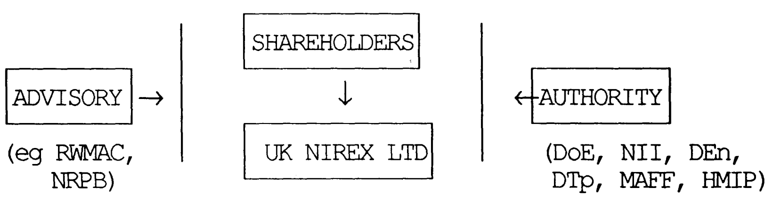
6. United Kingdom Nirex Ltd was constituted as a Company in November 1985, with all shares held by the partner organisations and by the Department of Energy on behalf of the Government.

7. United Kingdom Nirex Ltd's task is to implement the Government's strategy for the disposal of most low and intermediate-level radioactive waste produced by the UK nuclear industry and by users of radioactive materials in hospitals, industry, research and defence.

8. The current aims of this strategy are to ensure that:

a) wastes are safely stored, pending decisions on final disposal,
b) disposal facilities are developed for low and intermediate-level wastes that are at least as safe as supervised storage,
c) disposal decisions are based on considerations of the best practicable environmental options.

9. The following indicates the relationship of United Kingdom Nirex Ltd with Advisory Bodies, Government Authorities, and the Nuclear Industry.



Site Selection

10. International Atomic Energy Agency Recommendations and Safety Guides (Refs 3,4) demonstrate an idealised sequence of activities for site selection. Table 6.1 from Ref 3 explains more clearly.

11. Although each site must be considered on its own merits, it is possible to list some of the general factors governing the suitability of a repository:

(a) It should be possible to characterise the properties of the host rock in the vicinity of the repository to such an extent that the performance of the repository can be effectively predicted;
(b) The hydrogeological characteristics of the host rock and the groundwater regime of the surrounding geological environment should favour waste isolation;
(c) It would be an advantage if the host rock and/or the geological barriers can be used to retard the migration of radionuclides;
(d) The repository should be located at sufficient depth in the host rock so that the wastes will not be exposed to the biosphere until the radionuclides have decayed to insignificant levels;
(e) The repository should be constructed so as not to endanger the hydrogeological isolation of the wastes;
(f) The acceptability of a geological formation should also be based on the extent of its occurrence and its economic value.

12. Wastes containing significant amounts of radionuclides with long half-lives should be placed in formations for which a continuing regional geological stability is predicted.

13. The site selection should be undertaken in close connection with the work for the repository concept and design, and if necessary the introduction of engineered barriers should be taken into account.

14. In taking cognizance of these considerations the project has been developed to its present stage as described in Refs 1 and 2.

15. United Kingdom Nirex Ltd has followed the approach recommended by the International Atomic Energy Agency (IAEA) (Ref 3) in seeking to identify a preferred site for a waste repository. This requires locations to be evaluated initially on the basis of geological and environmental information and on societal considerations. Furthermore, the IAEA recommends that an evaluation should proceed in stages from generic to specific site assessments carried out in progressively increasing detail, the number of candidates being reduced as the requirements to be satisfied are refined and enhanced.

16. Essentially, three stages are involved:

- a search on a national scale to define favourable areas of the country; followed by;
- the identification of specific candidate sites for comparative evaluation and the selection of outstanding prospects for physical exploration to

confirm their suitability; and,
- the final choice based on the results of geological investigation and other studies.

17. The site selection process undertaken by United Kingdom Nirex Ltd began with 'desk studies' based on available data and theoretical knowledge. As the characterisation of favourable areas and of potential sites proceeded, some were eliminated and others emerged as offering potential. Overall, the convergence on a small number of outstanding candidates has enabled increasingly detailed appraisals to be undertaken of relevant matters specific to each site. The use of this procedure has progressively reduced the number of sites to be carried forward for consideration. The next step in the process of selection has been to embark on physical exploration, including both field drilling to provide cores for laboratory studies and geophysical testing and to gain hydrogeological data.

18. Simultaneously, enhanced socio-economic, planning, environmental, and transport studies are being carried out.

19. While the site selection process was founded on the approach described above, United Kingdom Nirex Ltd also sought to explore the broader social context in which it is required to discharge its technical responsibilities. Consequently, the decision was taken to stimulate an extensive dialogue with all interested parties by publishing an account of its proposals. This publication, entitled 'The Way Forward', was launched in November 1987 before members of the House of Commons, the House of Lords and the European Parliament.

20. In order to ensure that the general public was made aware of 'The Way Forward' copies were sent to all local authorities, county associations, parish and town councils, country and district libraries and to hundreds of organisations with a potential interest in the subject. National advertising was undertaken and more than 50,000 copies of 'The Way Forward' were distributed.

21. The University of East Anglia's School of Environmental Studies was contracted to analyse all the responses received. In its report, the University noted a widespread public objection to sub-seabed disposal with off-shore access and a general preference for avoiding areas of high amenity value such as areas of outstanding natural beauty.

22. While the safety of transport and disposal of radioactive waste was judged to be extremely important, there was no overall consensus of how this might best be achieved.

23. The deep, permanent disposal of low-level and intermediate-level radioactive wastes where they could be monitored and could be recovered, did seem to be broadly acceptable to a majority of those bodies and organisations.

in the United Kingdom with national rather than parochial roles and responsibilities.

24. Those organisations and individuals, with local rather than national interests, that expressed a preference, generally considered on-site storage more acceptable than immediate disposal. This 'Not in my Backyard' reaction, whilst predictable, contributed little of further substance to the debate.

25. The only areas of the country that showed a measure of support were Caithness (Scotland) and Copeland (Cumbria), which were already familiar with the nuclear industry developments at Dounreay and Sellafield respectively.

26. As indicated earlier, United Kingdom Nirex Ltd took the views expressed by interested parties from outside the nuclear industry into account in selecting sites. In a number of respects the responses coincided with the views of United Kingdom Nirex Ltd. For example, areas of high population were to be avoided, as were areas of national importance in terms of amenity or scientific interest. United Kingdom Nirex Ltd also recognised the need for long-term monitoring of the waste providing this does not prejudice the performance of the repository with respect to safety. The view that small islands and offshore sites should be avoided was not in conflict with the current views of United Kingdom Nirex Ltd.

27. United Kingdom Nirex Ltd noted the comments about on-site storage of wastes and has carried out a comparison between this and the early disposal option. United Kingdom Nirex Ltd is also aware of the transportation benefits that would accrue from siting a repository close to one of the major sources of waste arisings.

28. As the site selection work progressed in parallel with the discussion process it became clear that the various sites under consideration could be divided into two categories: those where there is a measure of support for nuclear activities in the local community and those where there is not. In view of this, and recognising the practical difficulties of investigating sites in parallel, Nirex has decided as a first step to limit further investigations to two areas where there is a measure of public support, namely, Dounreay in Caithness, and Sellafield in Cumbria. In doing so United Kingdom Nirex Ltd does not rule out the possibility of investigating other locations at a later stage or of utilising the sub-seabed options.

29. The selection of Dounreay and Sellafield for further investigation used the best data available in the areas of safety, transport, geology, design, planning and conservation. United Kingdom Nirex Ltd is currently undertaking the fuller evaluation of these two locations to determine whether either, or both, are suitable for construction of a repository or whether other locations should be similarly evaluated. This will be accomplished in

three general ways:

(a) by further site characterisation, including geological investigation;

(b) by using site specific information to develop preliminary designs and to enhance the post-closure safety analysis;

(c) by direct consultation with local authorities and other bodies to improve the understanding of socio-economic factors, nature conservation and transport considerations.

30. The Sieving Process for Site Selection is as follows:

GEOLOGICAL SURVEY BY BGS
INLAND AND OFF-SHORE AREAS WITH SUITABLE GEOLOGICAL AND HYDROGEOLOGICAL CHARACTERISTICS

ENVIRONMENTAL AND PLANNING CONSIDERATIONS
1. POPULATION DENSITY LESS THAN 5 PERSONS PER HECTARE
2. LANDSCAPE AND NATURE CONSERVATION

AREAS OF SEARCH
BROAD IDENTIFICATION OF SITES WITH POTENTIAL FOR REPOSITORY LOCATION

Nearly 500 sites carried forward

FURTHER SCRUTINY
SITES WITHIN THE AREA OF SEARCH OR OUTSIDE IF APPEARING TO HAVE A PARTICULAR POTENTIAL FOR REPOSITORY LOCATION

About 200 sites carried forward

OWNERSHIP OF LAND
LAND UNDER THE CONTROL OF GOVERNMENT, NIREX OR OFFERED BY PRIVATE OWNERS

About 160 sites carried forward

SIZE/CHARACTERISTICS OF SITE
COASTAL: 200 HECTARES (500 ACRES)
INLAND: 400 HECTARES (1,000 ACRES), NOT LESS THAN 350 HECTARES

About 120 sites carried forward

FURTHER GEOLOGICAL APPRAISAL
AVAILABLE DATA FOR EACH SITE SCRUTINISED

About 40 sites carried forward

EXPERT REVIEW OF SITES
NIREX AND TEAM OF CONSULTANTS EXAMINED THE LIST TO IDENTIFY "FRONT RUNNERS"

17 land-based and 2 off-shore sites carried forward

DECISION ANALYSIS
ATTRIBUTES EVALUATED FOR 12 OPTIONS

DOUNREAY & SELLAFIELD
ACCORDED PRIORITY FOR DETAILED EVALUATION

Site Investigation

31. The decision to investigate these sites was necessarily formulated from information, some of which is of a theoretical or generic character, which needs to be verified by specific analysis and testing. The geological prognosis for the sites is that both are underlain by about 500m of sedimentary rocks, principally sandstone, over hard basement rocks. It is the latter which is of interest as a repository host rock. To confirm the current understanding of the geological characteristics the first step to sink two boreholes to basement rock at each location and to conduct a regional geophysical survey is under way. The borehole investigations consist of the following:

- boreholes which are fully cored, as far as is possible, to obtain high quality continuous core;
- hydrogeological testing in the boreholes;
- geochemical sampling of the groundwater from the borehole and pore fluids from the core material;
- geophysical logging of the boreholes;
- initial geotechnical assessment of the rock mass for construction purposes.

32. The core samples are providing material for laboratory testing and the boreholes will provide access to the underlying structures for hydrogeological and geological testing. The geophysical survey includes comprehensive surface and aerial surveys using modern techniques to determine inter alia the properties and the extent of the potential host formations.

33. In parallel with the geological investigations site-specific conceptual designs of a repository have been developed to meet the specific features of the candidate sites. This work enables operational safety cases to be formulated and provides updated cost information. The post-closure radiological safety studies will be focused on the two sites and will benefit from the acquisition of specific information on the characteristics of the geological environments underlying these sites.

The Waste Expected at the Repository

34. As mentioned in the introduction, United Kingdom Nirex Ltd is charged with developing a national repository for disposal deep underground of low and intermediate level radioactive waste. There are four waste categories that we have to consider:-

(a) Operational LLW
(b) Decommissioning LLW
(c) Operational ILW
(d) Decommissioning ILW

35. There are two distinct package types:-

(a) Packages arriving in transport containers and requiring temporary shielding. These include all operational ILW and decommissioning ILW 3 m³ boxes.
(b) Self-shielded packages.

36. The arrival rates for road and rail vehicles to a repository at Dounreay and at Sellafield are shown below.

TABLE 1

AVERAGE WEEKLY ARRIVAL RATES OF ROAD AND RAIL VEHICLES TO A REPOSITORY BASED AT DOUNREAY

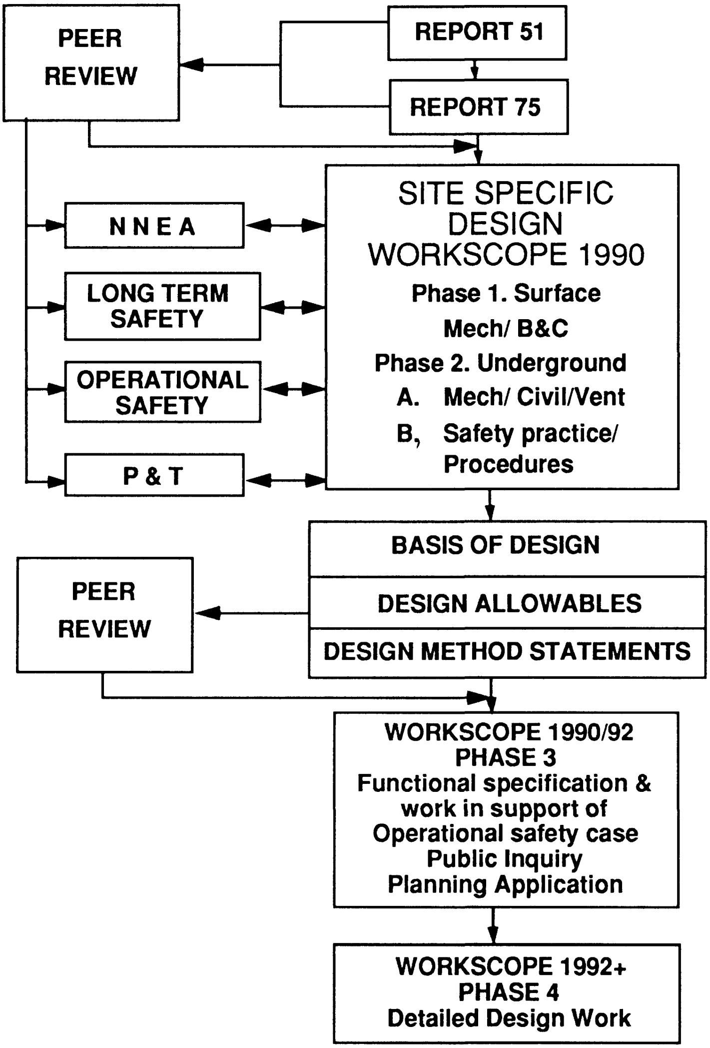
Data Sheet	Description	Average Weekly Rate (1)	Number of Transport Containers per Vehicle		VEHICLES		
					Rail Only	Road & Rail	
			Road	Rail	Rail Wagons (2)	Lorries (3)	Rail Wagons
6	$12m^3$ Self Shielded Box for ILW	6	-	1	6		6
8	$6m^3$ LLW Box	15	2	4	4	7.5	
9	$12m^3$ LLW Box	23	1	2	11.5	23	
11	R70 Transport Container	25	1	2	12.5	25	
12	R145 Transport Container	21	-	1	21		21
13	R210 Transport Container	17	-	1	17		17
14	R285 Transport Container	21	-	1	21		21
16	S150 Transport Container	6	-	1	6		6
19	Multi-Element Bottle (MEB))						
20	MAGNOX Fuel Transport Flask)						
21	Excellox Fuel Transport Flask)	2	-	1	2		2
22	AGR MK 1A Transport Flask)						
18	WAGR Box)						
10	Nirex ISO Freight Container(s)		1	2			
TOTALS		136			101	55.5	73
Block Trains (4)					8.4		6.1

TABLE 2

AVERAGE WEEKLY ARRIVAL RATES OF ROAD AND RAIL VEHICLES FROM OFFSITE TO A REPOSITORY BASED AT SELLAFIELD

Data Sheet	Description	Average Weekly Rate (1)	Number of Transport Containers per Vehicle		OFFSITE VEHICLES		
					Rail Only	Road & Rail	
			Road	Rail	Rail Wagons (2)	Lorries (3)	Rail Wagons
6	$12m^3$ Self Shielded Box for ILW	2	-	1	2		2
8	$6m^3$ LLW Box	9	2	4	2.5	5.0	
9	$12m^3$ LLW Box	14	1	2	7.0	14.0	
11	R70 Transport Container	5	1	2	2.5	5	
12	R145 Transport Container	7	-	1	8		8
13	R210 Transport Container	12	-	1	12		12
14	R285 Transport Container	6	-	1	6		6
10	Nirex ISO Freight Container (5)		1	2			
TOTALS		55			40	24	28
Block Trains (4)					3.3		2.3

37. Current Design Strategy



The Relationship Between Operational Safety and Design

38. The influence of a structured safety case in support of design, and a phased design, following sound construction methods, in support of operational safety are of paramount importance in the design strategy.

39. The main aims are:-

- to review conceptual site-specific-design, setting boundaries and targets for plant design.
- to demonstrate that the civil design provides adequate safety; placing no constraints on any further design and gains consent from the regulator to commence construction of the civil works.
- to demonstrate that the ME&I design provides adequate safety gaining consent from the regulator for the installation of this equipment.
- to present and justify proposals for inactive commissioning of the plant, resolving outstanding issues gaining consent from the regulator for inactive commissioning.
- to incorporate the results of inactive commissioning presenting and justifying proposals for active commissioning of the plant. Resolution of all outstanding issues prior to active commissioning and gaining consent from the regulator to commence active commissioning.
- to incorporate the results of active commissioning and by providing safety operating procedures demonstrating that the plant can operate safely.

40. Formalisation of these aims and the programmed design should achieve the requirements of a Planning Inquiry that could lead to a start on construction during 1995.

The Design Due to Conventional Loading and as a Result of Extreme Environmental Hazards/Abnormal Loading

41. Generally, the design of the repository will be subject to normal codes of practice. Considerations for conventional loading scenarios due to dead, live, wind, thermal etc will prevail in the first instance.

42. There will however be a requirement to consider abnormal loading due to scenarios such as fire and impact, and extreme environmental loadings due to seismic and extremes of wind, snow, rainfall and temperature. A combination of the various scenarios follows in tabular form. These are consistent with normal provisions for design accepted by regulatory authorities.

Table 3

Load Case	Load Condition	D	L	W	WE	N	ND	NE	SO	SE	O
1	Construction	1.2	1.2	1.2	-	-	-	-	-	-	1.2
2a	Normal	1.4	1.6	-	-	-	-	-	-	-	1.6
2b	Normal	1.4	1.6	-	-	1.6	-	-	-	-	-
	Climatic	1.4	1.6	-	-	-	1.05	-	-	-	-
		1.0	-	1.4	-	-	-	-	-	-	-
		1.2	1.2	1.2	-	1.2	-	-	-	-	-
		1.2	1.2	1.2	-	-	1.05	-	-	-	-
3	Seismic Operational	1.2	1.2	1.0	-	1.0		-	1.2	-	1.2
4	Extreme Environmental										
a)	Max Seismic	1.0	1.0	-	-	-		-	-	1.0	1.0
b)	Min Seismic	1.0	-	-	-	-		-	-	1.0	1.0
c)	Max Wind (Extreme)	1.0	1.0	-	1.0	-		-	-	-	1.0
d)	Min Wind (Extreme)	1.0	-	-	1.0	-		-	-	-	1.0
e)	Max Snow	1.0	-	-	-	-		1.0	-	-	1.0
f)	Min Snow	1.0	-	-	-	-		1.0	-	-	1.0

f	-	Partial safety factor for load as defined in BS 8110 and BS 5950
D	-	Dead load
L	-	Live load
W	-	Normal wind load
W_E	-	Wind load for 10^{-4} event
N	-	Normal snow load
ND	-	Drifted snow load
N_E	-	Snow load for 10^{-4} event
S_O	-	Load effects due to design operational safety shut-down seismic event
S_E	-	Load effects due to design safety shut-down seismic event
O	-	Other load not accounted for elsewhere

Quality Assurance

43. In applying quality assurance to an organisation it is essential that all concerned have a clear understanding of the requirements which are to be met. The origins of these requirements lie in a document 'Appendix B of the USA Federal Regulations 10CFR50'. This established quality assurance requirements for structures, systems and components for the design, construction and operation of nuclear power plant and fuel reprocessing plant. It identified what have become known as the 18 criteria quality assurance which are listed in Table 1 (Ref 5). These form the basis for most standards on quality assurance including the two British Standards

BS5882 'Specification for a total quality programme for nuclear installations' and BS5750 'Quality Systems'. In turn one or other of these often form the basis for regulatory or contractual requirements and can be regarded as the formal statement of quality assurance requirements. It is usual practice for an organisation to declare its quality assurance system to be compliant with one or both of these standards.

44. Quality Assurance programme, organisation and design control are the first three in the list of the '18 criteria'; these are of paramount importance in the management of a successful Project Design.

45. Whilst this Paper only covers the initial stages of design, sight of the end objective of the Company must not be lost. The total '18 criteria' will have its part to play.

46. Successful applications within the nuclear industry can be seen at major installations nationally, Heysham, Torness (Ref 6) and Sellafield (Refs 7,8) are examples.

47. Lovatt (Ref 5) concludes that 'Quality Assurance is a fact of life which in the form of nuclear regulatory requirements and more widely specified contractual requirements is here to stay. It needs to be applied with common sense and understanding so that it is an integral part of management control. What must be avoided is it becoming a bureaucratic overlay with little relevance to main stream activities which results in increased cost with no benefit'. The Authors of this Paper are in agreement.

Conclusion

48. The Institution of Civil Engineers charter describes the profession as:-

'A Society established for the general advancement of Mechanical Science and more particularly for promoting the acquisition of that species of knowledge which constitutes the profession of a civil engineer, being the art of directing the great sources of power in Nature for the use and convenience of man.'

49. More over, the Nevada Operations Office of the US Department of Energy, in their document 'Managing the Nations Nuclear Waste - the Yucca Mountain Story', issues the challenge:-

'Designing, constructing, and operating the nations first repository for high level nuclear waste is a challenge to today's technology. More importantly, it is a challenge to the ability of people to work together to resolve a complex issue. We have an unprecedented responsibility to safeguard the environment for our own and future generations and we all play a part in making these important decisions.'

50. The key words in the former are '... the art of directing the great sources of power in Nature for the use and convenience of man', whilst in the latter '... the ability of people to work together ... and we all play a part ...'. Both take cognizance of Nature and the environment, but just as significant, is the ability of people to direct, manage and work together to provide a sound economic engineering solution to a complex engineering problem.

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Discussion

R. DEXTER-SMITH, W.S. Atkins Northern, Whitehaven
Mr Costaz mentioned some quite startling settlements on one of his power stations. Would he care to give more information about why settlements occurred?

R. CROWDER, Taywood Engineering Ltd, Southall
I was interested by the concept of double containment as exemplified by the P4/N4 plants. However, the difficulty appears to be in continuing double containment beneath the containment base, the part of the structure perhaps most prone to flexural cracking. It may be postulated therefore that double containment is missing at the part of the structure where leakage is most likely. The question, therefore, is how this problem has been dealt with on French plants, and how the solution has been affected by the possible considerations for dealing with core melt and its influence on the containment base?

R. L. BRUCE, ABB Impell Ltd, Warrington
Could Mr Isgar give more details about the design of the repositories against extreme hazards, both internal and external, and in respect of both civil structures and mechanical handling plant?

A. CHALMERS, Taywood Engineering Ltd, Southall
I am sure that Mr Isgar is aware of the Swedish repository at Forsmark. My understanding is that it has been designed to take low-level and intermediate-level waste from the 12 Swedish nuclear power stations. It is at a depth of about 50 m. In the UK have we definitely abandoned the use of a similar facility, and are we irrevocably committed to a very deep repository, with the considerable engineering and technical problems and cost burden associated with

this? Mr Collier has mentioned the need to reduce the cost of the fuel cycle.

D. NAYLOR, BNFL Engineering, Warrington
What design life is required for the bearing? How do the designers demonstrate that the bearings are performing satisfactorily and maintain their original properties from the original design?

If a bearing needs to be replaced, what is the likelihood and how would this be achieved considering the massive vertical load applied to each bearing?

M. G. DOWN, NNC - Thyssen Corporation
Are there any major challenges to be overcome in the technology for construction of a deep repository? To what extent are conventional shaft-sinking and mining techniques appropriate?

S. N. FIELD, Taywood Engineering Ltd, Southall
In the light of the unexpectedly high rates of loss of prestress in certain stations, I would like to ask two questions. Firstly, how is this effect being observed? Is there some method of monitoring tendon stresses? If not, how does Mr Costaz compute concrete stresses from strains measured by vibrating wire gauges?

Secondly, would Mr Costaz care to give us some idea of the scope of any original experimental work that was used to provide concrete modulus and creep data before design?

D. SHILSTON, W.S. Atkins Consultants Ltd, Epsom
Quality assurance by peer review has been described as an important part of the Nirex design process. Would Mr Isgar comment on how peer review is procured and organized by Nirex for work involving specialist consultants. In particular, how is duplication of the consultants' work avoided in the peer review process?

R. CHAPLOW, Sir Alexander Gibb & Partners Ltd
I have been asked to give an update on the progress of the Dounreay and Sellafield geological investigations. Sir Alexander Gibb & Partners Ltd, in association with GeoScience Ltd and J. Arthur & Associates, are the consultants responsible for carrying out the geological investigations at both Dounreay and Sellafield.

At Dounreay the investigations comprise the following. Borehole 1 is schedule to be drilled to a depth of 1200-1300 m. It is currently at a depth of

1140 m and drilling is due for completion in mid-April 1991. The borehole has penetrated through the Devonian Caithness Flags and is now in the crystalline basement rocks. Borehole 2 is scheduled to be drilled to a depth of 800 m and to start in May 1991 on completion of borehole 1. There are also regional geophysical surveys which comprise onshore and offshore seismic reflection surveys, an airborne survey, and other regional surveys. The field work for these surveys is now complete and the results are being processed.

At Sellafield the status is as follows. Borehole 2 drilling was completed at the end of February 1991 at a depth of 1610 m. The final stage of testing is now in progress. Borehole 3 was begun in December 1990 and is scheduled to penetrate to a depth of 2000 m. It is currently at a depth of 1055 m and is on programme. Borehole 4 is due to commence drilling in April 1991 and to penetrate to a depth of 1250 m. The site preparation work for the borehole is in progress. Planning permission has also been obtained for boreholes 5 and 6, but no site work has yet started. Regional geophysical surveys, similar to those being undertaken at Dounreay, are in progress and will be completed shortly.

The investigations are generally proceeding according to programme, although some delays to the regional geophysical surveys have occurred due to bad weather. The borehole investigations generally comprise the following. There is rotary coring to produce an approximately 100 m dia. core. Extremely high-quality core recovery is being achieved using the triple tube, wire-line equipment. There are environmental pressure measurements to obtain values of the groundwater pressure and permeability throughout the holes. Down-hole geophysical logging, geochemical sampling, and vertical seismic profiling are also carried out. The consultants have also performed hydrogeological tests, comprising full sector tests to identify the presence of any flowing zones in the holes, and discrete sector tests to test individual sections of the borehole selected on the basis of the full sector texts.

J. -L. COSTAZ, Paper 1

In reply to Mr Dexter-Smith, the settlements of about 0.30 m occurred at GRAVELINES NPP where the ground was made of sand and clay. This value was predicted by the calculations, and the settlements are now stabilized.

To Mr Crowder, when doing air leak tests at the design pressure, the leaks across the base mat are the difference between the total leaks and the leaks across the cylinder and the dome which can be measured. It appears that the air leaks across the base mat are low and that they are close to zero when the mat is covered with water, which is the situation during an accident.

The core melt across the base mat has not been considered in the design, but the problem is the same with or without a liner.

In reply to Mr Naylor, the design life of a bearing is that of the plant: 40 years. Samples are kept in the same conditions as the actual bearings. Static and dynamic tests are made every five years If a bearing needs to be replaced, the column has to be destroyed and rebuilt. The stresses in the new bearings are given by flat jacks filled with cement grout or resins. The probability of such an event is very low.

To Mr Field, the loss of prestress is the consequence of both steel relaxation and concrete creep and shrinkage. The former is well known, the latter depends on the actual concrete. Our feed-back experience shows that the delayed strains are quite different from one site to another.

As we have standard designs which are made before knowing the actual concretes, the design relies only on the national regulations. More precisely, the BPEL (B}ton Pr}contraint aux Etats Limites) gives data that are generally convenient and conservative. There is one exception for which the creep rate is higher than predicted but we do not know, up to now, if the final value (40 years) will be different from the design value.

As far as the in-service surveillance is concerned, the actual values of the prestressing forces are plotted every three months by load-cells fixed on the ends of four straight vertical tendons (without friction losses). They can be also calculated with the concrete strain data of the vibrating wire gauges and the invar wires. If the original experimental work could not be done for standard reasons, a post-analysis of the results obtained on the various sites is very useful to confirm the design and to make improvements on future designs.

P. ISGAR, Paper 3

In reply to Mr Bruce, nuclear facilities are designed against the following (refer to p.24 of the Paper): conventional loading, abnormal loading, and extreme environmental hazards.

Conventional loading is self-explanatory and is confined to normal design methods using British Standards and Codes of Practice.

Abnormal loading (internally as mentioned in the question) is generally a result of normal operations being exceeded. Examples of this are increases in thermal loading as a result of non-standard operational procedures, or impact loading as a result of dropping items carried by cranes or collision of the item with the structure.

Extreme environmental hazards (externally as mentioned in the question) include seismic activity, extremes of wind, rainfall, snow and temperature.

May I refer Mr Chalmers to a statement made to the House of Commons by the Rt Hon Nicholas Ridley M.P., former Secretary of State for the Environment, on 1 May 1987 (Hansard 1/5/87/ Col. 504). Summarizing: Mr Ridley accepted the Nirex conclusion that although a safe near-surface disposal facility could certainly be developed at any of the four sites under investigation, the economic advantages of separate near-surface low-level waste disposal were not as great as Nirex first envisaged. He further accepted that it would be preferable to develop a multipurpose deep site for low-level and intermediate-level waste.

In reply to Mr Down, the technology for the construction of a deep repository exists in various technical areas allied to the Civil Engineering industry; mining is one such area. The UK experience is generally in soft sedimentary rock; however, hard rock shaft sinking and tunnelling experience exist worldwide. It is the task of the company to channel the right methods and techniques to support the specific requirements of the project. The company is exploring a drilling option as a possible alternative to conventional drill and blast, as used extensively in the UK.

In reply to Mr Shilston, peer reviews are used by the company as a way of carrying out technical appraisals of the work carried out against a specification. They identify the strengths and weaknesses of the work, and identify the shortfalls in the specification or work and recommend further studies. The need for peer review is documented in the recommendations of the Radioactive Waste Management Advisory Committee, 10th Annual Report, November 1989, and is consistent with the requirements of BS 5882 - Specification for a total quality programme for nuclear installation (see pp. 24 and 25 of the Paper).

The standard procurement technique is to use the actual reports/work to be reviewed as a 'scope of work', tenders being submitted together with a method statement and QA system. Duplication is avoided by clear instructions in the specification and correct listing of the deliverables required.

4. The HADES project—ten years of civil engineering practice in a plastic clay formation

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SYNOPSIS. Various civil engineering works and underground experiments have been performed during the last ten years in Belgium to assess the technical feasibility of building a repository for high level waste (HLW) disposal in a plastic clay formation; they lead to the conclusion that the construction of tunnels for this purpose may now be considered as technically and economically feasible.

INTRODUCTION

1. The industrial production of nuclear electricity implicates the management of the generated radioactive wastes. Geological containment can be considered as an ultimate defense against uncontrolled dispersion of radionuclides into the biosphere and therefore, Belgium develops at the Nuclear Energy Research Establishment (CEN/SCK) since 1974 a research programme concerning the final geological disposal of reprocessed HLW.

2. It appeared very rapidly that among the geological formations suitable for burial, some clay formations could present interesting characteristics. From previous drillings, it was known that an extended and homogeneous rupelian clay formation, called Boom clay, exists in the N-NE part of Belgium where the nuclear fuel cycle facilities and CEN/SCK are located. Fig. 1 presents a schematic map of the potential argillaceous formations in Belgium.

3. The geotechnical properties of this clay have been intensively studied in the past on behalf of construction works in the Antwerp region, at depths varying between 40 and 80 m; it was therefore evident that, although the same geological formation is concerned, the working conditions would be different at Mol, the depth considered for tunnels being 220 m.

THE USE OF GROUNDFREEZING

4. A geotechnical core drilling was performed at Mol in 1975 until the base of the Boom clay layer and a large amount of samples, considered as undisturbed, were taken. Fig. 2 presents the geological profile at the location of the borehole. From consolidated undrained triaxial tests, mechanical properties in effective stresses could be determined (ref. 1):

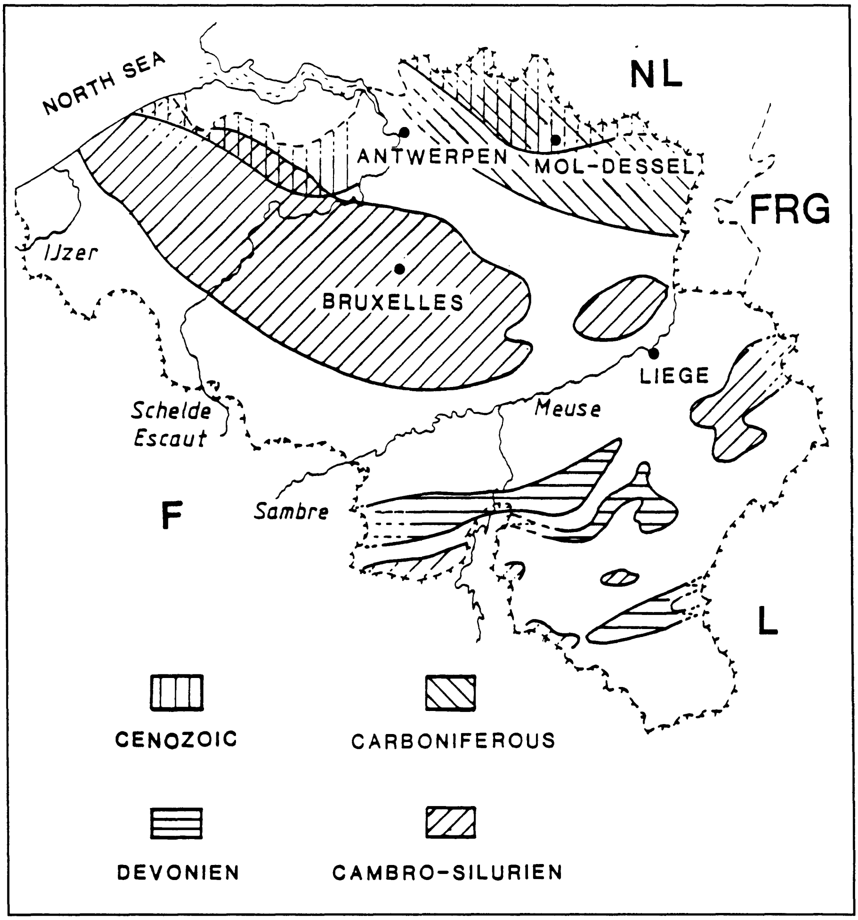

Figure 1: Potential formations for nuclear waste disposal

$$c' = 0.015 \text{ MPa} \quad \text{and} \quad \phi' = 22° .$$

They were in complete agreement with results obtained in the Antwerp region. For the undrained shear strength c_u, on the other hand, much higher values were predicted on basis of previous studies (about 0.85 MPa) than obtained with the samples (between 0.3 and 0.7 MPa). A conservative value of 0.6 MPa was finally considered in the design computations.

5. For the construction of galleries in such a clay, two geotechnical aspects have to be studied: the distance between galleries and the type of lining. For the distance, one of the criteria to be fulfilled might be that the damaged zones around the galleries, created by the excavation process and/or any other reason like temperature, should not cover each other.

6. Considering clay as an elasto-plastic medium without friction, a plastic zone with a radius R develops around an unlined excavation of radius r with the relationship:

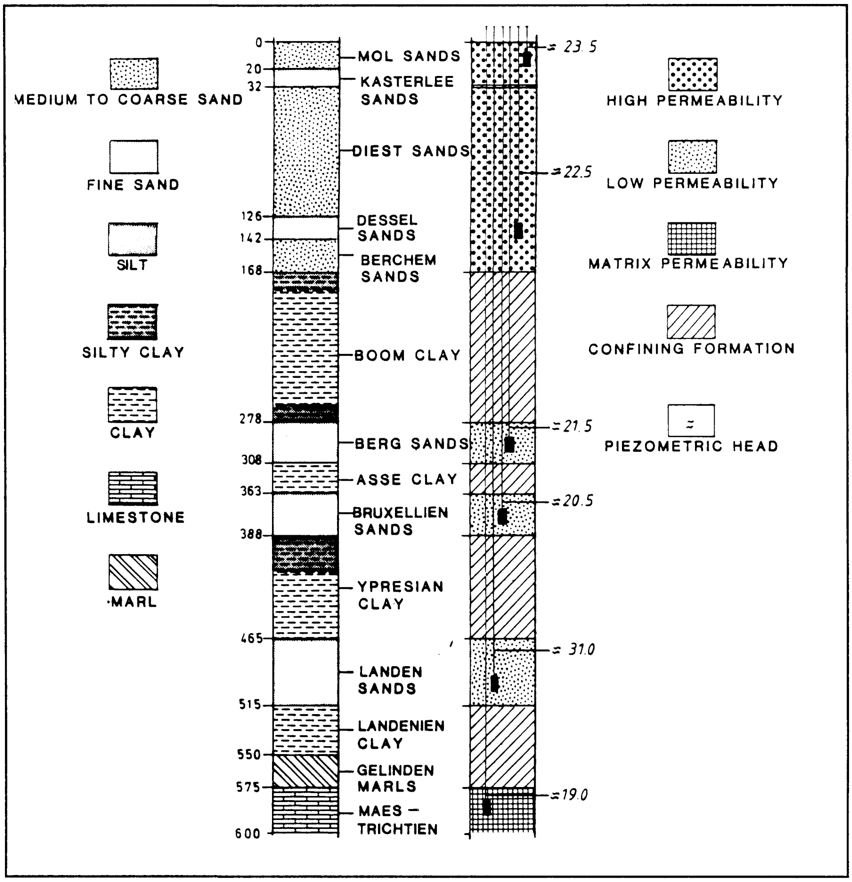

Figure 2: Geological profile of 1975 borehole

$$R/r = \exp(0.5\,(p/c_u - 1))$$

p being the mean normal stress. At a depth of 220 m, or p = 4.4 MPa, a ratio R/r of 23.8 is obtained, which means that for an excavated diameter of 4.0 m, a damaged zone of 47.5 m would be developed. It is to be enhanced however, that in such conditions, the whole thickness of the clay layer, around 100 m, would be concerned. This brought evidence that a ground conditioning technique was required. It was opted to use the ground freezing technique, as it was already needed for sinking the access shaft through the water bearing sands.

7. About the lining type and thickness, it was considered in the design calculations at that time (1978 - 1979), that the whole overburden pressure should be taken into account with severe restrictions on the K_0 value; using concrete liners with reasonable available strength and classical safety factors, the required thickness would then be important (more than 1 m for an internal diameter of 3.5 m), increasing the difficulties for

excavation.

THE SHAFT AND LABORATORY CONSTRUCTION

8. Both desk and laboratory studies pointed out the necessity of building an underground research laboratory in the clay. Besides the research opportunity that such a construction presents in geomechanical and mining fields, it constitutes an invaluable facility for the study of such phenomena as heat transfer, corrosion and ionic migration in a clay host, ...,

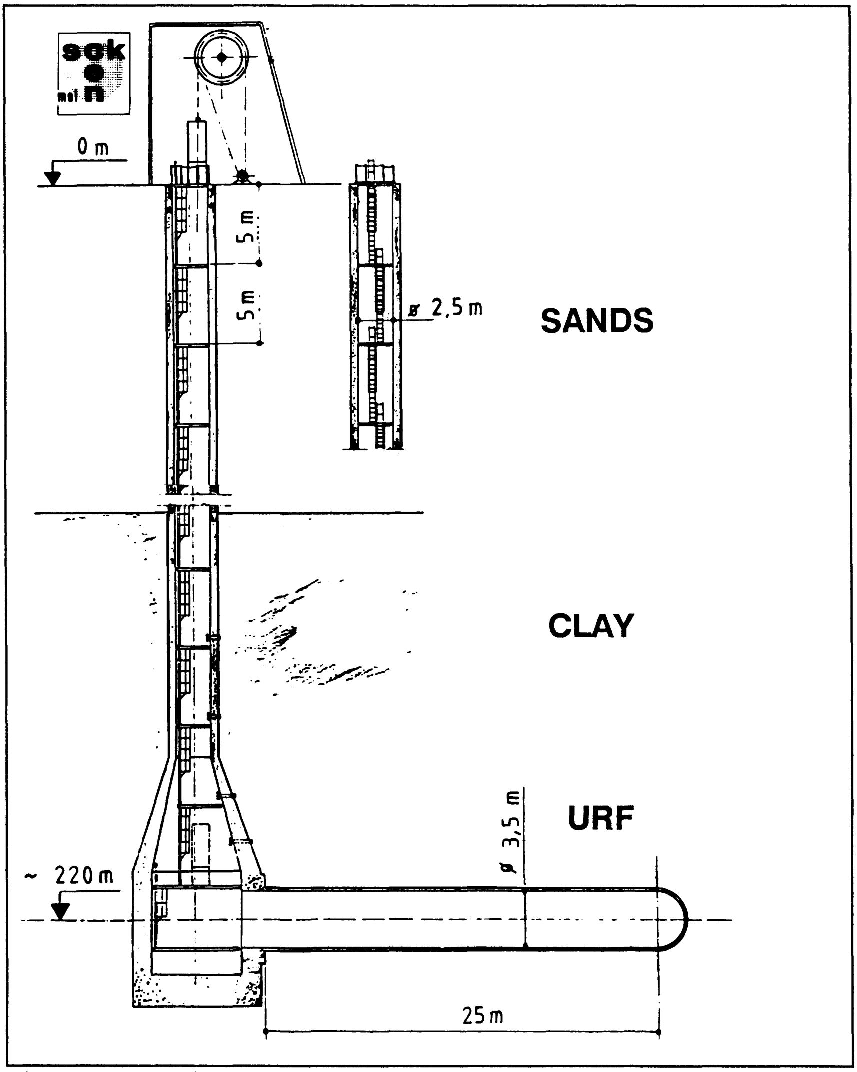

Figure 3: Original concept of the underground research laboratory (URF)

essential to demonstrate the long term safety of a HLW disposal.

9. A ground freezing technique was chosen for the digging works through the water bearing sand layers, a technique well known in Belgium as used for instance for every coal mine shaft, but also for the digging works in the clay, considering its poor mechanical resistance (see above) and the fact that a limited length of gallery, 35 m, should be built, excluding therefore the use of a TBM. Fig. 3 presents the original concept, comprising an access shaft, a crossing chamber and the experimental underground laboratory.

10. For the shaft sinking (ref. 2), soil was frozen by means of a calcium chloride brine, cooled down to about -25°C by ammonia frigorific groups. The underground facility being designed only for experimental purposes, the internal diameter of the shaft has been limited to 2.65 m. The sinking operation began in October 1980 and proceeded normally until the top of the Boom clay. Problems began to take place around a depth of 190 m in the frozen clay mass. The shaft lining was rapidly (2 to 3 weeks) under pressures significantly higher than the overburden. Displacements of the unlined clay mass at -12°C reached 1 cm per day. Modifications were therefore required in the design of as well the crossing chamber as the gallery. The shaft floor raft was poured in place on December 21st, 1981.

11. Fig. 4 presents the works as built: as the crossing chamber has been limited to 4 m internal diameter, the corresponding volume at the bottom of the shaft was not sufficient for the installation of freezing pipes required for the scheduled 4 m diameter gallery excavation; an access section, lined with steel profiles (2.10 m height on 1.60 m width), was first excavated in the already frozen area.

12. The rest of the gallery was planned in two phases, following the results of an intensive campaign on clay samples frozen at different temperatures (from -5°C to -30°C). The efficiency of the freezing was found to be dependent on several factors among them the temperature, which needs to be sufficiently low. The freezing period has to be reduced to the minimum to limit swelling pressures which can take place even after short periods of time.

13. Using freon freezing units, with a brine temperature around -32°C, together with an improved configuration of freezing pipes allowed excavation to start shortly after the latter were connected for a time period of 3 to 4 months. The measured displacements of the excavated front were reduced to 0.3 cm per day. The gallery, executed in 1982 - 1983, has a total length of about 35 m and is in its useful sections lined with cast iron segments of 20 cm thickness for an internal diameter of 3.50 m. A concrete plug, poured in place over a thickness of 2.5 m, finishes the gallery.

EXPERIMENTAL EXCAVATION WORKS (ref. 3)

14. The high cost, the deformations and other related problems caused by the freezing technique in clay led rapidly to the conclusion that, prior to any further design study, an in

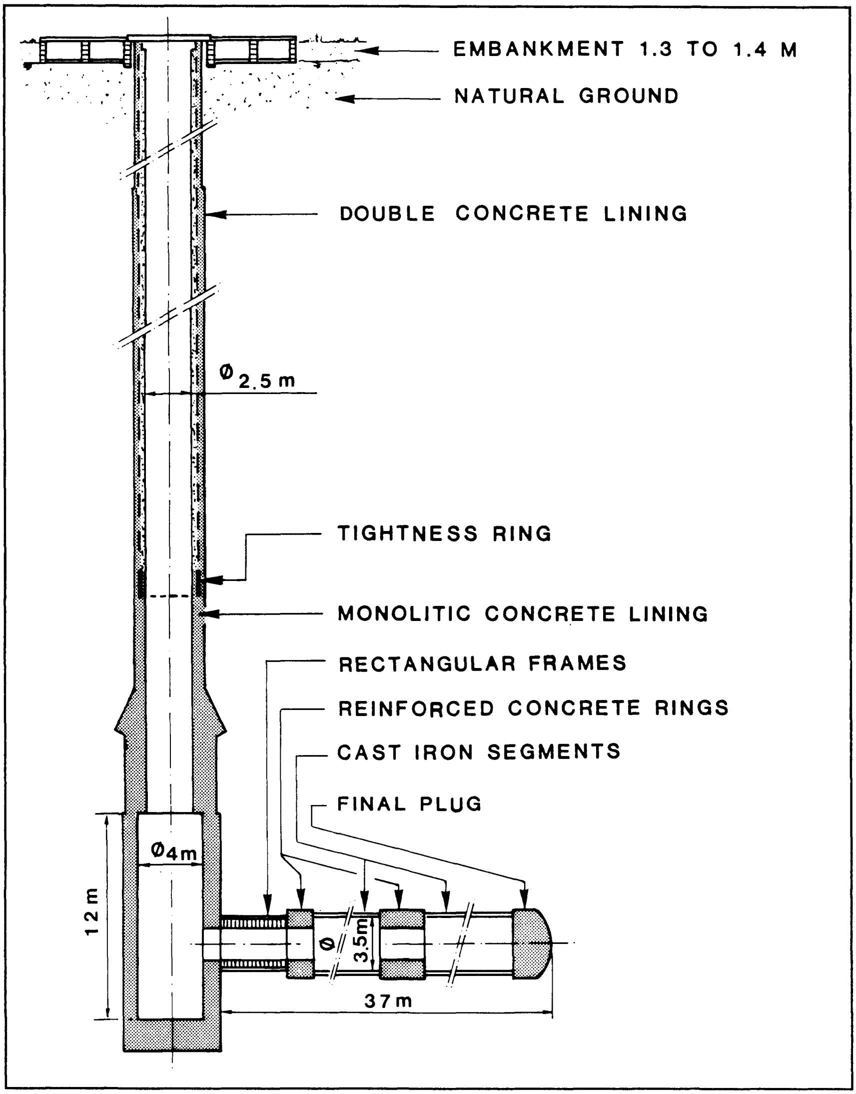

Figure 4: Underground research laboratory as built

situ study of the behaviour of this clay formation was a first prerequisite.

15. The construction of exploratory works, a small shaft from October to November 1983 and a small gallery from May to June 1984, both 2 m external diameter and lined with 0.30 m thick concrete blocks, was started directly after completion of the main laboratory, in parallel with the instrumentation of both clay body and liners. These works were the opportunity to initiate an bilateral cooperation between CEN/SCK and ANDRA, the French Agency for Radioactive Waste Management.

16. These works were performed without any major difficulty, creep phenomena in unfrozen clay being of lower magnitude than expected. Even several years after completion of the works, the lining behaviour is very fair, the pressure acting on the lining is much lower than the overburden, displacements in the clay mass are limited. The instrumentation allowed for the first time the determination of essential parameters such as the total convergence of the clay wall, 7%, and the thickness of the zone of influence of the excavation progress, 5 times the radius.

IN SITU AND LABORATORY TESTING (ref. 3)

17. Several in situ experiments were designed and performed together with ANDRA, LMS (Laboratoire de Mécanique des Solides of Ecole Polytechnique, Paris) and BRGM (Bureau de Recherches Géologiques et Minières, Orléans); efforts were concentrated on the determination of deformation moduli (pressuremeter tests), the application of the convergence confinement theory with retracting dilatometers, the free convergence of unlined boreholes and the simulation at small scale of various gallery linings by means of instrumented tubes in boreholes.

18. The digging works provided also for the first time the opportunity for sampling 'undisturbed' clay blocks much more representative than previous core samples taken from the surface for laboratory experiments; tests (triaxial compression, creep, ...) were performed in various institutions in Belgium and abroad, e.g. at LMS, at ISMES (Istituto Sperimentale Modelli e Strutture, Bergamo, Italy) and at BGS (Nottingham).

19. Mechanical parameters for the clay mass could be determined (laboratory campaign 1984 - 1986) on this 'undisturbed' clay, and compared to field measurements. Among others an elasticity modulus E of around 350 MPa and a c_u higher than 1.0 MPa, indicate a much higher mechanical strength, when compared with the values obtained ten years before on core samples, respectively 160 and 0.6 MPa; mathematical modelling, in total stresses approach and with Cam-Clay models, was intensively developed at CEN/SCK, LMS and ISMES and good agreement could be found between numerical simulations and in situ measurements.

DESIGN AND CONSTRUCTION OF THE TEST DRIFT (ref. 4)

20. Prior to the final design of a larger pilot facility, with a gallery length of at least 200 m, the construction of galleries without use of any ground conditioning technique needed to be demonstrated with industrial dimensions, e.g. with 5 m outer diameter.

21. The construction of an extension of the existing underground laboratory and accompanying geotechnical measurements was decided early 1986. High quality concrete segments were chosen for the lining (design ultimate strength: 55 MPa), safety factors and K_0 value were adapted in function of the experience gained during the previous excavation processes, but the whole overburden pressure was still taken into account in the calculations. The required thickness decreases then to 60

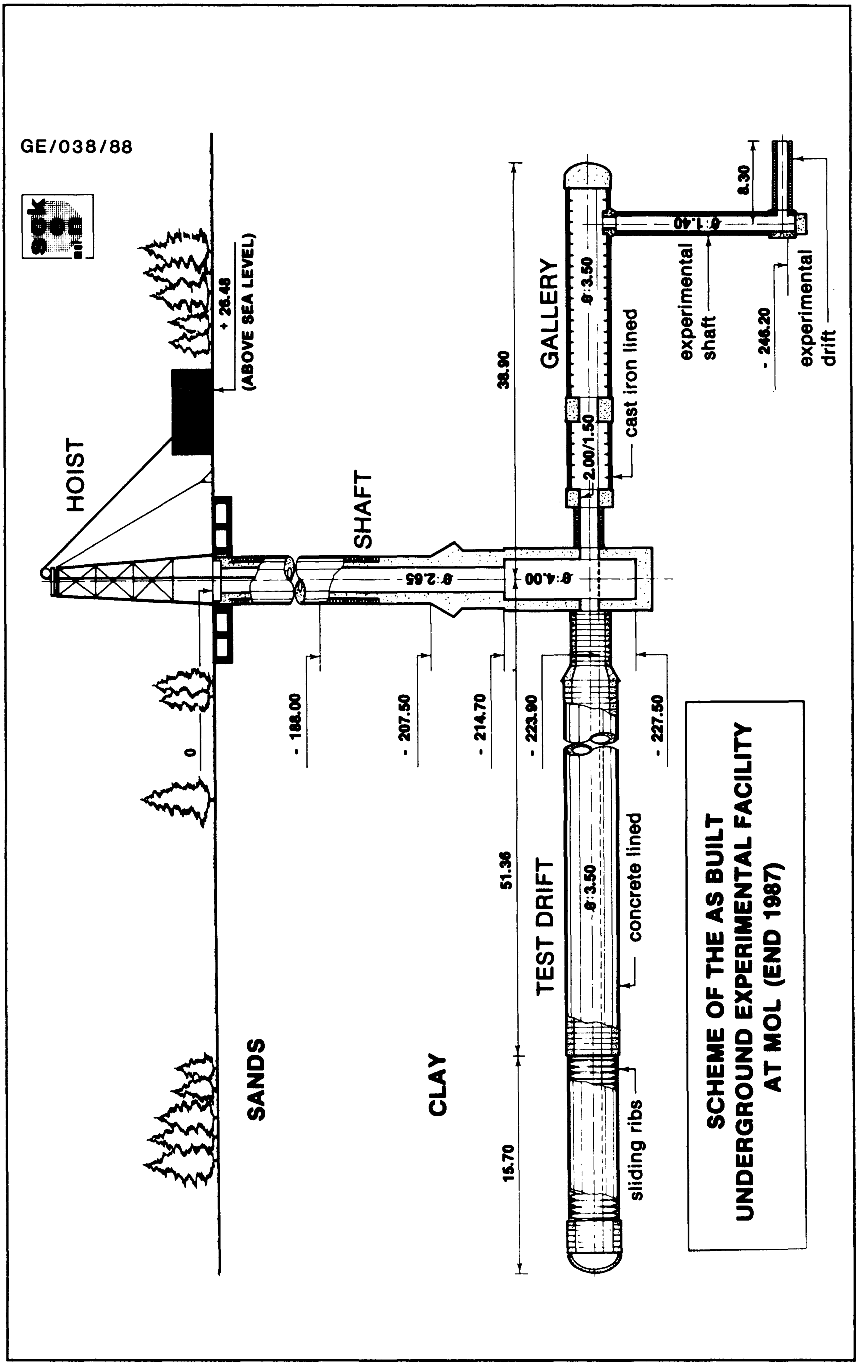

Figure 5: Layout of the underground facility at the end of 1987

cm, which is an acceptable value, for an internal diameter of 3.50 m.

22. During the entire construction, performed once more without mechanization from March to December 1987, no major difficulties were encountered. The executed works, enhanced on fig. 5, comprises, from the access shaft:

- a connection zone, 5.5 m long, of limited section in order to reach as soon as possible, a clay mass undisturbed by previous excavation works;
- the main part, 42 m long, lined with concrete segments (each ring, 0.33 m long, consists of 64 blocks of about 90 kg weight separated by wooden plates in order to increase its flexibility);
- a 12 m long part, experimentally lined with sliding steel ribs as already used in potash mines, built for ANDRA (ref. 5) and designed to sustain only part of the total overburden pressure;
- a 2 m long terminal buffer zone, required to sustain by friction the axial load coming from the terminal front;
- a 0.15 m thick, hemispherical shotcreted front cask.

23. On average, 5 working days with three 8-hours shifts were necessary to excavate and to line 2 m of gallery, both in the main section and in the experimental sliding ribs section. Reasons for this limited progress rate are to be found in the impossibility to mechanize the excavation, because of the limited length to be built, and the limited capacity of the access shaft, as well in volume as in weight.

24. An intensive observation and measuring programme was conducted, in order to verify the calculation methods and to determine the mechanical and hydraulic disturbances during the excavation progress, as well as the delayed effects; this programme is often called 'mine by test' by analogy with other field investigation of that type (underground facilities in Pinawa, Canada or WIPP in the USA). Results of the 'mine by test' are extensively described in (ref. 6). Fig. 6 illustrates the main devices, settlingmeters and pore water pressure cells.

25. Seven rings of the lining were instrumented with total pressure and load cells as well as convergence bolts. A maximal stress up to 2.2 MPa is measured, about half the existing pressure at rest at this depth. Delayed convergence after construction reaches nearly 1.5% but needed a long time after completion of the excavation works to reach a pseudo stability. Pore water pressure drops very rapidly during the excavation process, indicating a disturbed zone of 10 m radius or more around the drift.

26. Settlements in the clay mass were higher than expected, reaching the following figures:

distance to drift axis	8.1	6.9	4.9	(m)
short term settlement	40	53	107	(mm)
long term settlement	48	63	123	(mm)

explanation for the high displacements could be found in the

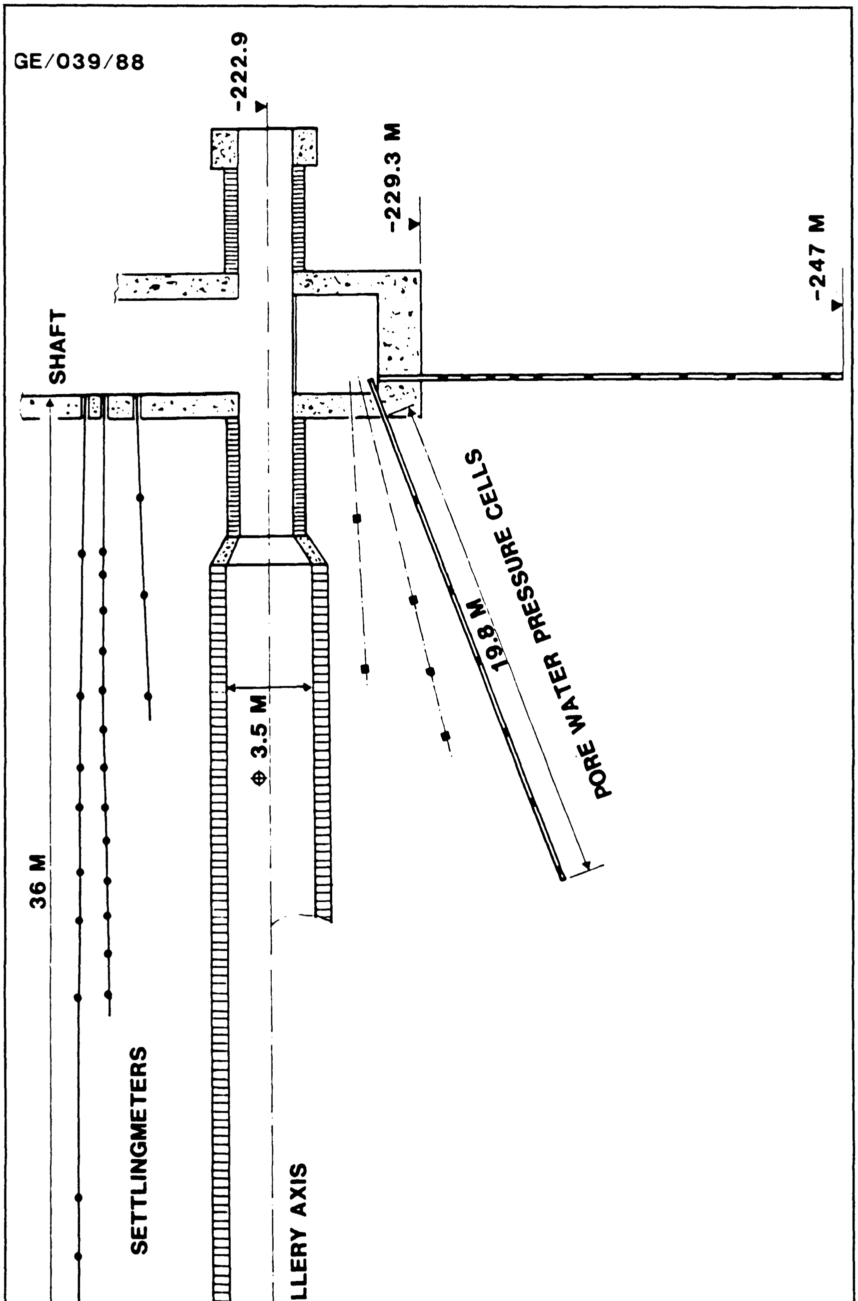

Figure 6: Emplacement of measuring devices around the Test Drift

slightly lower mechanical strength, c_u reaching only 0.8 MPa at this level (-223 m) than at the level of the exploratory works (1.0 MPa at -240 to -246m); water content is also 6 to 7 % higher at the level -223m.

THE FUTURE: THERMOMECHANICS

27. Many experimental works are running, e.g. the long term survey of the gallery linings and the shotcreted front, or scheduled for the next years which will see a new extension of the underground facilities with the realization of a crossing chamber at the end of the test drift, followed by the use of various mechanized tunnelling techniques for different configurations. This new available area will give the opportunity to assess essential aspects like heating of gallery lining, backfilling around heated sources and sealing of the repository, which performance has not yet been demonstrated.

CONCLUSION

28. The construction of tunnels for the final disposal of radioactive waste in Belgium has been demonstrated as technically and economically feasible. Further developments considering among others lining interaction, mechanization and thermal effects still need to be taken into account, together with many other experiments with nuclear finalities.

ACKNOWLEDGMENTS

29. The financing of this R&D project is obtained in the framework of several contracts between CEN/SCK and respectively the CEC (Commission of the European Communities, Brussels) and ONDRAF/NIRAS, the Belgian Waste Management Authority. Different collaboration agreements were established with ANDRA, PNC (Power Reactor and Nuclear Fuel Development Corporation, Tokyo) and CEA (Commissariat à l'Energie Atomique, Fontenay).

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5. Ground investigations for nuclear facilities and their impact on UK site investigation practice

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C. D. ELDRED, BSc, MIGeol, Sir Alexander Gibb & Partners Ltd

SYNOPSIS. The quality standards required for ground investigations for nuclear facilities, coupled with the availability of the resources necessary to meet these standards have provided the opportunity and the impetus for the development of new and enhanced ground investigation techniques. These techniques are now finding wide acceptance within the UK site investigation industry.

INTRODUCTION

1. During the period 1986 to 1991, the authors have been responsible for carrying out ground investigations and associated geotechnical studies for nuclear facilities at Fulbeck Airfield (for the possible shallow disposal of low level nuclear waste), Sizewell 'B' Power Station (monitoring of foundation dewatering), Heysham AGR Buffer Dry Store (geological investigations) and at Dounreay and Sellafield (possible deep disposal of nuclear waste).

2. As an integral part of these investigations, new developments have occurred in techniques related to coring and sampling methods, geophysical logging, permeability testing, instrumentation systems and quality assurance systems.

3. These developments are finding general acceptance within the UK ground investigation industry.

CORING AND SAMPLING METHODS

4. Rotary coring has for many years been a standard method for obtaining samples in rock. Within the UK, rotary coring has been used to a limited extent for sampling soils and weak rocks, although percussion techniques and open drive samplers have tended to be the dominant method employed for sampling such materials for economic reasons. These percussion methods have very limited value in glacial till deposits which contain cobbles and boulders.

5. During the mid nineteen eighties the use of rotary coring for sampling soils and weak rocks became much more of a standard with the additional costs associated with these techniques being regarded as justified in terms of the higher quality achieved. Conventional double tube barrels were used to obtain 100mm diameter cores in a wide range of soils, including glacial tills, using water and bentonite mud as the flush fluid. Results tended to be somewhat variable, but generally improved with the more widespread adoption of foam and polymer muds as the flush.

6. One of the major problems with achieving good core recovery in weak and friable materials arose as a result of the methods used for extracting the core from the barrel. The

results of careful coring were often completely negated by the careless removal of the core from the barrel resulting in excessive disturbance of the recovered core.

7. Split inner barrels, the use of Mylar sheeting and specialist triple tube barrels, especially the French Mazier barrel were used occasionally, but failed to achieve wide acceptance.

8. In 1986, when the specifications were being drafted for the investigations at Fulbeck Airfield, the requirement was established to achieve continuous core recovery in a sequence of clays and weak mudstones with thin bands of hard limestone. Full recovery was an essential requirement in order to demonstrate with confidence whether or not pathways for radionuclide migration existed.

9. None of the existing coring systems were considered to fully meet the stringent requirements of the project and hence the use of double tube core barrels which were specially modified to accept an inner rigid plastic tube was specified. Experimentation within some initial trial holes was undertaken to determine the optimum relationship between the inner diameter of the bits and the inner clearance within the plastic tube and site rules established related to the soil/rock types and whether the core was to be split for logging or retained as 'undisturbed samples' for geotechnical or radiological testing.

10. A variety of flush systems were also employed, comprising air, water, polymer mud, bentonite and foam. By optimising these variables it was found that a consistently high quality of core recovery was achieved. This "Coreline" system, manufactured by Core Drill (UK) Ltd, is becoming an industry standard where consistently high quality core recovery in potentially weak, friable or fractured rock is a prime requirement. The system has been used also with considerable success for sampling sands and other soils.

11. A controlled system was also developed whereby cores, within their rigid plastic liners, could be labelled at the drill site, transported to a covered store where the cores were split in a specially constructed jig. Half of the core was photographed and logged in detail by the geologists. The other half was sealed in heat-shrunk plastic sheeting and transported to an archive store. In this way the cores could be logged, sampled and tested and yet a complete record of the strata encountered was always available for reference in the archive.

12. The "Coreline" system is now being used for drilling the deep exploratory boreholes at Sellafield and Dounreay. Coring depths of up to two thousand metres are envisaged using a 159 mm diameter wire-line string fitted with "Coreline" tubing. Results to date are excellent with consistently high quality core being recovered in all the formations being encountered.

GEOPHYSICAL LOGGING

13. Even in situations where full core recovery is obtained, many geological units and formations are visually monotonous. In such strata, the absence of marker horizons can lead to difficulties in establishing the geological correlations necessary to define the extent of faulting or folding to the detail required for nuclear facilities. These problems become even more significant if core loss occurs within the boreholes.

14. Down-the-hole geophysical logging was used extensively during the investigations at Fulbeck Airfield and at Heysham. In both cases it was found to provide a cost-effective and efficient way of identifying markers in the geological succession with a precision and speed not possible by any other means.

15. The use of geophysical logging of boreholes is widely accepted as standard practice in the mineral exploration industry, but has been less widely used within the geotechnical industry. The use of slimline logging with a range of sondes proved to be a very valuable technique at Fulbeck within the Lias succession, and was subsequently used in the Carboniferous sequence at Heysham, with equal success.

16. The sondes used at Fulbeck were as follows:

Caliper
Multi-channel sonic log
Gamma-gamma log
Natural gamma
Neutron-neutron
Spontaneous potential
Single point resistivity

17. It has been found that many sedimentary sequences which are visually monotonous, and hence difficult to log using standard geological and geotechnical methods, tend to possess characteristic geophysical signatures. These signatures can be used to uniquely recognise marker horizons within a visually uniform sequence. The precision found at the Fulbeck site was remarkable. There was close correlation between the geophysical logs and the detailed sequence obtained from detailed geological and palaeontological studies.

18. Furthermore, it was found that individual markers could be identified from the geophysical logs, thus enabling the structure of the site to be defined, and small faults causing individual bands to be missing from boreholes, or repeated, to be clearly recognised. The clearest example of the value of the technique was the recognition of unique markers in two boreholes drilled 2.5 km apart (Fig. 1). At Fulbeck, the sonic log was found to provide the best means of recognising geological markers, whereas at Heysham it was the gamma and neutron logs which were most effective, hence justifying the use of a range of sondes.

19. Once the site stratigraphy has been established from boreholes which have been both cored and geophysically logged, considerable cost savings can arise from replacing cored boreholes with open holes plus geophysical logging. At the Fulbeck site the cost of an open hole plus geophysical logging was less than half the cost of a cored borehole. Rapid progress can also be achieved since open-hole drilling and geophysical logging is significantly faster than core drilling, particularly with deeper holes.

PERMEABILITY TESTING

20. One of the major objectives of the investigation of a potential site for nuclear waste disposal is to determine possible rates of migration of radionuclides in the groundwater. This requires an examination of the distribution of permeabilities at the site and of the hydraulic gradients likely to produce flow. At the Fulbeck site the main problem which was anticipated with measuring the permeability of the ground was that the anticipated permeability was several orders of magnitude below that normally measured in conventional geotechnical investigations. Conventional geotechnical testing methods involving the use of pumps and flowmeters were therefore rejected as unacceptable.

21. The objectives which were established for the testing programme at Fulbeck were as follows:

22. The techniques to be used must be fully validated on the basis of theoretical studies and appropriate mathematical models.

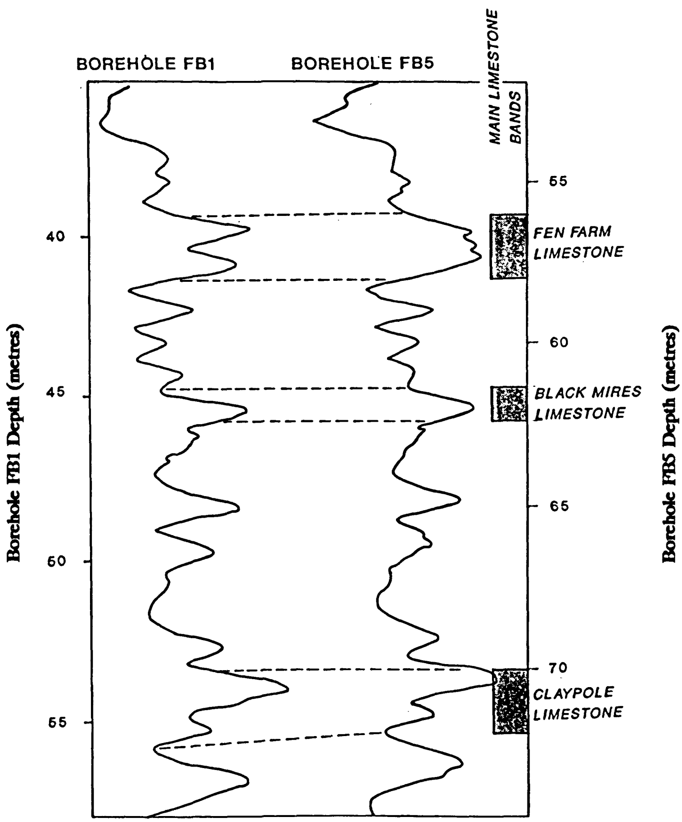


Fig. 1. Shear wave velocity geophysical logs from Borehole Nos. FB1 and FB5 at Fulbeck Site to illustrate correlation with stratigraphy.

23. The tests must yield reproducible results and must bear a close resemblance to actual conditions likely to exist around a repository. This necessitated the use of low head tests and the consequent requirement to measure very low flows.

24. The objectives were ultimately satisfied by the development of two main test procedures. The site was fully screened by continuous tests in overlapping test sections isolated using single packers as the boreholes were advanced. These tests incorporated environmental pressure measurements and low-head, constant-head permeability tests. Higher permeability zones were subsequently retested by isolating the section using double packers and carrying out constant discharge tests.

25. A total of 620 single packer and 80 double packer tests were completed using the purpose built test systems. These systems were housed in small caravans which were towed around the site as required. The small caravans, housing a wide range of sophisticated equipment and associated microcomputers for test control, data logging and computation, are becoming an increasingly common feature of sites where sophisticated investigations are in progress.

26. The test programme enabled a clear correlation to be established between the site stratigraphy and the permeability profile (Fig. 2). The results were reproducible within close limits, and good correlation was achieved between the two main test methods, thus providing added confidence that the conditions at the site had been reliably determined.

27. Permeability values as low as 1×10^{-10} m/s were routinely measured in tests lasting only one or two hours.

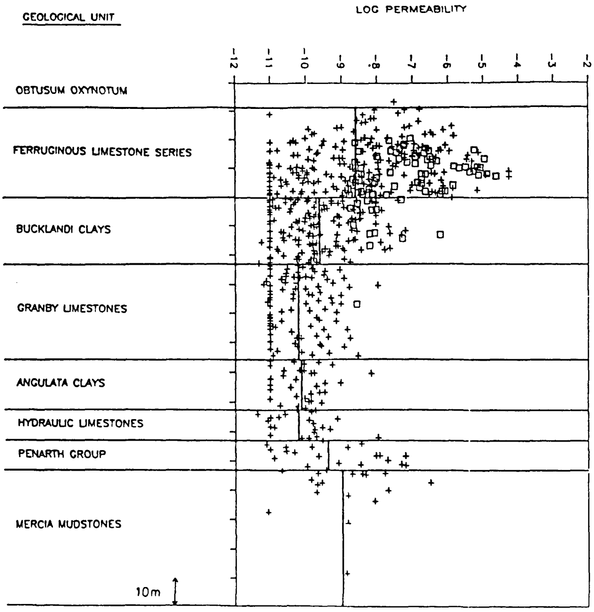
28. The test procedures adopted for the current investigations at Sellafield and Dounreay incorporate similar principles to those used at Fulbeck in that the full depth of the borehole is first screened with environmental pressure measurements (EPMs) from which both an environmental pressure and a permeability value are derived. Full sector tests are then carried out to detect any flowing zones. Subsequently, discrete sectors of the borehole are isolated using a double packer system and tested separately to define the characteristics of the individual zones, whether flowing or non-flowing.

INSTRUMENTATION SYSTEMS

29. The accurate measurement of groundwater pressures is a major part of many investigations at sites of nuclear facilities. At potential nuclear waste disposal sites, the determination of hydraulic gradients is a critical part of establishing the possible rate of radionuclide migration along groundwater pathways. At other sites the requirements are somewhat different. For example, at Sizewell, it was necessary to control the fluctuations in the groundwater pressures beneath the existing 'A' station during the dewatering of the site for construction of the 'B' station in order to avoid settlement of the existing foundations on sand due to an excessive lowering of the water table.

30. The measurement of water pressures using installed piezometers is a standard procedure during ground investigations. However, the need to measure extremely small variations in piezometric pressures at Fulbeck revealed that time dependent changes in pressure exceeded the variations in pressure across the site.

31. At Fulbeck a total of 48 piezometers were installed throughout the site to monitor groundwater pressures. A range of instruments were used including standpipes (plastic and stainless steel), hydraulic, pneumatic and electrical devices.



Double Packer Tests
Single Packer Tests
Vertical Line Indicates Average

Fig. 2. Fulbeck Site - Results of Permeability Tests and Correlation with Stratigraphy.

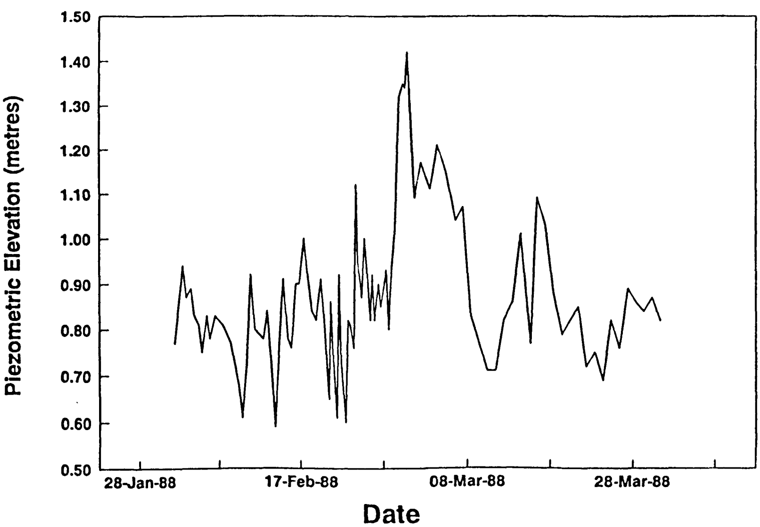


Fig. 3. Sizewell 'B' Dewatering - Readings from Piezometer No. 207B.

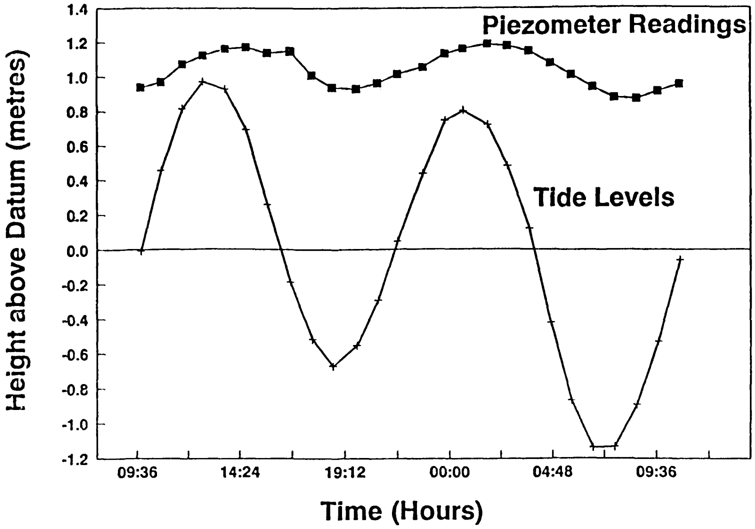


Fig. 4. Sizewell 'B' Dewatering - Correlation between Readings from Piezometer No. 207B and Tide Levels.

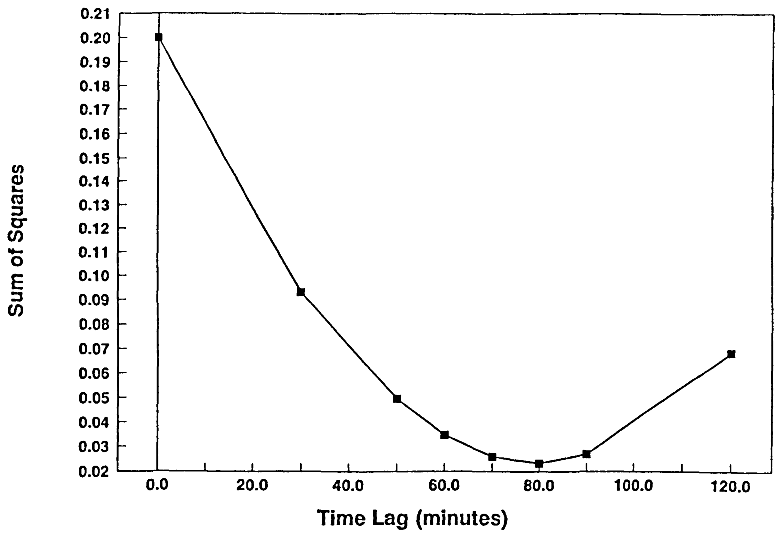


Fig. 5. Sizewell 'B' Dewatering - Computation of Time-Lag for Piezometer No. 207B.

An early finding was that those instruments which incorporated a closed, relatively stiff system, namely the electrical, pneumatic and hydraulic devices, displayed fluctuations in the pressure readings which were of a magnitude equal to or greater than the hydraulic head drop across the site.

32. These fluctuations were demonstrated to be caused by variations in barometric pressure and hence it was necessary for all piezometer readings to be routinely corrected to a standard atmospheric pressure before interpretations could be made.

33. Processing of instrumentation readings to remove the effects of external factors in order to reveal underlying trends in the data is becoming a routine procedure, which is greatly facilitated by the application of various filtering techniques applied to computer stored data sets.

34. A recent example to illustrate this technique is taken from the dewatering of the Sizewell 'B' nuclear power station. The site is located close to the sea adjacent to the existing 'A' station. Dewatering of the 'B' site has taken place within a deep diaphragm wall, but nevertheless, careful monitoring of the 'A' site was necessary to ensure that drawdown did not occur beneath the operating station and lead to settlements of the foundations. As the site was close to the sea the piezometer readings were affected by the tides (Fig. 3). It was therefore necessary to filter out these tidal effects in order to obtain early warning of any general drawdown effects.

35. The installed piezometers were subjected to hourly readings over a 24 hour period (Fig. 4) and the readings compared with published tide tables. It would have been more accurate to measure actual tide levels, but this proved to be impractical in the time available. The amplitude of the tidal effects recorded in the piezometers were less than the tidal variations and the effects showed a time lag behind the actual tide levels.

36. Having computed the time lag for each piezometer (Fig. 5), the amplitude factor could also be calculated and corrections applied routinely to the measured readings (Fig. 6). The time-lag and amplitude factors varied with the distance of the piezometer from the sea.

37. The effect of this simple filtering is clearly demonstrated in Fig. 7. The results would have been even more convincing if actual tide data had been available since significant fluctuations in tide were known to be caused by onshore winds. Nevertheless, even using the published tide tables, the amplitude of the fluctuations in readings were reduced such that underlying trends could more easily be recognised.

QUALITY ASSURANCE SYSTEMS

38. There is an absolute requirement that work undertaken within the nuclear industry is carried out in accordance with a Quality Assurance System incorporating written procedures and appropriate documentation to define how all activities were undertaken. Such systems have been applied to all the ground investigations carried out at the potential nuclear waste sites, and although such systems do not have general application to all ground investigations, there are certain aspects which are, with benefit, being applied to general investigation practice.

39. The scale of the investigations undertaken at the nuclear waste sites and the absolute priority given to quality have led to the development of a number of positive benefits to the ground investigation industry in general. The production of written procedures for a wide range of standard operations have helped to better define requirements and have acted as check-

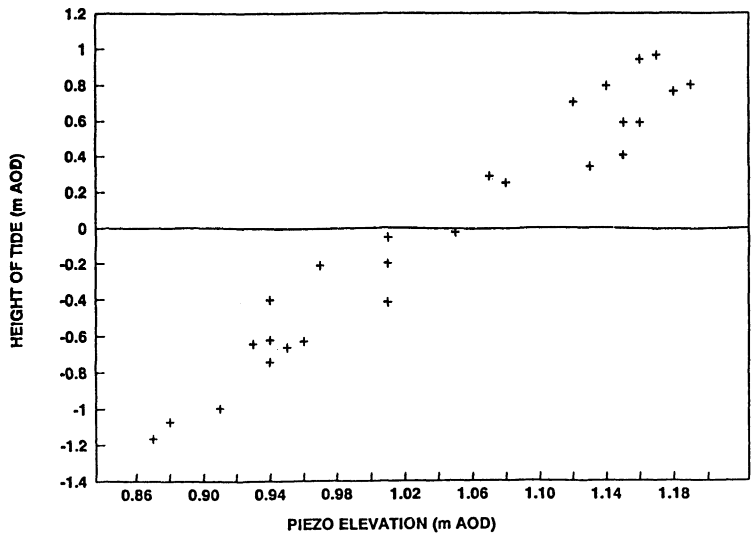


Fig. 6. Sizewell 'B' Dewatering - Computation of Amplitude Factor for Piezometer No. 207B.

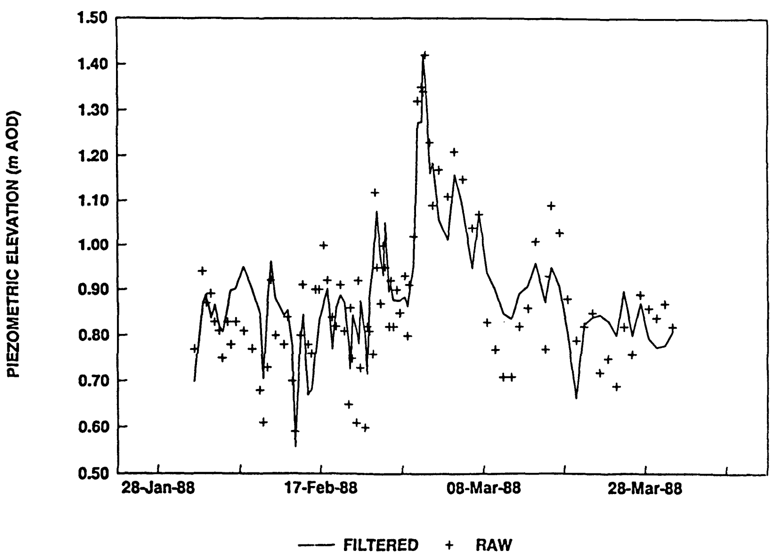


Fig. 7. Sizewell 'B' Dewatering - Comparison between Filtered and Raw Data from Piezometer No. 207B.

lists. The adoption of new techniques, with the associated validation work done as part of the development process, have served to improve the general level of reliability of field operations.

40. During the field and laboratory activities it has been a requirement to determine critical parameters using a variety of independent techniques. Examples have been given above of permeability measurements undertaken using two separate test procedures, and the use of a variety of piezometer types for monitoring groundwater conditions. The use of these independent methods has helped to provide added confidence in the validity of the site parameters and has also indicated some of the limitations and advantages of individual techniques.

41. Finally, the need to ensure that the measurements provided a reliable determination of site conditions taking due account of possible equipment malfunctions has provided much valuable data of the reliability of tests and equipment. Such data is of value in permitting the appropriate level of redundancy to be incorporated into the design of future investigations.

CONCLUSIONS

42. Some of the techniques applied to the investigations at the nuclear facilities as described in this paper are finding a more general application in the ground investigation industry. In particular, the nuclear waste programme is having a significant impact on general investigation practice.

43. This impact arises from the political and environmental sensitivity of the nuclear waste programme and the requirement for the highest quality of investigation practice. In these circumstances, the resources are available to permit quality to be achieved and for new and improved techniques to be developed. The value of such major, sensitive projects in advancing the art and science of ground investigations cannot be overemphasised.

ACKNOWLEDGEMENTS

44. The permission to publish this paper granted by Sir Alexander Gibb & Partners Ltd is gratefully acknowledged. The Heysham investigation was carried out by Mott Halcrow Gibb. The Sellafield and Dounreay investigations are being carried out by Gibb in association with GeoScience Ltd and J. Arthur & Associates.

6. Druridge Power Station—site studies

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SYNOPSIS. Site studies were undertaken between 1979 and 1988 in connection with a proposal to construct a nuclear power station on the coast in south east Northumberland. The studies described in the paper were part of a wide range of investigations undertaken by the Central Electricity Generating Board. Two potential sites were identified and it was established that the selected site could accommodate two PWR stations similar in size to Sizewell 'B', although no firm proposals for the development were made at the time of the studies, and none exist at present.

SITE SELECTION.

1. In 1979 there were only a limited number of known sites in England which satisfied the criteria set by the Central Electricity Generating Board (CEGB) for the construction of a nuclear power station. The CEGB therefore undertook a number of preliminary site selection studies in order to increase its portfolio of potential sites which could be considered when the demand for increased generating capacity had been established. (Nuclear Electric plc have now taken over the CEGB's responsibilities for generation by nuclear power).

2. The System Planning Department of the CEGB commissioned Merz and McLellan and William Armstrong and Sons, (now practising as Wardell Armstrong), to carry out initial studies to identify any potential sites between Cresswell village and Hauxley Head, (Fig.1).

3. William Armstrong's brief was to report on the location of coal mines as the area contains many underground mines and opencast pits, some still in operation today. With this information the two consultants identified two sites which were neither sited over abandoned underground mine workings nor had been worked by opencast mining. During this desk study phase of the initial investigations a paper by Anson and Sharp (ref.1) gave a valuable insight into the rockhead levels, which have been affected by glaciation to form two 'buried valleys', one under each of the identified sites. The absence of opencast mining in these areas was not accidental as the deeper thickness of glacial till overlying the coal measures made it less attractive for opencast working, particularly in the 1950-60 period when opencast pits were typically much shallower than now.

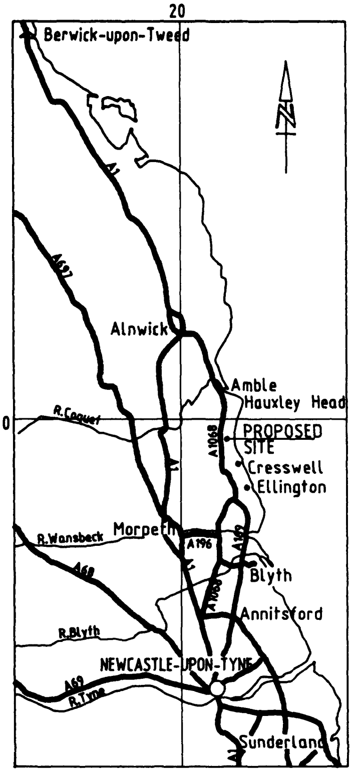

Fig. 1. Location Plan

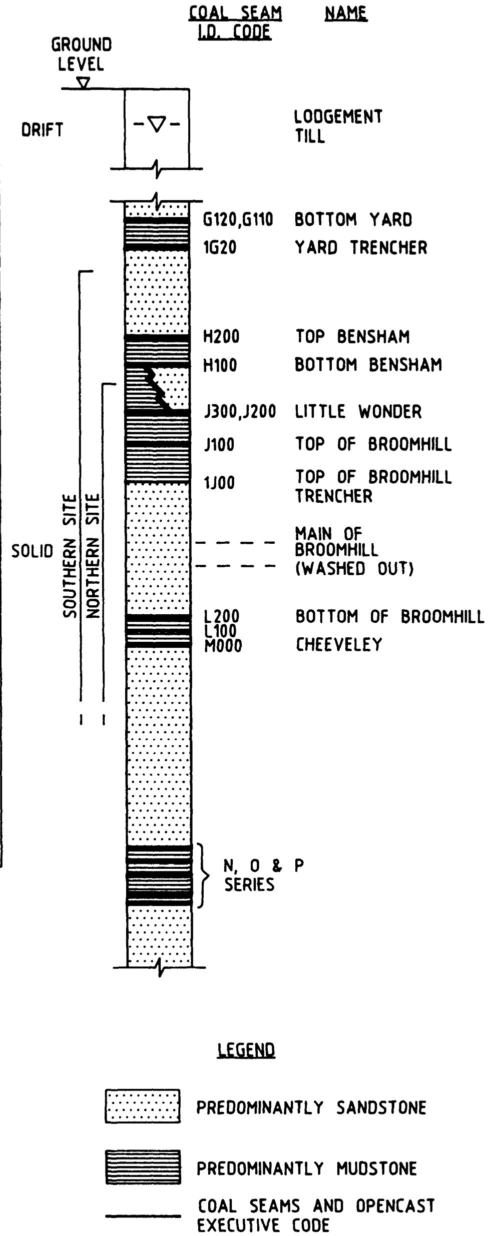

Fig.2. Simplified Stratigraphic Column

4. Anson and Sharp had obtained their data from a study of the results of exploratory boreholes sunk to assess the potential for opencast coal mining. It is unusual to have such data available when studying a 'green field' site. Moreover large opencast mines in operation nearby also afforded a unique opportunity to observe the stratigraphy of the upper 150 m of the drift and solid geology.

5. Merz and McLellan was then authorised to arrange a preliminary ground investigation in the autumn of 1979 on each of the two sites in order to assist in selecting the preferred site. Seven boreholes were sunk in each site to form a traverse across the suspected 'buried valleys'. A hydrographic survey was also undertaken to assist in cooling water studies. The programme for the ground investigation was carefully arranged to minimise interference with harvesting and sowing. The programme for the off-shore survey was determined by the salmon sea-netting season, which is unique to the north east coast of England.

6. In parallel with the field work further desk studies were undertaken to establish the capacity of local sources of construction materials and to investigate road, rail and sea access for normal and abnormal construction traffic. A preliminary study of the seimicity of the site was carried out with the assistance of Dr RE Long of Durham University. Meteorological data was collected and consideration was given to risk of flooding on the site. As the result of these initial studies one of the sites, now referred to as Druridge Power Station, was selected and the stage was set for a series of much more comprehensive investigations.

PRE-APPLICATION STUDIES

7. At the end of 1982 the CEGB decided to proceed with the design of the Druridge Power Station in sufficient detail to enable the statutory application to be made for consent to build the station. Many organisations, both internal and external to the CEGB, were involved in these design studies, all under the coordination of the Corporate Strategy Department of the CEGB. This paper only describes some of the activities of the firms represented by the authors. Merz and McLellan acting as lead consultants, together with Wardell Armstrong and James Williamson and Partners, (referred to hereinafter as 'the consultants'), were asked to collaborate to extend the work undertaken during the initial studies. The consultants worked closely with and under the direction of the CEGB's Generation and Construction Division who provided much of the basic data necessary for the conceptual design of civil engineering aspects of the station. Seismic hazard studies were not included in the consultants' brief as the subject was then being studied on a nation-wide basis by the Seismic Hazard Working Party set up by the CEGB.

DESK STUDIES

8. The consultants' first task was to study all the data which had been collected in 1979/80 and to supplement it with further desk studies, culminating in a report which outlined the recommended more detailed site investigations necessary to fulfil the brief. Whilst the desk study report was presented in May 1983, desk studies continued for another twelve to eighteen months in a number of subjects, some of which are described in the following paragraphs.

Geography

9. The site is located in a relatively featureless coastal plain which is drained by a number of rivers flowing from west to east through steep sided valleys. The district surrounding the site is sparsely populated compared with the Tyne and Wear conurbation to the south.

10. British Rail's east coast main line runs close to the site and a number of sidings off the main line serve the opencast and deep mines. The main road routes radiate from Newcastle, including the A1(T), which passes the site, about 8 km to the west. Blyth Harbour and the Port of Tyne are the main local ports, with Warkworth harbour to the north of the site near Amble, being one of several minor ports serving the fishing industry.

11. There are a number of local sites of historical interest, although none on the proposed power station site. The nearest Site of Scientific Interest is located about 1 km to the south. The coastline attracts numerous birds, particularly during the autumn and spring. Druridge Bay is a well used and relatively un-spoilt recreational beach of considerable importance to the locality.

Mining

12. The site selection studies had indicated that there was no recorded mining on the majority of the Druridge Power Station site. However, extensive opencast and underground mining had taken place in the surrounding area which extended into the boundaries of the site. Detailed records of previous mining activities where examined at the Mining Records Office in order to gain the maximum amount of information on the geological structure and stratigraphy of the site and surrounding area. These were supplemented by archive material held by the Northumberland Records Office on the historical development of this part of the Northumberland Coalfield.

13. The site is almost surrounded by previous opencast mining sites, now backfilled and restored to agriculture. The oldest is the Druridge opencast site to the east which extracted the Bottom Yard seam at shallow depth, (Fig 2 and 3). The site was restored in 1954. The Radar South site was located to the west and mined five seams from the Top Bensham down to the Main of Broomhill. This sequence of strata was anticipated to underlie the Druridge Power Station site. The opencast mine was fully restored in 1963.

14. Radar North was located to the north of the minor road which forms part of the northern boundary to Druridge site. The opencast mine extracted the same seams as Radar South and was restored by 1974.

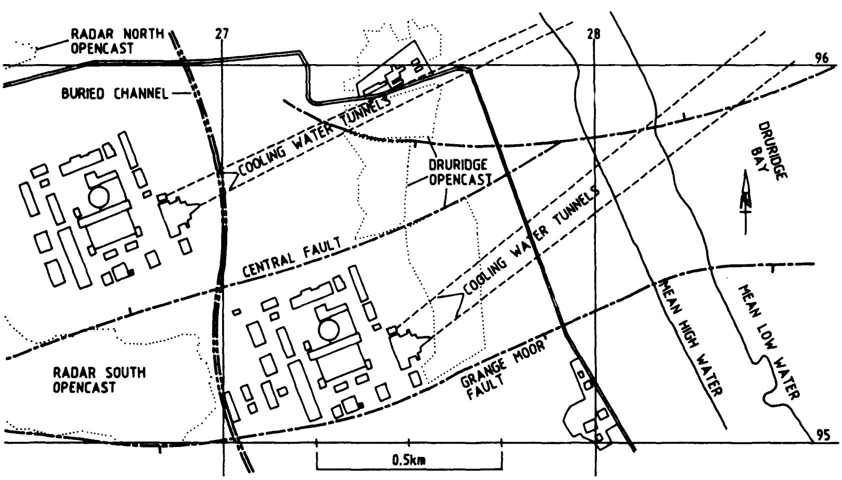

Fig.3 Site Plan

15. The opencast mining records indicated the distribution and pattern of minor faulting in the vicinity of the site and the general location of two significant faults which traversed the site. The southerly downthrowing Grange Moor fault formed the southern boundary of both the Druridge and Radar South sites and could be interpolated as lying close to the southern boundary of the power station site. A northerly downthrowing fault intersected both opencast mines and was expected to cross the central part of the power station site.

16. Previous underground mining took place to the south of the Grange Moor Fault. Ferneybeds Colliery worked the Bottom Yard seam in the early 1900's to the south west of the site; the mine was abandoned in 1924. Mining took place by room and pillar methods with subsequent extraction of some pillars prior to abandonment.

17. Underground mining also took place between 1944 and 1968 from Ellington Colliery located to the south of the site. Four seams were extracted, from the Ashington down to the Bottom Yard, using longwall methods which involve almost total extraction of the coal.

18. Workings from Ellington Colliery also extend off-shore beneath Druridge Bay and extraction is progressing northwards through the area east of the power station site. The upper two seams, the Main and Bottom Yard, are worked by room and pillar methods because of the shallow cover to the sea bed. The deepest seam, known as the Brass Thill, but also called Main of Broomhill (Fig.2) is mined by longwall methods at some distance off-shore. Mining in the Bottom Yard seam was progressing shorewards adjacent to the site during the course of the studies.

19. The detailed information provided by this data enabled specific features to be addressed in the site investigation and facilitated planning the overall investigation strategy.

Geology and Geotechnical Properties

20. Existing information indicated the near-surface solid geology to be wholly Carboniferous, the Druridge area lying within the broad subcrop of the Middle Coal Measures of the Upper Carboniferous Series comprising cyclothems (repeated sequences), of mudstones, sandstone, siltstones, seatearths and coal seams, as shown on Fig. 2.

21. Records from the opencast sites gave information about the position and throw of faults in the area of the workings and enabled the consultants to plan three possible power station locations within the site for detailed investigation. The most significant general conclusion which was drawn at this stage was that part of the site, (for the proposed southern power station unit) was immediately underlain by a 15 m thick sandstone unit with potential as a favourable founding stratum. The rest of the site was underlain by mudrock, coals and thinner sandstone units.

22. Rockhead was seen to be overlain by lodgement tills which are generally from 5 to 15 metres thick but which thicken considerably along the rather sinuous line of the buried channel which travels across the site from north to south. Sands and gravels had been found between boulder clay horizons in some boreholes but such sands and gravels were not expected to be laterally continuous. The sand dunes and beach deposits along the coast were of geologically very recent origin.

SITE INVESTIGATION

23. An extensive literature search was made to compare and contrast the engineering properties of the Glacial Till and Coal Measures obtained from the previous investigation with that recorded elsewhere. Published case histories of the performance of foundations engineered upon Coal Measure rocks were also scrutinised.

24. It was concluded that a rigorous programme of drilling and testing would be needed in the tills, sandstones, siltstone, mudstone, coal and seatearth of varying thickness and engineering behaviour in order to provide the confidence required in predicting foundation behaviour for a nuclear power station.

SITE INVESTIGATIONS

General

25. The result of the 1979/80 investigations suggested that the Druridge site might provide areas for three power station sites The 1986 investigation concentrated on the southern power station site, which was potentially the best, with the other two possible sites being examined in lesser detail, but with sufficient information being obtained to establish the major features of the Druridge site and to allow the variation of properties between the three power station sites to be correlated and assessed.

26. The investigation was designed to:

(a) Confirm and determine the stratigraphy and structure of the soil and rock.

(b) Determine the hydrogeology of the site and the immediate surrounding area.

(c) Confirm and determine the parameters of the soil and rock as required for design purposes i.e. their strength for bearing capacity; their deformation characteristics for settlement calculations; and their permeability characteristics.

27. The density and location of the boreholes at the southern power station site were guided by the recommendations of the US Nuclear Regulatory Commission for 'Site investigation for foundations of nuclear power plants' (ref. 2). However, the pattern of boreholes drilled was less dense than that suggested by these recommendations.

Onshore

28. The onshore investigation was conducted under the consultants' direction in 1984 by Soil Mechanics Ltd (SML) and their specialist subcontractors. This investigation included: 11 boreholes in the drift; 63 boreholes into rock using rotary core drilling methods ranging in depth up to a maximum of 150 m below ground level; 21 open rotary holes drilled through the overburden into rock with no core recovery, used for in situ testing; a single large diameter bore hole augered through the drift to rockhead for plate load tests and a well sunk to the base of the sandstone immediately underlying the southern power station site. Some of the rotary cored holes were sunk at inclinations of 45°or 60° to the horizontal in order to locate faults. Extensive in situ testing was carried out in the boreholes to determine the mechanical properties of the drift using SPT, Menard and self boring pressuremeters; and in the rock using high pressure dilatometers (ref.3). Cross, up and down hole geophysics and borehole geologging was carried out by Auger Geophysical Services and BPB Instruments Ltd respectively. A programme of groundwater well pumping tests, tracer tests, water level observations, falling head permeability and packer water pressure tests were carried

out to assist in the interpretation of the hydrogeology of the site. A separate contract for a Mini Sosie Survey was also let to Horizon Geophysics Ltd.

29. The investigation confirmed that the structural geology of the site is dominated by two east-west trending faults: the Grange Moor fault lies on the south boundary of the site whilst the second fault, named the Central fault for the purpose of the investigations, bisects the site. Other minor faults were detected or are suspected to exist. The presence of a buried channel which runs in a north-south direction was confirmed. No tectonic folding was detected.

30. The hydrogeological investigations confirmed that the rock is overlain by a blanket of practically impermeable glacial till. Ground water levels in the rock are artificially depressed by pumping in the adjacent mine workings with a 20 m drop in ground water level from the north to the south of the Grange Moor fault. Ground water tracer studies confirmed that ground water flow is generally north to south following the path of the buried valley to its intersection with the Grange Moor fault.

31. Field and laboratory strengths have been determined for all the main rock units and soil layers. Examination of the results indicates that the rock strata can be ranked in order of decreasing strength and stiffness, as follows: sandstones; siltstones; mudstones; seatearths; coals.

Off-shore

32. The consultants were able to examine mining records and the results of off-shore ground investigations carried out by the National Coal Board (now British Coal) as part of their exploration programme.

33. Early in the studies it was recognised that there could be a potential conflict of interests between the CEGB and the NCB because of the need to locate the cooling water intake structures in an area where mining might take place. The NCB was prevented from mining seams which lie near to the sea bed because of the danger of flooding the workings. Therefore it was important to both the NCB and the CEGB to establish the nature of the coal seams and their depth in relation to rockhead and sea bed so that the limits to mining could be determined. This in turn allowed the consultants to plan the off-shore works, as will be described later in this paper.

34. The data obtained from the off-shore investigation suggests that the stratigraphy and engineering properties of the rocks are similar to those found onshore. There is some evidence to suggest that the Central fault found onshore extends into the off-shore region, but the 1986 results tend to indicate a reduction in throw with increasing distance off-shore. A significant fault with a throw of about 36 m is believed to be located outside the area shown on Fig.3 and at the extremity of the area selected for the cooling water works.

Hydrographic Surveys

35. Competitive tenders were invited for an off-shore survey contract which included meteorological observations, bathymetry, seismic profiling, side scan sonar, wave and tide recording and beach profiling using land surveying techniques. Soil Mechanics Ltd were successful in obtaining this contract.

36. The object of the geophysical survey, using seismic reflection profiling, was to confirm the stratigraphy in the off-shore region in between areas covered either by exploratory boreholes or by mining records. However in this instance the

interpretation of the field data did not correlate well with the available information from other sources and therefore the results were not judged sufficiently reliable to be used either for design or as evidence in a public enquiry.

37. The bathymetric survey, coupled with beach profiling and side scan sonar observations was used to plan the location of intake and outfall structures for the cooling water system and to gain some understanding of the degree of mobility of the sea bed deposits and beach system. The surveys were repeated at intervals over a period of nearly a year. Considerable mobility of the sea bed in the inshore zone was observed, but Hydraulics Research Ltd was able to recommend a limit beyond which the sea bed level variation should not exceed 0.3 m. This limit was then used to establish the sill level for the intake and outfall structures. No evidence of littoral drift was found in the inshore region, the movement of sand being predominantly onshore/off-shore with little evidence of net gain or loss.

38. The contract also produced tide and wave records for the period between off-shore salmon fishing seasons in 1983/84. These measurements were to the CEGB's specification, to supplement the hydrographic survey undertaken by Off-shore Environmental Systems Ltd in 1979 during the initial site selection studies described previously. These data were used in the Board's cooling water dispersion studies and enabled them to determine a spacing between the cooling water intake and outfall structures which would minimise recirculation of heated water.

HAZARD STUDIES

Mining Subsidence

39. The recorded underground mining which took place to the south of the Grange Moor fault gave rise to surface subsidence. Whilst the effects of such subsidence should by now have ceased an envelope of the likely surface effects was constructed. The envelope was based on the recorded limits and depth of mining using the guidelines set out in the NCB Subsidence Engineers Handbook (1975). The closest point on the envelope to the Grange Moore fault was 130 m. It was therefore considered that subsidence had not affected the fault system or the strata to the north of the fault.

40. North of the Grange Moor fault opencast workings had intersected unrecorded mineworkings in the vicinity of Widdrington. The borehole investigation had identified workable seams which were present beneath the site. It was therefore considered that the possibility that unrecorded mining had taken place beneath the site could not be discounted.

41. The collapse mechanisms of shallow mineworkings are well understood and are usually limited by depth to thickness ratio of the seam. The exceptional foundation loading, and the requirement to minimise load-induced settlements, necessitated consideration of unrecorded mining at greater depths than normal. This problem was reduced by the absence of coal seams below the Top of Broomhill horizon arising from a major washout of the seams by a sandstone channel. Attention could therefore be focussed on the possibility of unrecorded mining in the group of four seams encountered in most site investigation boreholes down to 50 m depth.

42. The normal approach to an investigation of unrecorded mining is to undertake a borehole investigation over an area exceeding the area to be occupied by the buildings. This would require boreholes on a close spaced grid pattern which would be pressure tested with cement/PFA grout. The method can be adapted to test four

seams individually by the use of borehole packers. However, the requirement for founding seismically qualified structures on rockhead would result in the excavation and exposure of a large part of the area to be investigated. This offered an alternative to the investigation whereby if no mine entries were identified from inspection of the exposed surface, it would only then be necessary to drill and grout around the perimeter of the excavation to prove the absence of mined roadways.

43. This approach would necessitate construction of a curtain grid of close spaced investigation boreholes around the perimeter with accurate control of verticality in order to maintain the grid at depth. This curtain could be combined with hydrogeological measures proposed for controlling groundwater. The normal method and the curtain drilling approach therefore have differing cost and programme implications and remain open for consideration if it is decided to proceed to the detailed design stage.

44. British Coal's off-shore mining proposals were examined with respect to the siting and design of the cooling water system. It was considered that the potential subsidence effects of the proposed room and pillar workings could adversely affect a tunnel-based system. The implications for a submerged tube system were less predictable but it was considered that the worst case theoretical seabed strains could not be tolerated by a submerged tube. In view of the potential problem the proposed route of the cooling water system was diverted northwards into an area where it was considered that off-shore mining would not take place due to the shallow cover on the Bottom Yard seam.

Spontaneous Combustion

45. The phenomenon of spontaneous heating, leading to combustion, is known to occur in broken coal exposed to limited quantities of air. The reaction involves the oxidation of coal surfaces in the presence of moisture. The reaction is exothermic and consequently, under the right conditions of restricted airflow, heat builds up and the reaction accelerates. Ultimately the temperature reaches a level where spontaneous combustion takes place. Once this stage is reached the process is very difficult to reverse.

46. Spontaneous combustion is more usually associated with poorly compacted coal stockpiles, coal contained in ships holds during transport and broken residual coal left in mineworkings. The presence of shallow coal seams within the Druridge Power Station site, the presence of backfilled opencast workings, together with the possibility of unrecorded mineworkings, merited consideration of the susceptibility of the coals to spontaneous combustion.

47. The properties of the coal were investigated by a consultant coal scientist, Dr DA Hall. Samples of the coal seams were selected from the borehole cores for analysis of coal quality and rank. In addition five samples were tested for their oxygen absorption properties using the Static Isothermal Method. The results were compared with previous tests on various coal seams located throughout the UK which had been correlated with susceptibility to spontaneous combustion. The results of the analyses indicated the coals were NCB Rank 702, a relatively low rank which indicates susceptibility to spontaneous heating. The oxygen absorption results gave "A96" values (the absorption of oxygen after 96 hours at 30°C) which averaged 375 ml/100 g. Values in excess of 300 ml/100 g also indicate susceptibility to spontaneous heating.

48. After comparison with previous test data it was concluded that, given suitable conditions, the coal seams underlying the site would be very susceptible to spontaneous heating. Proposals for the southern part of the site were unlikely to expose any coal seams outcropping at rockhead. Construction on the northern part of the site would, however, be likely to expose outcrops of the Top and Bottom Bensham seams. It was recommended that the disturbed outcrop areas should be excavated and removed from site to produce an undisturbed face of coal and roof strata. Whilst a solid coal face presents minimal risk of spontaneous combustion, the coal should be sealed off from contact with air before disturbance and spalling occurs.

49. If the cooling water system were to be constructed by tunnelling methods then at least one coal seam would be intersected. Similar precautions should be taken to seal off surfaces of exposed coal and to infill any voids behind the lining system, particularly where the seam lies above soffit level, so that excessive loosening and spalling of the coal is avoided.

50. Potential coal-air contacts elsewhere within the site could in theory occur where coal seams intersect opencast areas around Radar South and Druridge and, if there are any unrecorded mineworkings, within the mined voids. It is considered that any coal surfaces above the present water table are in a stable oxidised condition and that any potential contact with air is very restricted. It follows that activity which is likely to change this situation or create new coal-air contacts during construction works, either by excavation, dewatering of the site or by borehole drilling, should be undertaken with an awareness of the potential for spontaneous heating. Measures to seal-off any such contacts should therefore be taken at an early stage to minimise the oxidation process.

Methane

51. Coal Measures are known to contain natural hydrocarbon gases, primarily methane, within the joints and fissures. The methane has resulted from burial of carbonaceous materials in the Coal Measure sequence which causes an increase in heat and pressure. This process leads to coalification whereby the methane-rich volatile content is driven out of the carbonaceous material into fissures within the coal seam.

52. Methane has been widely encountered in coal mining operations and all underground mining is carried out under flameproof conditions. Flameproof equipment and techniques would have to be used throughout the underground construction works associated with the cooling water system.

53. Whilst methane is predominantly associated with coal seams, where it tends to be contained by the predominantly argillaceous roof and floor strata, the occurrence of interconnected faults, joints and fissures throughout the Coal Measures allows the gas to disperse into the surrounding strata. The possibility of gradual migration of low concentrations of methane to the power station foundations, particularly where they are to be founded on rockhead, will therefore have to be considered and adequate precautions taken.

54. The main problem envisaged would be a situation where low concentrations of methane entered a foundation structure through a structural joint and accumulated within an unventilated air space. Methane concentrations in air which exceed 5 per cent by volume form an explosive mixture, but concentrations in excess of 1 per cent within structures are considered to require action. The problem can be readily avoided by monitoring and adequately ventilating air spaces. Since the site

studies methane resistant membranes have been utilised to seal building foundations, and this might be considered for application at Druridge.

Flooding

55. A search of local history records failed to locate any references to flooding in the Druridge area, although frequent references were found to flooding in Newcastle, Morpeth and Alnwick. This negative result may be because a flood at Druridge is not as newsworthy as in a more populated area. The CEGB had commissioned Wimpol Ltd to conduct a study of extreme sea levels for various potential sites, including Druridge, and therefore the consultants' study was concentrated on flooding from surface water flows in combination with the marine flooding predictions obtained from Wimpol's study. Dr PS Kelway of Walton Systems was retained by the consultants to undertake the hydrological studies, which used techniques developed by the Institute of Hydrology, described in the Flood Studies Report (1975) and subsequent publications.

56. For comparison purposes five methods of predicting the Possible Maximum Flood (PMF) were used. There has been considerable discussion about the problem of determining the PMF since the Flood Studies Report was published. Large variations in the calculated value for the PMF occur, depending upon the assumptions made. Although the effects of combined freshwater flooding and marine flooding were studied using an upper envelope value of the PMF it was considered that it would have been more satisfactory to adopt a Design Maximum Flood derived for a finite probability level.

57. A plan showing the estimated area of inundation was prepared to illustrate the conclusion that a ground floor level of +8.5 m OD would give adequate security against the flooding hazard from even the most extreme flood predicted using the PMF envelope.

58. An examination was also made of the risk that flooding would affect road or rail communications in the area; locations were identified in the region where problems might occur.

CONCEPTUAL DESIGN

Site Layout

59. The basic power station layouts were produced by the CEGB (GDCD) with the intention to adopt a standard layout for all the main buildings for all the sites being currently considered in the UK. Both an AGR and a PWR design was included in the studies.

60. The layout of the power stations within the Druridge site will be determined by two geological factors and one man-made constraint, namely:

depth to satisfactory foundation strata;
proximity to significant faults;
the presence of backfilled opencast mine workings.

61. The relatively shallow depth of the drift (approximately 11 m) over much of the site allows heavy seismically qualified buildings to be founded through the drift onto the rock. The drift is an overconsolidated glacial till which will be capable of allowing lighter buildings to be founded on it.

62. The main conclusion of the 1979/80 initial site investigation was that one station could be located east of the buried valley at the southern end of the site and one, or possibly two to the west of the buried valley. The present investigations have confirmed that the southern site is viable. The rockhead beneath the southern power station site is a plateau at approximately -2 m OD. The sandstone is classified as moderately strong and overlies other strata classified as between moderately weak to moderately strong, thus providing a suitable founding stratum.

63. On the western side of the buried valley, rockhead is again relatively close to the surface, but the sandstone which overlies the coal seams is thinner. The area over which a suitable founding level, with a sandstone depth of cover over the coal seam of at least 3 m is limited and outwith this area it is considered necessary to found beneath the two closely spaced coal seams and their seatearths. Due to an increase in plan area compared with the earlier station layout and the need for adequate clearances for construction only one station could now be located on the west side of the buried valley.

64. Faults are known to exist on the site. For preliminary planning and layout purposed it was advised that for seismic considerations, safety related plant should not be located closer than about 100 m to a significant fault.

65. It is also desirable to avoid locating heavily loaded foundations over significant faults in order to ensure that settlement characteristics are reasonably uniform across the foundation.

66. Taking the above considerations into account the location of the northern station in a two station development was examined in an attempt to minimise rock excavation and suit other criteria set by station layout requirements. A total of eight different locations were examined for the northern station; some were rejected because they would involve encroachment beyond site boundaries whilst others would have left insufficient space during construction between the two stations or between the station and the north or west boundary. The selected locations for the northern and southern stations are shown on Fig. 3.

67. The base of the opencast mine workings follow the dip of the strata and the depth of the pits are determined by the seams mined. It was recommended that no significant structures should be founded on the backfill, and thus the perimeter of the opencast mine workings effectively constrain the areas available for development on both the northern and southern power station sites.

68. The factors which control the selection of the power station levels are:

(a) The station level must be high enough to avoid inundation by fluvial or marine flooding.
(b) The level should be such that the foundations of the more heavily loaded buildings are in competent strata with an adequate thickness of these strata between the foundations and weaker underlying strata such as coal seams.
(c) The cut and fill requirements should be made to balance as nearly as possible.
(d) In the case of stations with non-recirculatory open circuit cooling water system, the level of the condensers above mean sea level influences pumping costs.

Foundations

69. The conceptual design of the main building foundations was approached in four stages as follows:

(a) Consideration of the type of foundation appropriate for seismically qualified structures.
(b) Selection of geologically suitable founding levels.
(c) Idealisation of the foundation loads, foundation layouts and levels, and geology.
(d) Assessment of the likely settlement, both uniform and differential.

70. To reduce seismic effects concrete raft foundations bearing directly onto rock were recommended.

71. Using an appropriate PWR reactor load of 600 kN/m^2 it was considered that an adequate allowable bearing capacity would be attained provided the rock was moderately strong ie an UCS value of 12.5 N/mm^2 to 50 N/mm^2 (ref.4). As well as rock strength, cognisance was taken of the presence of fractured zones of rock, and clay filled fractures prior to nomination of geologically suitable founding levels.

72. Use of the Boussinesq equations to assess the stress increase in each layer and the use of the 'classical' one-dimensional method of settlement assessment for each layer and their summation to yield the total, drained settlement was considered appropriate. This followed the recommendations of Burland et al (ref.5) and examination of the rock strata to ensure that they complied with the qualifications proposed by Burland et al.

73. The modulus values of each of the rock layers was determined using a correlation of the large diameter plate loading test on the sandstone with high pressure dilatometer (HPD) tests carried out directly beneath the centre of the plate test, as well as using borehole geological moduli (ref 3). Thus correlation was extended to the 150 m deep boreholes located beneath the potential reactor sites, where similar testings had also been carried out. In this manner modulus values for the rock under each proposed station site were assigned.

74. Typically the maximum calculated settlement is about 24 mm under a PWR reactor and 26 mm between the twin AGR reactors on the southern site. Slightly smaller settlements were calculated for the northern site. The maximum calculated tilt across a reactor foundation diameter is of the order of 1/3750 for a PWR on the southern site. Most of this settlement would occur during construction. The predicted foundation behaviour was acceptable in terms of permissible settlement and tilt.

Cooling Water System

75. An open circuit cooling water (CW) system is proposed utilising sea water from Druridge Bay. The layout of the system within the station boundary is based on the generic design for Sizewell 'B' power station with appropriate adjustments to suit the station ground level and sea level at Druridge. The system comprises twin submerged intake structures located about 1.5 km off-shore, a single 5 m inside diameter conduit connecting the intakes to the cooling water pumphouse, which is located within the station boundary fence, CW culverts to and from the turbine house, a surge chamber adjacent to the CW pumphouse and a 5 m dia. outlet conduit leading to a single submerged outlet structure about 0.5 km off-shore. The CW pumphouse

also houses pumps supplying sea water for cooling auxiliary systems which discharge into the surge chamber. This system is repeated for each station.

76. The advantages and disadvantages of construction of the CW conduits in varying combinations of tunnel, submerged tubes and onshore open trench construction were examined in depth before concluding those options which incorporated onshore trenches or off-shore tubes did not present an attractive proposition.

CONCLUSIONS AND ACKNOWLEDGEMENTS

77. The consultants concluded that the site could accommodate two PWR stations of the type currently under construction at Sizewell. It was further concluded that the site could accommodate two AGR power stations of about 1300 MW each, but the arrangement of seismically qualified buildings in relation to known geological faults needed further study. A list of other topics requiring further study was prepared for use if consent to build a power station on the site is obtained.

78. We hope that it will be clear from this paper that the initial and Pre-Application studies were undertaken as a team effort by many people and organisations within and external to the CEGB. The authors were privileged to play a part in this effort and acknowledge the contribution made by their colleagues and by the members of staff of the CEGB, the contractors and other consultants who participated in the project. The authors thank Nuclear Electric for permission to publish this paper and British Coal who allowed reference to the mining records and geological data in their possession.

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7. Recent developments in the methodology of seismic hazard assessment

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SYNOPSIS. To establish design criteria matching the levels of safety appropriate to any nuclear facility required to withstand earthquakes, it is necessary to carry out a site-specific seismic hazard assessment. Since 1982, the Seismic Hazard Working Party (SHWP) has been responsible, formerly to the CEGB and now to Nuclear Electric, for developing and applying an overall methodology for such assessments. The paper describes the praxis of the SHWP and, in particular, the adoption of Bayesian concepts and use of a logic-tree formulation for all aspects of hazard model parameterisation. Although these methods have, so far, been applied primarily to potential power station or storage sites in Britain, they can be adapted for existing facilities (where appropriate hazard probability levels may be higher), or for disposal sites (where they may be lower), and they are applicable in any seismotectonic environment.

INTRODUCTION

1. Whereas earthquake occurrence is a proper subject for scientific study, a seismic hazard assessment is an evaluation, with specific engineering objectives, carried out at a particular moment in the development of that science. Such assessments, therefore, inevitably call for judgements to be made on the basis of incomplete knowledge.

2. Seismic 'hazard' is here defined as the annual probability of occurrence of an earthquake exceeding a given severity. It is one factor in the wider concept of seismic 'risk' which includes the 'value' of the elements exposed to the hazard (e.g. property or lives), and the 'vulnerability' of such elements to damage or loss of function because of the occurrence of the hazard (ref. 1).

3. Particularly in areas with sparse or short earthquake records, some have preferred to retain the so-called 'deterministic' approach to hazard assessment. However, such an approach effectively accepts unquantifiable risk levels and precludes comparison or consistency between sites or between hazards (e.g. earthquake and flood or ground motion and ground rupture).

4. Almost all earthquakes nucleate on existing (often unknown) faults, and almost all fault displacements represent past earthquakes. Depending on the size of the earthquake rupture and its depth of nucleation, various hazardous phenomena can result:

(i) the fault movement can extend directly, or through connecting fractures, to ground level or rockhead causing permanent displacement at that level (i.e. 'ground rupture')
(ii) the fault movement can give rise to strong shaking as the earthquake waves arrive at, or travel across, the surface of the Earth (i.e. 'ground motion')
(iii) the strain relief caused by the fault movement can affect the hydrogeological regime in the volume of crust which surrounds the fault
(iv) secondary, and even tertiary, phenomena can be caused by any of the above 'primary' manifestations. For example, ground rupture can cause tsunamis and ground motion can cause landslides which, if underwater, can themselves generate tsunamis.

5. The requirement to consider any or all of these hazards depends on the acceptability of their consequences for the particular engineering development.

6. Of these hazards, the engineering implications for power stations are most severe in the case of ground rupture because the vulnerability of safety-related structures to this phenomenon is generally regarded as being almost total, i.e. in effect, 'hazard' is equivalent to 'risk'.

7. For this reason, in the presence of an outcropping fault, the potential for ground rupture cannot be ignored at the levels of safety required of nuclear structures, even in a region of modest seismicity like Britain where, particularly on old bedrock, it is likely that a site large enough for a nuclear facility will either be crossed by, or be close to, one or more geological faults.

8. This paper reviews the more important arguments the authors have used in performing seismic hazard assessments for potential nuclear sites in Britain. It concentrates on the two hazards of ground motion and ground rupture.

THE LOGIC-TREE METHOD

9. In many regions, earthquake occurrence appears to be a stochastic process in space and time, whose very description involves the parameterisation of intrinsic random variables associated with fault rupture and seismic wave propagation. As with all such processes (in particular fluid turbulence, with which seismicity has been compared, ref. 2), the probabilistic approach is best able to offer adequate characterisation.

10. Probabilistic methodologies for ground motion hazard assessment have been developed and implemented in a number of seismotectonic environments. Where the understanding of the seismogenic process is poor (as in Britain), the uncertainties are most appropriately modelled by a hazard logic-tree, whose branches correspond to judgements on parameter values and whose nodes represent points at which a choice of alternatives is made (ref. 3).

11. At every node, probabilities are assigned to each branch. These probabilities represent the likelihood of that branch bearing the correct input parameter value. Since the branch options are mutually exclusive and exhaustive, the conditional probabilities at each node sum to unity. These conditional probabilities are based on the available data and their expert interpretation, so as to reflect the relative degrees of confidence in the respective parameter values held by those engaged in the investigation.

12. Recourse to the judgement of experts is a Bayesian concept (ref. 4) that is an integral part of the logic-tree formalism. De Finetti (ref. 5) has shown that subjective probabilities have a sound mathematical basis, and, in consequence, the techniques developed for assessing them can be made rigorous.

13. Increasingly, logic-tree methods are favoured for state-of-the-art seismic hazard assessments (see, for example, refs. 6 and 7). All non-trivial contributions to statistical uncertainty are represented within the logic-tree: the maximum magnitude, focal depth, b-value and activity rate of each seismic source and the sigma value for each attenuation relationship. Any individual fault sources included in the logic-tree are also weighted according to the likelihood that they are active, to their tectonic style, and to uncertainties in their geometries. All of these contributions cumulatively produce the confidence bands on the expected hazard curve. The lognormal spread in ground motion attenuation is itself an inherent uncertainty which affects the computation of the annual probability of exceedance, but does not affect the confidence limits (ref. 3).

THE ROLE OF EXPERT JUDGEMENT

14. The assessment of seismic hazard is fundamentally a decision-making process, in which expert judgement is exercised in formulating the methodology and in specifying parameter values for hazard modelling. For example, some might argue that it is not important to distinguish between magnitude scales in using an earthquake catalogue or in using attenuation relationships. Right or wrong, such decisions are, in effect, expert judgements and should be recognised as such. The relative importance of expert judgement, compared with expert knowledge based on geological and seismological data, varies regionally according to the quality and quantity of available information. Thus, formal studies of the use of expert judgement in seismic hazard analysis have been pioneered

for the eastern United States, where it is held that uncertainties are greater than west of the Rockies. The use of expert judgement plays a similarly significant role in British seismic hazard analysis.

15. As in the original eastern US studies, expert judgements may be combined analytically through the practice of weighting the probability distributions assigned by individual experts. A number of algorithms exist for combining these judgements, which vary in sophistication from simple averaging and self-weighting, to scoring experts according to performance on test calibration questions. Alternatively, collective judgements may be formed into a consensus through discussion at plenary meetings. The latter decision-conferencing approach, informed by extensive sensitivity studies, has been favoured by the SHWP for resolving methodological issues and assigning probability distributions for parameter values.

DATA ACQUISITION POLICY

16. In practice, the first exercise of expert judgement concerns the generation of the database. Because many secondary, interpreted sources of information, such as maps, catalogues etc., are deficient or 'contaminated', it is necessary to decide whether or not to accept passively an existing database without either re-evaluating that database or actively acquiring new data.

17. Fig. 1 depicts each element of an ideal database in the context of this decision. The choices, first to use data of a certain type and then to move from passive acceptance to active acquisition of such data, are related to 'engineering anxiety': an amalgam of all those technical, strategic and social concerns which influence the engineer who has to make decisions. Some of these factors are:

- seismotectonic environment
- relative importance of the data type in modelling the particular hazard
- quality of existing database
- annual exceedance probability appropriate to the facility (itself dependent on the vulnerability and value of the facility and on the consequences of failure)
- lifetime of the facility
- cost of acquiring new data
- regulatory requirements
- public concern

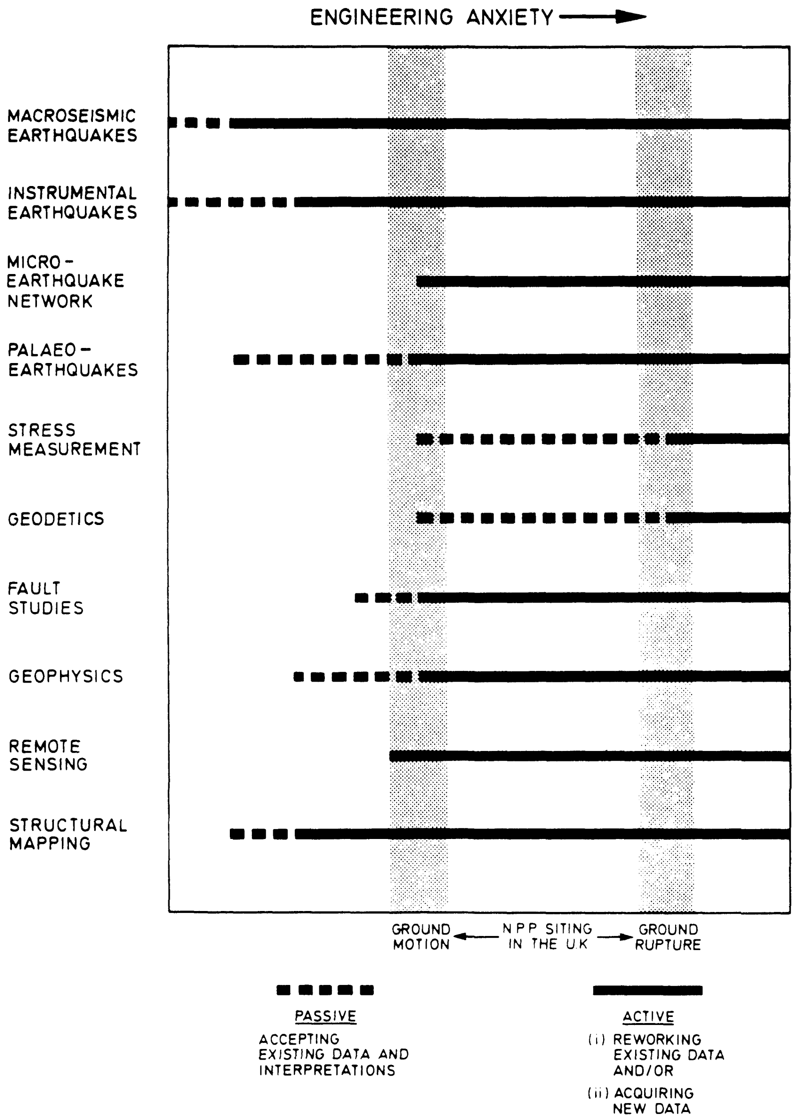

Fig. 1. Active/passive data acquisition

18. At a low level of 'anxiety', the engineer may decide that ground motion hazard is adequately calculated from some simple treatment of an existing catalogue of historical earthquakes with little attention paid to any other data. At the other extreme, as, for example, in studying the hydrogeological seismic hazard for a long-term high-level waste repository, observations of regional and local stress and strain assume high priority while historical earthquakes probably become of far less consequence.

19. In general, as one moves further away from the site, the passive acceptance of existing information becomes more acceptable.

20. Decisions should respect the need for all the pertinent hazards at a particular site to be assessed coherently from the same database. Wherever possible, computational models must be 'data-driven' and there should be no inconsistencies between parameter values used to calculate the probabilities of ground motion and of ground rupture.

21. For NPP siting studies, decisions are conditioned by regulatory practice (e.g. ref. 8) requiring assessments to be carried out on a site-specific basis.

INVESTIGATIVE STRATEGY FOR POTENTIAL NPP SITES IN BRITAIN

22. The stages of a site-specific investigation, from data collection to hazard calculation, are shown in Fig. 2 and the following paragraphs discuss some of the more important aspects of this process.

Assembling the data: earthquakes

23. The problems which can be introduced by using previous interpretations of earthquakes, both macroseismic and instrumental, are numerous: the St. Albans earthquake of 27 December 1768, described by Davison (ref. 9), translates into a landslide in Herefordshire two days later and a single earthquake off Pembroke on 20 February 1247 gave rise to a flurry of events reported by various secondary sources as occurring in the mid-thirteenth century (several of which are associated with London and with St. Valentine's day), see ref. 10. As demonstrated by Ambraseys (ref. 11), instrumental earthquake catalogues often suffer from comparable errors which are again only revealed by detailed re-evaluation.

24. Apart from the active reappraisal of known earthquakes and the seeking of 'new' historical or palaeoseismic events, the only action that can be taken to extend the seismological database comes from installing microearthquake networks, as used by the SHWP at Hinkley Point, Wylfa, Trawsfynydd, Dungeness and Heysham (ref. 12). Such instrumentation serves a multiple purpose in that it can:
(i) discriminate even very small genuine earthquakes from man-made events; (ii) yield accurate information on the distribution of small events in the seismogenic crust, and (iii) contribute to the understanding of the crustal stress tensor.

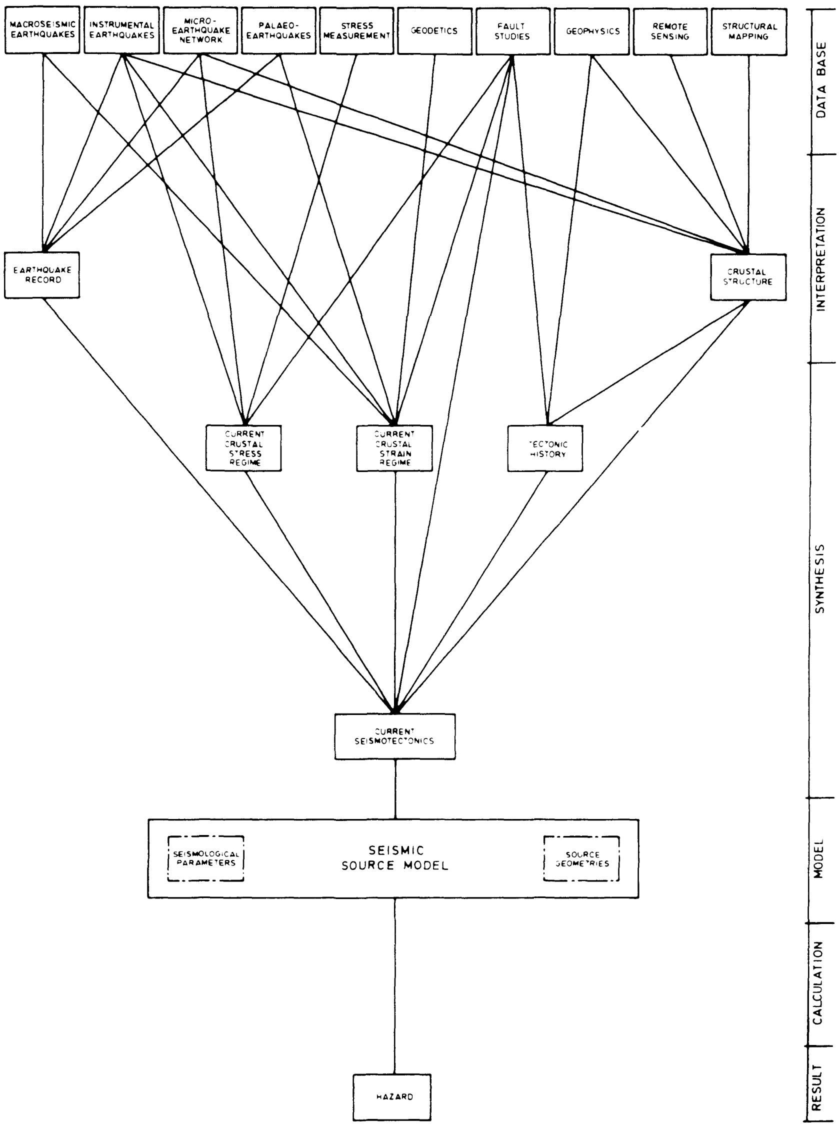

Fig. 2. Information flow for site-specific seismic hazard assessment

Assembling the data: geology

25. Problems also exist with secondary geological information: geological maps are essentially interpretative documents, compiled partly by inference, because only some of the information shown will have been directly observed. They are subject to revision as knowledge, especially of sub-surface geology, accumulates. Most geological maps demonstrate an association between the amount of detail recorded and the proportion of bedrock exposed and the representation of faults is usually very incomplete Out of seven sites considered since 1982 for possible nuclear power stations in England and Wales, only two lie within areas mapped to modern standards.

26. Structural interpretations appearing in the scientific literature must also be critically evaluated. This has been helped by the introduction of 'fault data sheets' which require the geologist to study chronologically all the literature concerned with a specific geological structure and, thereby, separate observation from inference. Examples of 'faults', found under critical scrutiny to be unsubstantiated, are the 'Cannington Thrust' 7km south-east of Hinkley Point and the 'Colchester Fault', a hypothetical north-east trending structure passing about 10km north of Bradwell and traditionally associated with the 1884 Colchester earthquake.

27. Where significant geological data are deficient, it is necessary to acquire additional information by outcrop investigations, by the interactive use of satellite and aerial imagery, and by interpreting commercial or public domain geophysical (primarily seismic reflection) data. In particular in the site vicinity, new seismic reflection profiles may have to be shot as part of the investigation.

Assembling the data: ground investigation

28. The implications of faults, for both ground motion and especially ground rupture, have placed significant new demands on the way in which site geology is investigated, in particular where the subcrop is relatively deep or otherwise inaccessible. Recent experience of investigations for twelve new facilities has revealed (with some notable exceptions) civil engineers slow to rise to these new investigatory challenges, many of which have more in common with petroleum reservoir exploration and development than with traditional borehole-dominated, bonus-related, ground investigation practice. State-of-the-art geophysical analysis procedures will often be required and there is always a need continually to adapt the investigative strategy in the light of newly acquired information: both requirements demand sophisticated data handling and display facilities (e.g. CAEX).

29. Beyond investigating the position and tectonic history of any site faults, such faults may need to be sampled for microtextural analysis or dating. Particularly for these activities, but also for all other aspects of tectonic understanding, it has proved essential to involve relevant specialists in determining the evolving investigative strategy.

Treatment of uncertainties in the data

30. Only familiarity with primary sources exposes the uncertainties inherent in the database. The problems arising from earthquake location and the nature of evidence suggesting whether or not an individual fault is active are discussed below. There are, however, many other less obvious complications arising from data quality and interpretational uncertainty which need to be recognised and graded against some near-formal scale. Elements as diverse as geophysical seismic reflection sections, lineaments seen on satellite imagery (ref. 13) and earthquake focal mechanisms (ref. 14) have all been treated in this way.

Consequences of timescales in the database

31. One important limitation on knowledge arises from the enormous time-span of the process. While fault rupture in even a major earthquake lasts for a few tens of seconds (about 10^{-6} years), away from plate boundaries, that earthquake is a response to a process which can take over 10^6 years to reveal its full complexity. Seismic hazard assessment must recognise the time-scales involved, not only in the earthquake generation process itself, but also in the various elements of the database which have to be explored to understand that process. Typically, seismological information spans up to 10^0 or 10^1 years for microseismic networks; up to 10^2 years for instrumental earthquakes and up to 10^3 years for historical earthquakes. In Britain, geodetic surveys have been relevelled (for some areas) after 10^2 years, geomorphological evidence is generally (outside southern England) restricted to just the Post-glacial period (10^4 years) leaving only geological manifestations (extending from 10^6 to 10^9 years) to pass beyond the likely recurrence intervals of major fault movements.

32. The significance of these time-scales is demonstrated by Fig. 3 which illustrates the relative capabilities of seismological and geological evidence to reveal individual active faults in areas with differing crustal strain and earthquake occurrence rates.

TREATMENT OF EARTHQUAKES

33. Because earthquake sources are best characterised by instrumental data, it is necessary to convert the principal information resource on British earthquakes (historical Intensity data) into magnitudes consistent with those for the instrumental era. The SHWP methodology for handling historical data is summarised in ref. 15.

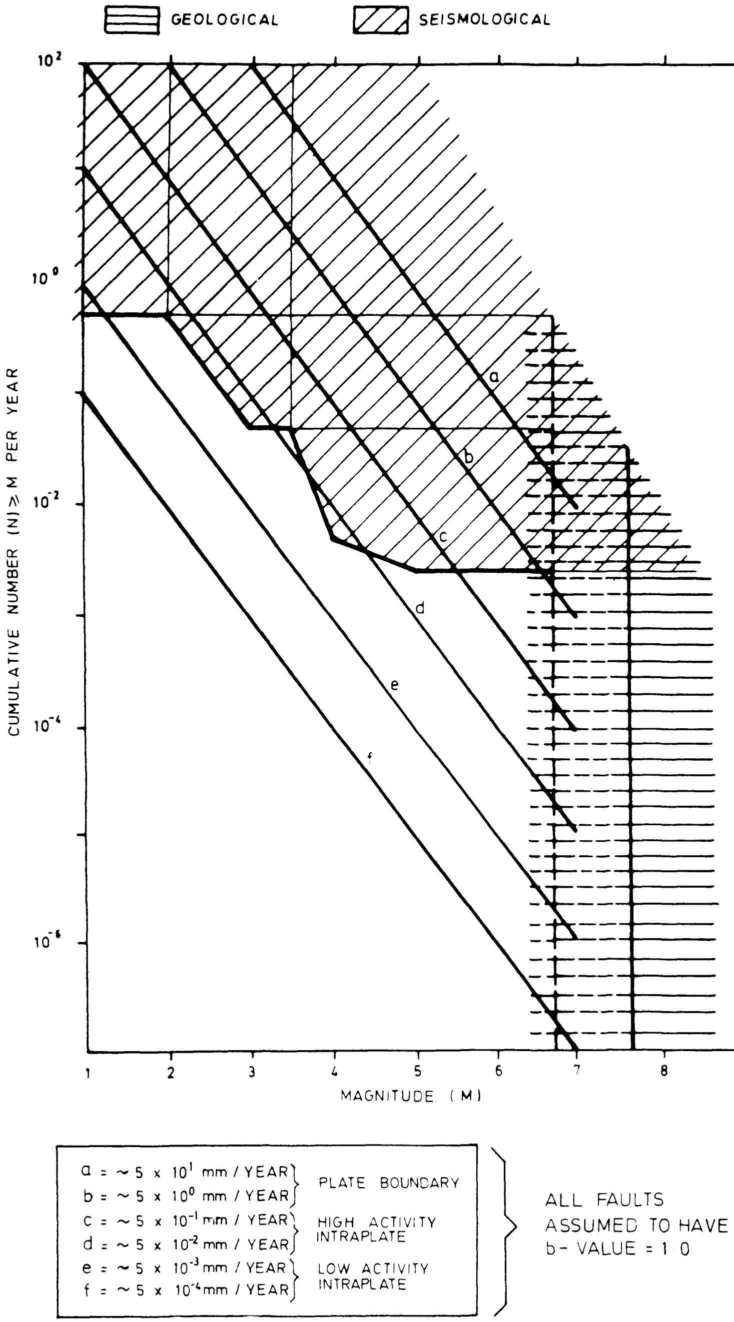

Fig. 3. Fault activity related to typical observational domains

34. In brief, each event is first categorised with respect to the quality of data. The data are then analysed to prepare an Intensity map (using an adaptation of the MSK-scale) from which the three required parameters for the earthquake are obtained:

(i) The location of the macroseismic epicentre ('macrocentre') is estimated and the uncertainty on this location assessed. Associations between earthquakes and specific crustal structures can only be tested using earthquakes whose locations are accurately known.

(ii) The magnitude of the earthquake is derived from magnitude/felt area correlations, first developed by Principia Mechanica Ltd. (ref.16) from a consistent set of instrumental surface wave magnitudes for eleven British events which were later published as part of a wider dataset (ref. 11). The current versions of these relationships provide a magnitude, now described as M_{SA} to make the distinction from a true instrumental surface wave magnitude, as follows:

$$M_{SA} = -0.217 + 0.967 \text{ Log } A_{III} \quad (\text{sigma} = 0.15) \qquad (1)$$

$$M_{SA} = 0.898 + 0.814 \text{ Log } A_{IV} \quad (\text{sigma} = 0.15) \qquad (2)$$

where areas within isoseismals III and IV are expressed in sq.km.

(iii) Focal depths estimated from Intensity information are much less reliable than magnitudes and, therefore, the depth of the earthquake is only broadly categorised, from the 'contours' of the Intensity surface, into one of three depth bands.

35. Other procedures are employed for similarly evaluating and interpreting instrumental earthquakes (ref. 14). Thus, each earthquake in the final site-specific catalogue is provided with a data quality index, a location with a grade of uncertainty, a magnitude and a depth.

Completeness of the earthquake record

36. Seismicity statistics and their geographical variations can only be reliably derived from that part of the earthquake catalogue that is 'complete'.

37. The completeness of any earthquake catalogue must depend on the production and survival of contemporary records. It will also have been affected by the distances between recording centres (which, if too great, might fail to 'capture' intervening events below a certain size), by the state of communications between them, and on the present-day availability of the documentation (see, for example, ref. 17). These historiographic influences vary both temporally and geographically from one region to another.

38. It is possible, therefore, to determine a regional threshold for each historical period above which the retained earthquake record provides a complete dataset, see also ref. 18. A typical threshold plot is shown in Fig. 4. For a coastal site, thresholds offshore will be higher than those onshore. Similar analyses of completeness thresholds are carried out on the instrumental database (ref. 19).

TREATMENT OF FAULTS

39. A nearby active fault can have a significant effect on site ground motion hazard while a fault outcropping beneath a planned facility can, even at low activity rates and very long recurrence intervals, have an intolerable impact on site viability.

Definitions

40. All interpretations of active faults, and the very use of the term "active", depend either explicitly or implicitly on some observational time period. However, this time period must not be constrained by any artificial limitation such as the duration of the historical earthquake record, or (as enshrined in some regulatory definitions, e.g. ref. 20) the limit of a specific dating technique.

41. The significance of faults can only be comprehended in the context of the tectonic history of the region in question. Faults that have moved in the present phase of this tectonic history (the 'Current Tectonic Regime', CTR) must be treated as active. Faults that have not moved within the duration of the CTR are deemed 'extinct', see ref. 21.

42. By definition, the CTR has been in operation for as long as current features of stress and strain have prevailed at the regional scale. There is no a priori definition for this period, and it must be estimated by analysis of local geological and tectonic data. For Britain, such an analysis shows that the CTR is no older than 8 million years (ref. 22).

Ground motion hazard

43. Active faults need only be modelled as distinct sources of ground motion hazard where the combination of their activity rate and location perturbs the results from assumptions about the random distribution of earthquakes through the crustal volume which surrounds the site. At the probability levels relevant for the design of NPPs in Britain, it can be shown that beyond 30km from the site ground motion hazard levels are not sensitive to the inclusion of fault sources (Fig. 5). Even within this range, for realistic activity rates, the addition of fault sources makes a relatively small contribution to the 10^{-4} p.a. ground motion hazard, except when they are very close to the site.

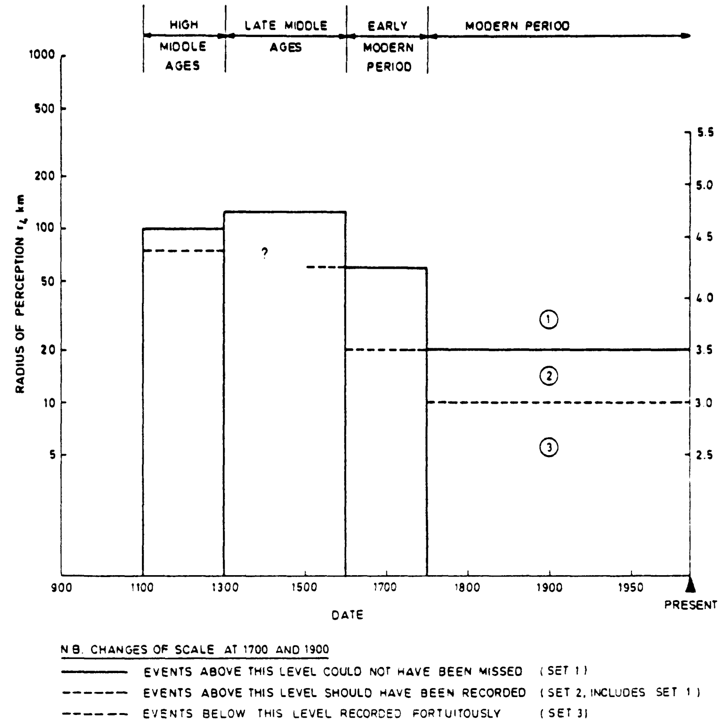

Fig. 4. Regional completeness thresholds for historical earthquakes

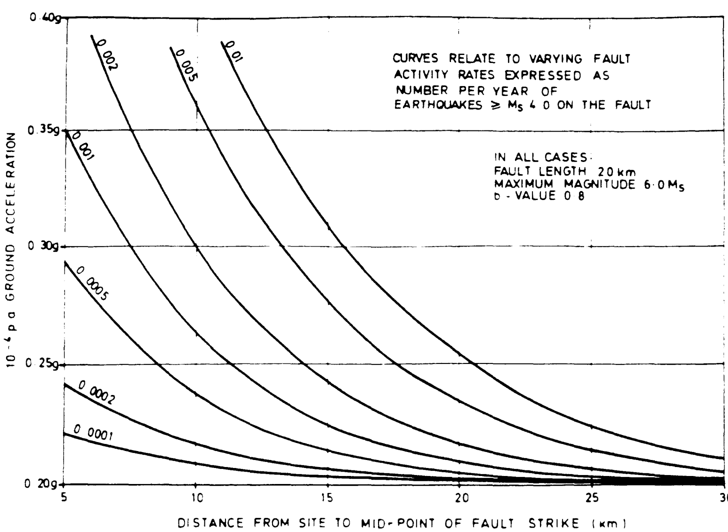

Fig. 5. Sensitivity of 10^{-4} p.a. ground motion hazard to addition of a fault source

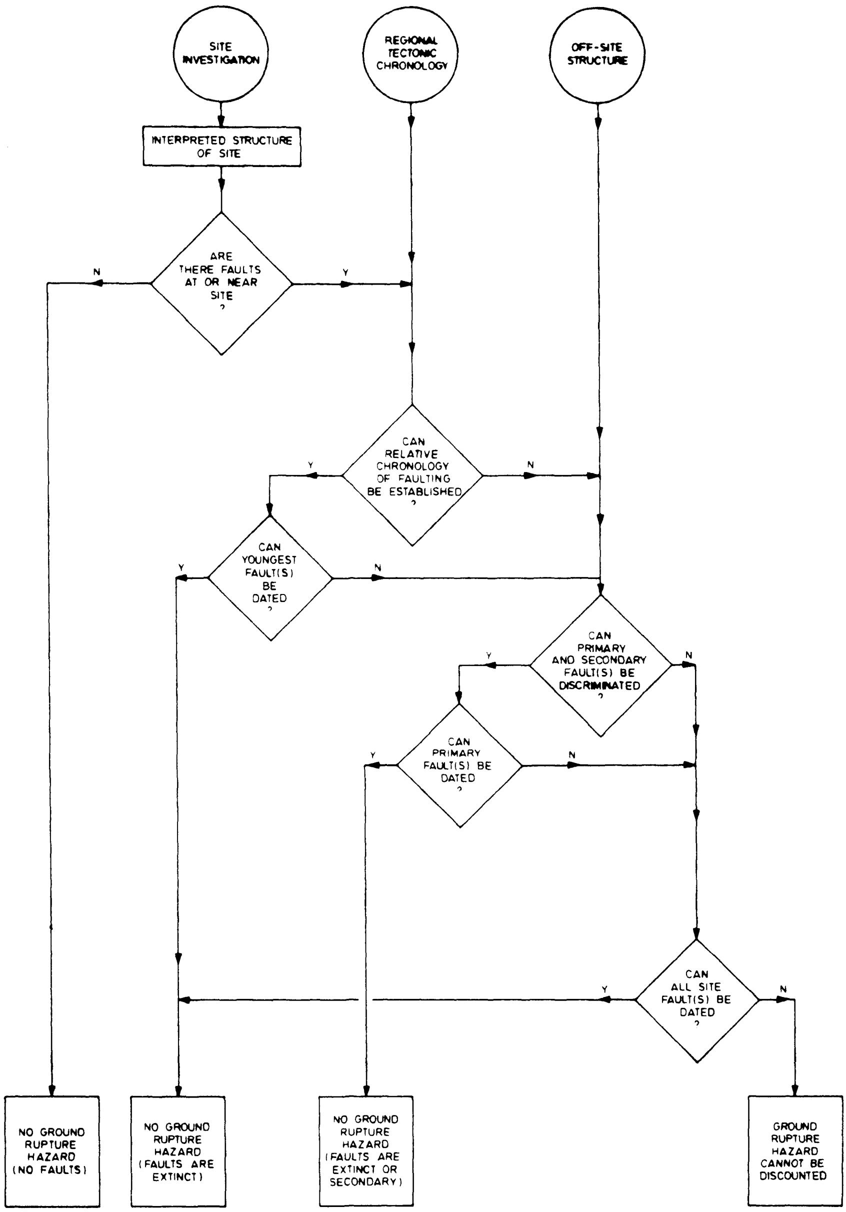

Fig. 6. Flow-chart for fault dating studies

44. To preclude the modelling of local faults, it is necessary to show that no geological or seismological, data exist to suggest that they could be active. However, such information is all too often ambiguous because of uncertainties in the location of seismological events or in the ages of geological materials. It then becomes necessary to employ fault dating techniques (see below) to show that the fault has not moved under the CTR. Failing this, the fault must be included, as a weighted seismic source, in the hazard model.

Ground rupture hazard

45. Wherever possible, critical facilities should be sited so as not to straddle faults. Unless a surface fault can be demonstrated not to have moved under the CTR (i.e. the fault is extinct), ground rupture cannot be discounted. The hazard can be quantified, but only with a low level of confidence. Depending on their proximity, the presence of such faults may prove to be an exclusionary criterion for nuclear development.

46. In studying near-site faults, detailed field examination is required to seek both chronological or cross-cutting relationships (to identify the youngest fault or faults) and kinematic relationships (to discriminate primary faults from secondary structures which cannot move independently). Experience has shown the difficulty of successfully achieving unambiguous interpretations in areas of structural complexity: the requisite procedures are idealised in the flowchart presented in Fig. 6.

47. Ultimately, as shown on Fig. 6, it is necessary to have some 'absolute' date of last movement to compare with the age of the CTR. At its simplest, 'direct' dating requires the identification of a geological material (such as an igneous dyke, hydrothermal vein or overlying undisturbed geological formation) that demonstrably postdates the last episode of displacement. This material is then dated: a process that can be relatively straightforward for igneous rocks but more problematic for veins and unconsolidated geological deposits. (Radiocarbon and U-series methods have been used by the SHWP for this purpose, see ref. 23). In the absence of such material it becomes necessary to attempt to date some component of the fault gouge itself.

48. SHWP studies have pioneered investigative techniques for the direct dating of fault gouge by isolating and radio-isotope (K-Ar) dating sub-microscopic mineral grains that developed subsequent to the final phase of movement (ref. 24). Other techniques employed include palaeomagnetic studies of the orientation of the remanent magnetisation of minerals formed in the gouge (refs. 23 and 25), and electron-spin resonance investigations to show the age when certain mineral grains were last 'shocked' in fault displacement.

49. Without direct dating information, the status of an on-site fault would be prey (throughout the lifetime of the facility) to changes in the regional or site database.

MODEL CONSTRUCTION

50. It is nowhere possible to know enough about all earthquake sources to characterise seismotectonic understanding solely in terms of active faults.

51. The crust generates a wide range of earthquakes, from those corresponding to fracture dimensions of a few metres to 'crustal' earthquakes involving the rupture of the entire seismogenic layer. As the fracture dimension becomes smaller so the association of an earthquake with any one of the increasing number of candidate crustal faults becomes less certain. Imprecisions in the location of both seismic events and subsurface geological structure further obscure this association. Hence, in any seismotectonic synthesis it is necessary to invoke 'active crustal volumes', which are considered to generate those earthquakes not attributed to known active faults.

52. The concept of active crustal volumes is not merely a device for handling uncertain data. For much of the Earth's crust, randomly distributed background seismicity truly reflects distributed strain release.

53. In practice, scientific endeavours in slowly deforming areas generally do not provide sufficient data to define fully the parameters of either active crustal volumes or active fault systems. Under these conditions the engineering requirements of seismic hazard assessment are satisfied by a 'seismic source model': a simplification designed, in the present case, to analyse the earthquake exposure of a particular site and, through its logic-tree formulation, able to absorb uncertainties in scientific interpretations. The seismic source model is made up of fault sources and area zone sources, corresponding respectively to the concepts of active faults and active crustal volumes.

54. The simplest seismic source model for ground motion hazard consists of a single area zone source surrounding the site. As statistically significant geographical variations in seismicity become apparent, so this model needs to be partitioned into various area zone sources. As an example of the way in which seismological data can inform partitioning, Fig. 7 displays regional geographical variations in seismic moment release for north Wales.

55. Distant from the site, the need for sub-division and the location of boundaries between area zone sources become of little consequence. Close to the site, however, boundaries should be chosen by expert judgement, using sensitivity studies to balance the available data against the consequences for the site hazard. Such seismic source models are tailored for a specific site location. For any other site, the model boundaries may need to be redefined.

56. While the regional earthquake database is usually sufficient to define activity rates for area zone sources, other seismological parameters (b-value, upper-bound magnitude etc.) often have to be derived from more numerous, consistently treated, datasets collected from larger relevant areas, such as NW Europe. At low probability levels, tectonic information assumes increasing importance in determining activity rates.

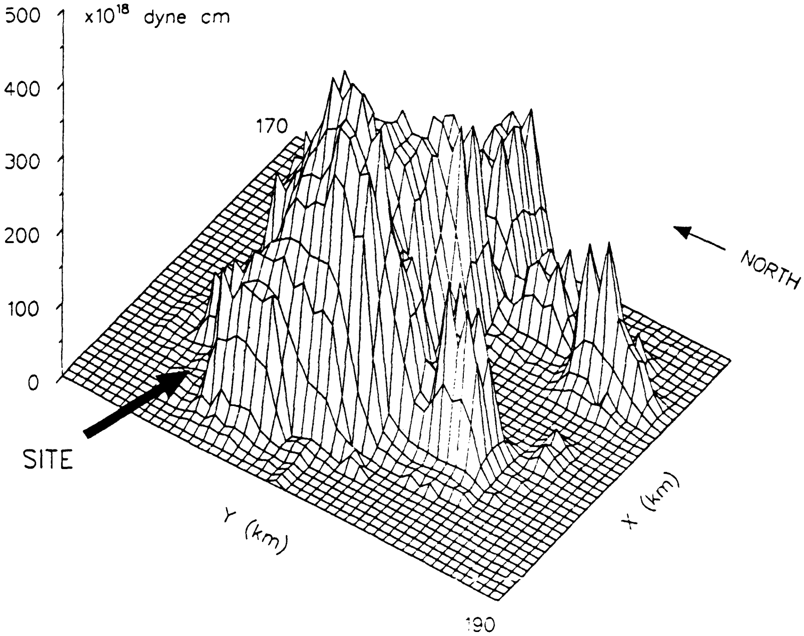

Fig. 7. Contoured cumulative seismic moment release

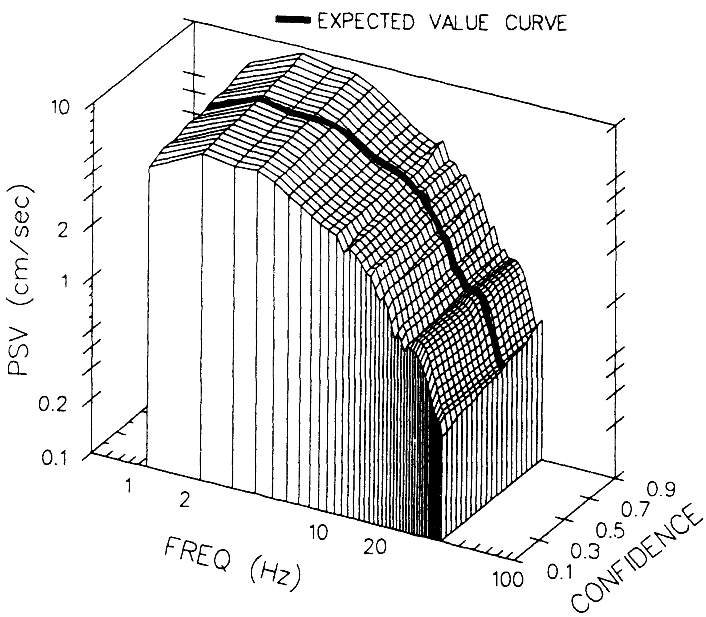

Fig. 8. Uniform risk spectra: 10^{-4} p.a. horizontal motion

57. Further refinement in the definition of the seismic sources, from detailed seismological or geological information, would hope to pass some way towards a model with reliable fault sources as well as area zone sources.

58. However, faults can only be introduced into ground motion seismic source models as and when their activity becomes manifest by the occurrence of earthquakes or revealed by the uncovering of geological evidence for their displacement under the CTR. The acquisition of such evidence depends on the intensity and duration of the investigations, as well as on the seismotectonic environment, and practical constraints ensure that only an incomplete picture of active faults can be drawn. However, in an area deforming as slowly as Britain, the first faults to be recognised as active will not necessarily be the most active faults.

59. The seismological parameters attributed to fault sources will usually have to be assessed from geological data.

60. The seismic source model for enumerating ground rupture hazard consists of a single fault source. In such a calculation, the ground motion attenuation relationship is replaced by one relating rupture area to earthquake magnitude.

61. All SHWP hazard computations, including sensitivity studies, have been carried out using the program PRISK (ref. 26).

THE RESULTS OF SEISMIC HAZARD ASSESSMENTS

62. Where required, the ground rupture hazard is expressed as the best-estimate curve linking displacement at rockhead or at ground level to its annual probability of exceedance.

63. The ground motion hazard can be expressed either as the best-estimate curve linking free-field peak horizontal acceleration to its annual probability of exceedance (preferably supported by a family of similar curves at constant confidence levels), or as uniform risk spectra, see Fig.8.

64. In either case it is important that ground motion hazard curves are accompanied by the results of sensitivity studies exploring the consequences of plausible post-assessment data, for example, the occurrence of a significant regional earthquake or the discovery of evidence of activity on a nearby fault.

65. The methodology described in this paper has been applied, with many sensitivity studies, to potential NPP sites in Britain. Robust results for site-specific peak horizontal accelerations at 10^{-4} p.a. probability of exceedance have ranged from less than 15%g to more than 25%g.

ACKNOWLEDGEMENTS
The methods described in this paper have been developed with the active participation of all the members of the SHWP. The authors are pleased to acknowledge their many contributions and, in the particular context of this paper, must recognise the assistance provided by Willy Aspinall and Gordon Woo. The authors are grateful to Nuclear Electric for permission to publish this paper.
This is SHWP contribution no. 47.

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Discussion

G. TROTT, W.S. Atkins Consultants Ltd, Warrington
The seismic hazard studies in Paper 7 are aimed at producing curves of peak ground acceleration against probability of exceedance. What consideration has been given to the form of the ground motion which was used in subsequent structural analysis/design; in particular in terms of frequency content and time for strong motions.

Is the use of artificial time histories generated from a design spectrum a reasonable or unduly conservative approach?

P. ISGAR, Nirex Ltd, Didcot
How long did the ground remain frozen after the release of freeze? What power was required to drive the refrigeration plant? How quickly could the associated surface works be built in the freeze-affected area?

G. VACIAGO, Rendel Geotechnics, London
The evidence on fault activity and seismological observation presented in Paper 7 (Fig. 3) suggests that as well as encountering the difficulties with proving fault activity suggested in the paper, engineers may find the existing database inadequate for a statistically significant analysis of hazard at a site in terms of ground motions. Would the Authors comment?

B. LEACH, Allott & Lomax, Manchester
I wish to raise the question of economics in investigative work. The initial role of a sound understanding of the structural geology at these sites has been highlighted by David Mallard. At some sites the establishment of the geology would be relatively easy; at others it would be extremely impressive to reach a satisfactory understanding, and at the limit there will be sites where it would be virtually

impossible to do so. Where do the economics take over, where do you draw the line on expenditure?

I. P. FORBESTER, SDRC Engineering Services Ltd, Warrington

Does Mr Chaplow feel that the selection of the Dounreay and Sellafield sites is based on good geological grounds, or has it been based more on areas that would be sympathetic to a repository being built near them? As a result of this, is not the industry being forced into a much higher cost option with more engineering difficulties?

B. O. SKIPP, Soil Mechanics Ltd, Wokingham

Advances in site investigation and geotechnics have been stimulated by the nuclear industry since its inception. Witness the work on soft-rock creep and the engineering classification of soft rocks such as the Mercia Mudstone. Now new techniques are being further progressed, but can the site investigation contractors and consultants cope intelligently with the increased amount of geological and geotechnical data? I was impressed by the graphics shown by Mr Isgar to illustrate what can be done. How does Dr Chaplow view this whole issue of information handling and treatment?

D. DE BRUYN, Paper 4

About the freezing groups used during the shaft construction: three groups, each with a power of 250 000 kcal/hour (about 290 kW) were used. The freezing was stopped for the construction of the underground laboratory immediately after the concrete plug (2.50 m thickness) was poured in place (mid 1983). In the whole clay mass around the gallery temperatures were already positive at the end of 1984 (1.5 years after construction), and were stabilizing at the end of 1986 (3.5 years after construction).

The surface works (hoist system, etc.) being limited in size and in foundation area, were built at the same time as the shaft excavation works.

R. CHAPLOW, Paper 5

In reply to Mr Forbester, the Sellafield and Dounreay sites were selected by Nirex on the basis of a wide variety of factors including social considerations. The purpose of the geological investigations being undertaken at the site is to characterize the geological conditions as part of the process which will allow Nirex to select the preferred site.

To Mr Skipp, information handling and treatment is an important aspect of the investigations. Nirex are currently undertaking a study aimed at identifying the requirements for establishing a database for storing and handling geological information. Data being collected at site are routinely being obtained and stored in digital format to facilitate future interpretation.

8. Seismic analysis and design practice for UK nuclear installations

J. P. NEWELL, MSc, MICE, and G. P. ROBERTS, BSc, WS Atkins Engineering Sciences

SYNOPSIS. Many UK nuclear installations that have been designed during the last 15 years have been required to withstand a seismic event. Designers, prime contractors and equipment suppliers have therefore been required to consider seismic loading as a major load case on these projects.

This paper draws upon experience of seismic qualification programmes on a number of contemporary UK nuclear installations. The various methods available for the seismic qualification of civil works are explained and guidance is given on the choice of appropriate qualification methods. Approaches to the qualification of M&E equipment are also summarised.

SEISMIC DESIGN PHILOSOPHY

1. The seismic design philosophy for UK nuclear installations is founded on a probabilistic approach to the assessment of the seismic hazard. A deterministic approach is used to define the seismic loading and response of the structure. The structure is then designed to meet defined performance criteria.

Risk assessment

2. New nuclear installations are designed to withstand an earthquake with a probability of exceedance of 10^{-4} per annum. This is commonly known as the 'safe shutdown earthquake' (SSE). The requirements for power reactors are described in ref. 1. A safe design is ensured by a conservative calculation of structural response and an assessment of stresses against code allowable values. Ref. 2 is an example of this philosophy.

3. Existing plant which was not seismically designed is generally assessed on a case-by-case basis having regard to the possible consequences of failure. The available design margins inherent in the structure are identified and a justification for the safe performance of the structure is formulated. Although the method has not yet found commonplace application, the concept of 'constant risk' or 'constant probability of failure' (ref. 3) may be helpful.

Safety case preparation

4. A structured approach is taken to the justification of the safety of nuclear installations and this is contained in the 'safety case'.

5. Owing to the vulnerability of structures to seismic loading, it is important that a detailed safety case is prepared at an early stage. A programme of timed safety case submissions is required by the independent assessment authority at each stage of design release.

6. Designers and contractors can therefore expect to be required to prepare detailed qualification reports which describe their approach to seismic qualification. These are independently assessed to ensure that an appropriate approach has been adopted.

Performance requirements

7. It is particularly important in seismic design to ensure that the performance requirements of structures are clearly defined and this should ideally be completed early on in the project.

8. Performance criteria are generally defined on a project-specific basis. In most cases, structural performance requirements fall into one of the following categories

(a) elastic design
(b) limited ductility
(c) no structural collapse
(d) no seismic requirement.

9. The majority of new nuclear installations are designed to elastic stress limits at the SSE level.

Load combinations

10. Seismic analysis is often regarded as a specialist activity which cannot be covered by the conventional design office. The treatment of the seismic loadcase should however be an integral part of the design process and the seismic loading should take its place alongside the other types of loading that the structure is required to withstand. Appropriate load combinations must be considered and these are defined in the design codes of practice.

Design codes

11. British design codes of practice do not currently provide specific guidance on seismic design. American codes tend to be used for UK nuclear installations and these include ACI 349 and ASME III.

12. British codes, such as BS 8110 and BS 5950, may be applied, particularly where an elastic design is required at

SSE level. American codes offer the advantages of defining load combinations which include seismic loading and providing guidance on design for ductile structural response.

Beyond-design basis events

13. The SSE forms part of the normal design basis. Reserve strength is required beyond this level to ensure that the structure does not suffer failure 'just beyond the design basis'. This is sometimes referred to as the 'cliff-edge' or 'seismic margin earthquake' condition. Once designed to the SSE level, the structure is then checked to identify the seismic design margin. A safety case is often made if an available margin of 40% on earthquake ground motion can be demonstrated. See, for example, ref. 2.

SEISMIC QUALIFICATION OF CIVIL STRUCTURES

14. The seismic qualification of civil structures is carried out by analysis. Generally, the simplest method should be used that will adequately predict the seismic response. Almost invariably, trade-offs are made between the level of modelling detail and the appropriate resources that are available within the overall project programme.

Modelling aspects

15. It is essential to define at the outset the scope and level of detailed design information that will be required from the seismic analysis. For example, seismic design forces in the seismic load resisting system may be required from the primary structure analysis, whereas secondary structures may require floor-level accelerations for localised design purposes. These considerations determine the type of mathematical model of the structure that will be required.

16. A prime objective is to represent the global mass and stiffness distributions of the structure, since the global dynamic response of the structure is governed by these factors. The dynamic characteristics of secondary structural elements tend not to affect the global seismic response and these may generally be omitted from the structural model. Beam finite element models are most generally used, but detailed plate finite element models of complete buildings have also found application (ref. 4).

17. In most cases, the best approach to structural modelling is to utilise a two-stage procedure (refs. 5 & 6). In the first stage, the global seismic behaviour of the structure is represented and in the second stage the localised response of structural elements is modelled. This is sometimes known as the global/local modelling approach. An example of this approach is illustrated by the global (multi-stick) model of a complex concrete shear wall building shown in Fig. 1.

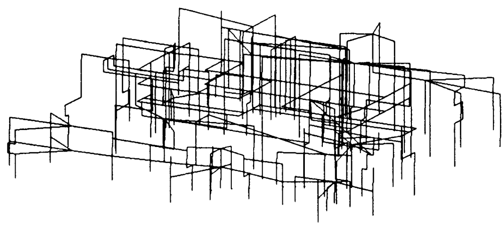

Fig. 1. Stick model of shear wall building

Analysis methods

18. Seismic analysis is carried out by one of the following methods:

(a) static analysis
(b) equivalent-static analysis
(c) dynamic analysis

19. Static analysis is the most conservative approach. The acceleration response of the structure is calculated from the peak of the ground response spectrum and is multiplied by a 'static coefficient', typically 1.5. Seismic forces are then calculated by multiplying the structural mass by this acceleration.

20. Equivalent-static analysis treats the structure as a single degree of freedom system. The natural frequency of the single degree of freedom system is calculated and the acceleration is determined from the ground response spectrum at this frequency. Seismic forces are calculated by multiplying the structural mass by this acceleration. A static coefficient may also be applied.

21. Dynamic analysis may be carried out by hand calculations, finite element modelling or problem-specific computer programs. Analysis may be carried out by response spectrum or time-history methods and may be linear or non-linear.

22. Most commonly, a linear finite element analysis using the response spectrum method is employed. This approach is generally the most efficient of the methods mentioned here in producing realistic seismic design forces in a form suitable for the designer.

DEFINITION OF THE DESIGN GROUND MOTION

23. The design ground spectrum in the UK has been defined as standard response spectra for hard, medium and soft generic

site types (ref. 7), scaled to a peak ground acceleration (pga) value from seismic hazard studies.

24. The design ground motion is taken to apply at ground surface level in the free-field; ie distant from any buildings.

Hazard curve

25. The seismic hazard curve, derived in the seismic hazard studies, relates pga to probability of exceedance. The 10^{-4} per annum (SSE) design level horizontal pga lies in the range from 0.15g to 0.25g for the UK (ref. 7).

26. Design horizontal pga values of 0.20g and 0.25g have been used in the UK, together with a vertical component equal to 2/3 of the horizontal component.

Response spectra

27. The design ground motion hard site spectrum scaled to 0.25g pga is shown in Fig. 2 on tri-partite plot axes.

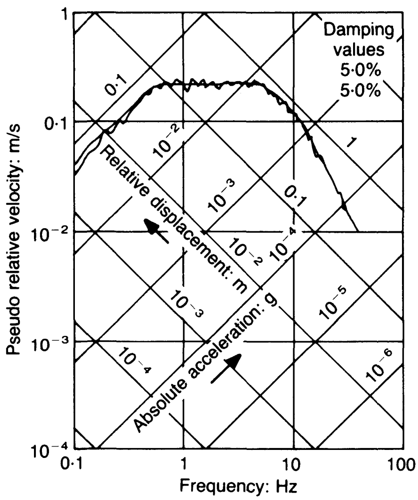

Fig. 2. UK hard site response spectrum scaled to 0.25g

Accelerograms

28. Artificial accelerograms can be generated to provide ground motion data for input to time-history analyses of the structure. Three independent accelerograms are required for the three coordinate directions. Suitable accelerograms are generated with computer programs, such as THGE. A typical artificial accelerogram is shown in Fig. 3, which matches the spectrum shown in Fig. 2.

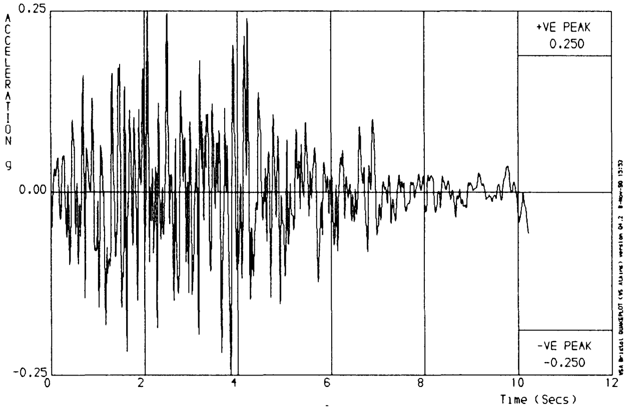

Fig. 3. Artificial UK hard site accelerogram

29. It is common practice to use a single set of 3 artificial accelerograms which envelope the design response spectrum, to carry out a structural analysis (ref. 8). For complex 3D structures, particularly if a non-linear analysis is performed, it is prudent to check the sensitivity of the seismic response by using additional accelerograms.

SOIL-STRUCTURE INTERACTION STUDIES

30. Geotechnical conditions at the site can induce a modification of the design ground motion. Additionally the presence of a massive building can affect the ground motion in the vicinity of the building. In such conditions, a site response analysis and a soil-structure interaction (SSI) analysis is normally required to define a modified input ground motion. These types of analysis are described in refs. 9 & 10 and the general principle is illustrated in Fig. 4.

31. In the case of a hard rock site, neither type of analysis is generally required and the free-field ground motion may be input directly to the structural analysis.

Site response

32. In the site response analysis, the free-field ground motion is applied to the top of a 1D shear beam model of a geological section through the site, as shown in Fig. 4. The shear modulus and damping ratio of each layer of soil or rock are required as input to the analysis. The variation of these parameters with seismic strain amplitude are also required, however such data requires specialist geotechnical testing. Generic data are often employed in the absence of site-specific test data. The geotechnical properties of the 1D deconvolution model should represent the free-field site conditions.

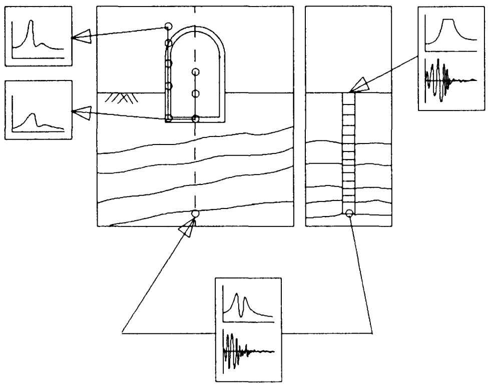

Fig. 4. Soil-structure interaction analysis

33. The base level of the model is typically located at the depth of the deepest available borehole and must be sufficiently deep so as not to be affected by any SSI effects. This base level is regarded as bedrock level, or equivalent. The ground motions are 'deconvolved' through the 1D model to calculate the motions at base level. Computer programs such as SHAKE and FLUSH are commonly used for this purpose.

Soil-structure interaction

34. An SSI analysis is typically required for soft and medium sites, and tends to be a specialist activity.

35. The compliance of the ground can cause rocking of the building on its foundations, giving rise to large displacements towards the top of the building. Such dynamic behaviour is normally the governing condition for the sizing of seismic gaps between adjacent structures, particularly when considering beyond-design-basis events. It is also important in the generation of secondary response spectra for the qualification of plant and equipment.

36. SSI analyses are normally of the following types:

(a) impedance methods
(b) finite element methods

37. The simplest and most common SSI approach is the impedance method using frequency-independent springs and damper elements to represent the ground. The shear modulus information required to calculate the spring and damper constants is based on the strain-reduced values derived from a site response analysis.

38. More complex analyses may also be carried out using 2D and 3D SSI models, which enable layered sites and complex foundation shapes to be represented.

Modelling aspects

39. Owing to the natural variability of ground conditions at a site, sensitivity studies are carried out for a range of geotechnical properties. The shear modulus, which represents the ground stiffness, is usually varied in the range 50% to 200% of the best estimate value (ref. 8). If justified by the quality of the site investigation, this range may be narrowed to 67% to 150% of the best estimate value.

40. Depending on the particular site conditions, the Poisson's ratio for ground below the level of the water table should be chosen carefully. Whilst the presence of pore water does not affect the propagation of shear waves (horizontal ground motion), it can have considerable bearing upon the propagation of compression waves (vertical ground motion). For this reason, separate analyses tend to be necessary for the horizontal and vertical direction SSI analyses.

DESIGN INTERFACES

41. The seismic analysis is normally divided into a number of successive stages. Some stages reflect natural interfaces between different engineering disciplines and others provide for analytical convenience. Design conservatisms are introduced at each interface which, when compounded, lead to over-design resulting in significantly increased capital cost.

42. The geotechnical engineer provides an assessment of the properties of the ground for input to the SSI analysis. His assessment of the shear modulus values must be realistic, rather than pessimistic, to ensure that the natural frequencies and response spectra of the soil-structure system are predicted with confidence. Any shift in the spectral peaks would be reflected in possible over or under-design of the structure.

43. An interface is often introduced, for design convenience, between the structure and the foundation. This enables the foundation to be designed in detail to resist the global seismic forces imposed by the structure, which are often derived from a separate fixed-base analysis of the structure.

44. Owing to the heavy plant content of most nuclear installations, the interface between the plant and civil structure is important. The plant imposes a mass loading onto the structure, which affects its seismic response. Some plant, such as major cranage, is both heavy and flexible. In these cases, it is necessary to model the dynamic flexibility of the plant in addition to its mass within the civil analysis. Secondary response spectra calculated at this interface are nearly always a source of conservatism.

DESIGN OF PRIMARY STRUCTURE

45. The primary structure is normally designed from a structural analysis model, rather than from an SSI model which only requires a coarse representation of the structure. SSI effects are usually embodied in a modified ground response spectrum for input to the structural analysis, as indicated in Fig. 4.

Foundation design

46. Basemats are normally designed using a plate or grillage finite element model on an elastic foundation. Elastic continuum and Winkler spring methods are both used to represent the compliance of the ground. Continuum methods offer a number of advantages, including the modelling of a layered site, but such methods require additional computational effort.

47. An equivalent-static analysis of the foundation is carried out with the seismic forces from the structure applied, together with the other imposed load conditions, such as wind, thermal, live and dead load. It is important to ensure that the seismic moments and shears in the basemat are calculated separately for the three spatial components of earthquake loading. The seismic actions in the basemat are then combined by the SRSS or 100:40:40 combination rule prior to combination with the basemat actions from the other load cases.

48. Redistribution of earth pressures due to localised basemat uplift must be incorporated in the design. Twist moments should be combined with the two orthogonal direct moments using, for example, the Wood-Armer method to derive design moments for top and bottom reinforcement.

49. Piled foundations under seismic loading require special attention to ensure that lateral loading at the pilehead can be accommodated in addition to the conventional axial loading. Unless raking piles are used, the natural frequency of the structure-foundation system in the sidesway mode tends to be very low, leading to large relative deflections between the structure and the surroundings. Pile group effects must also be considered.

50. Conventional earthworks are normally designed using an equivalent static approach (ref. 3). Stability is commonly the governing criterion and liquefaction should also be checked if a build-up of pore water pressure is possible.

Design of walls

51. In-plane and out-of-plane moments and shears are required for the conventional design of walls. In-plane actions are most readily obtained from a finite element beam 'stick' model representation of the wall. A fine mesh of plate finite elements may also be used, but this requires considerable post-processing of the calculated edge stress distribution. Shear

lag effects in concrete walls must also be accommodated within the mathematical model.

52. Out-of-plane actions are normally assessed by hand on the basis of a two-way spanning panel under lateral loading. The lateral loading is obtained by multiplying the panel mass by the spectral acceleration corresponding to the natural frequency of the wall panel and also by a 'static coefficient'. Out-of-plane actions must also include for the effects of inter-storey drift and equipment attached to the wall.

53. This approach is conservative because the loading is based on the secondary response spectrum at the floor level corresponding to the top of the wall, rather than at the middle. The secondary spectrum itself embodies some conservatism. A further conservatism arises from the assumption of a uniform lateral loading distribution across the face of the wall.

Design of floors

54. Floors are designed primarily to resist vertical, out-of-plane seismic loading, in combination with normal floor loads. In addition, diaphragm action must be considered, wherein horizontal seismic tie or strut forces are transferred from one vertical load-bearing member to another.

55. Hand calculation methods are normally used, but detailed plate finite element models are also employed if secondary response spectra are required for the qualification of floor-mounted equipment. The seismic loads applied from any fixed equipment on the floor must also be included in the floor design.

Steel frameworks

56. The natural frequencies of steel-framed buildings tend to lie towards the low-frequency end of the ground response spectrum. Such structures are therefore vulnerable to amplification of the response spectrum in this frequency range by SSI effects.

57. Steel roof structures and in-structure plant support frames are commonly designed with seismic forces derived from secondary response spectra. Such structures generally appear to be over-designed, owing to the level of conservatism embodied in secondary response spectra. Whilst steel structures have an inherent capability for ductile energy absorption, elastic instabilities and end-connection failures must be guarded against by design.

QUALIFICATION OF PLANT AND EQUIPMENT

58. Equipment qualification is carried out by shake-table testing or by analysis, and utilises secondary response spectra calculated by either time-domain or frequency-domain analysis methods.

Generation of secondary response spectra

59. Secondary response spectra are calculated at main floor levels in the building and, particularly, at the mounting positions of essential equipment.

60. Time-domain and frequency-domain methods are both used in practice to derive secondary spectra from the ground response spectra. The two techniques are equivalent, although the time-domain method is the more conventional method. The spectral peaks are normally broadened by 15% in frequency to allow for minor modelling sensitivities.

61. Sensitivity analyses are carried out to determine, for example, the effects of different site conditions and material properties on the secondary response spectra. The secondary spectra from these sensitivity analyses are normally enveloped together to provide conservative design spectra for plant and equipment. Arguably, the spectra corresponding to variations in assumed ground conditions might best be quoted individually to avoid over-design of major plant items. Indeed, care should be exercised in enveloping spectra together to reduce the effect of compounded conservatisms.

62. The three spatial components of the earthquake response are normally combined prior to issue of the secondary response spectra to plant designers. The stresses in the plant are then analysed and the stresses due to the three directional secondary response spectra combined again. This again is a conservative approach which can be improved by retaining the spatial components of the seismic responses in the plant, until the final stress analysis stage.

Qualification by analysis

63. The qualification of equipment by analysis follows in a similar manner to that of the building. Cranes, pipe runs, vessels and cable racking systems are common examples of equipment qualified by analysis.

64. Holding-down bolts and anchors are designed by analysis. Experience of equipment performance during earthquakes shows that equipment exhibits an inherent ruggedness, provided that it is adequately restrained. The effects of long-term loss of preload must also be allowed for in designing fixings anchored in concrete. Contractors experience some considerable difficulty with post-drilled concrete anchors where provision has not been made at design stage for reinforcement-free pockets.

Qualification by shake table testing

65. Shake-table testing is normally required for electrical equipment to demonstrate that a required functional performance has been met.

66. Full-scale testing is performed by subjecting equipment to base motions which conservatively simulate the specified secondary response spectrum motion. This normally takes the form of shake-table testing to a specified 'required response spectrum' (RRS). A 'test response spectrum' (TRS) is developed from the measured shake-table motion and this is required to envelope the RRS.

67. The TRS normally envelopes the RRS by a substantial margin, as illustrated in Fig. 5.

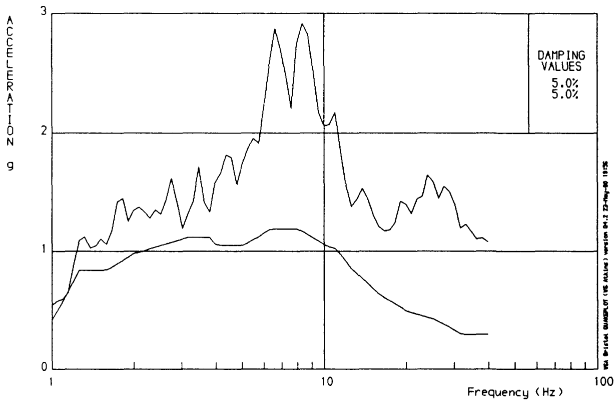

Fig. 5. Typical TRS and RRS from shake table testing

RE-ASSESSMENT OF EXISTING STRUCTURES

68. The seismic re-assessment of existing structures centres on the definition of acceptable performance requirements. These are based on the function of the structure in relation to its contribution to plant safety and may also have regard to the desired future life. The starting point for a typical approach is described in refs. 8 and 11.

69. Typically, structures not designed to contemporary SSE levels will not have sufficient inherent design margins to withstand normal loads in combination with the SSE, where this is assessed by the conventional design methods described above.

70. In such cases, the conservatisms in the conventional analysis and design assessment process may be minimised by reducing the number of interfaces in the analysis, or by the use of more complex analysis. For example, non-linear time-domain analysis may be used to quantify the amplitudes of

seismic movements and the nature of post-elastic behaviour. A safety case is then put forward on this basis.

71. The offshore structure shown in Fig. 6 was re-assessed for its resistance to seismic loading. The conventional response spectrum analysis method, using a linear analysis, indicated that the structure would fail by pull-out of the raking piles. Non-linear time-domain analysis indicated that the structure would, however, retain its lateral stability, because pull-out failure of the raking piles would be a transient phenomenon offset by additional shear mobilised through the deck into the vertical piles.

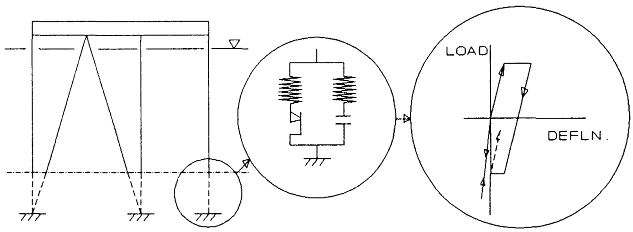

Fig. 6. Seismic re-assessment of an offshore structure

CONCLUSIONS

72. The current seismic analysis and design practice for UK nuclear installations has been summarised.

73. It is important to define performance criteria for structures and equipment early in the project programme. The design should then be targetted on these performance criteria. Seismic design must be documented in a safety case, which is most successfully prepared at an early stage.

74. The current seismic design practice for UK nuclear installations is conservative. Conservatisms arise in the specification of the ground motion, by division of the seismic analysis into distinct stages and at interfaces between different engineering disciplines.

75. These conservatisms offer the opportunity to re-assess the seismic capability of existing structures by the use of a rationalised approach utilising more realistic, but often more complex, analysis.

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9. Alternative approaches to the design criteria for earthquake forces applied to structural sub-systems

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SYNOPSIS

Present practice in the UK nuclear industry can result in gross overestimates of the seismic design forces in many secondary or tertiary structures. Outlines of three alternative design approaches are presented, two of which are based on the use of response spectra and the third is based on solution in the frequency domain.

CURRENT NUCLEAR INDUSTRY PRACTICE FOR UNCOUPLED SECONDARY SYSTEM DESIGN

1. For a primary structure, the response to ground motion is normally calculated by direct analysis of the structure on its own. Where a secondary structure is to be added time history analysis is used to produce a 'floor response spectrum' (FRS) at the point of attachment, which describes the behaviour of the primary structure alone. This FRS is then used as a basis in producing the design FRS (DFRS) for the secondary structure.

2. To allow for uncertainty in estimating modal frequencies, and other modeling uncertainties such as the soil properties, the peaks of the DFRS are widened and some enveloping introduced (ref.1). Further enveloping of the DFRS for a secondary system may be introduced to cover the effects of multiple supports. Where the secondary system is an item of plant which is designed for installation in more than one location, further enveloping again may be introduced into the DFRS to cover multiple locations and even multiple sites.

3. This practice introduces substantial conservatism into the design. Chen (ref.2) gives a comprehensive summary of current practice in the analysis of secondary systems and identifies a number of inherent problems in the uncoupled method:-

(a) Interaction between the primary and secondary system is not accounted for. This will occur where the secondary structure is large in relation to the primary structure, and/or the natural frequencies of the two are close. Hence it is only acceptably accurate within certain mass, modal frequency ratios and damping limits outside which it cannot be used.
(b) Multiple supports are difficult to deal with. Cross correlations between support excitations are often inaccurately considered or neglected. Response is separated into 'pseudo static' and 'dynamic' parts leading to difficulties in modal combination.
(c) Closely spaced modal frequencies involve complicated cross correlation calculations.
(d) If tertiary structures are introduced they can only be dealt with by introducing further levels of conservatism.
(e) Each level of enveloping introduced into the DFRS incorporates unrealistic levels of conservatism into the design.
(f) DFRS are based on time history analysis and accordingly are expensive to produce.
(g) The final degree of conservatism built into each system is never known and an accurate computational model of the final system is never produced.
(h) Non-classical damping which occurs where the primary and secondary structures have different damping levels cannot be accurately dealt with. Chen (ref.2) gives an example where the neglect of off diagonal coupling terms in the damping matrix can lead to errors in the secondary response spectrum of over 100%.

4. The advantages of this approach are:-

(a) Design of each primary or secondary system is compartmented so that it can be completed independently (although this is only achieved by ignoring dynamic interaction).
(b) Design of singly supported secondary systems is simple.
(c) The use of response spectra is common practice in the industry so that engineers are accustomed to its use.

5. An example is given in (ref.3) of the use of uncoupled reponse spectrum methods for the design of a pipework system for ground shaking applied to the primary structure. Various uncoupled response spectrum analyses are compared with a time history analysis. The over-prediction of moments in the pipework from using response spectrum analysis runs to over 1000% while in a few cases there is actually an under prediction.

REQUIREMENTS FOR A SECONDARY SYSTEM DESIGN METHOD

6. Igusa and Der Kiureghian (ref.4) list six principal requirements and comprehensively discuss previous work which they conclude does not meet them. The (IDK) requirements are:-

(a) Multi degree of freedom (MDOF) primary and secondary systems can be modeled.
(b) Multiple points of attachment can be modeled.
(c) Multiple tuning of modal frequencies is possible.
(d) Dynamic interaction between primary and secondary systems i.e. coupling is accounted for.
(e) Cross correlation between modes can be included.
(f) Non classical damping - i.e. different damping in the primary and secondary system - can also be modeled.

7. The three coupled design methods which are examined in the following sections all meet the IDK requirements (except that the first two only deal with coupling insofar as it affects the secondary structure) with varying degrees of accuracy and ease of implementation.

THE GUPTA-JAW IN-STRUCTURE RESPONSE SPECTRUM (IRS) METHOD

8. This method is described in Gupta and Jaw (ref.5) and Gupta (ref.6). Seismic input is defined by the response spectrum of ground motion and this is used directly without conversion to a power spectrum or time history. Separate eigensystems for the primary and secondary structures are computed in the normal way and an iterative process is used to generate the complex eigenvalues and eigenvectors for the combined primary and secondary structure. This can then be used to derive the response of the secondary system and takes account of coupling between the two (insofar as it affects the secondary system). Reference (6) compares results from the conventional (uncoupled) response spectrum method and results from the coupled approach with 'exact' results from time history analysis using nine differing structures and these are summarised in Table 1.

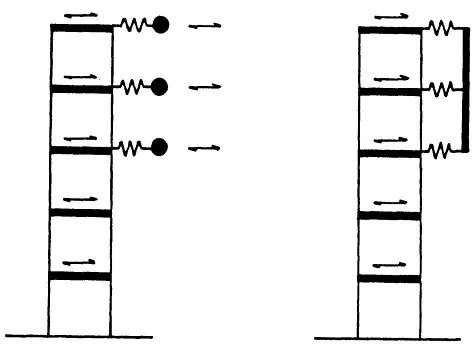
TABLE 1
Percentage errors for coupled and uncoupled analysis

	COUPLED		UNCOUPLED	
	MEAN ERROR	STD. DEV	MEAN ERROR	STD. DEV.
DISPLACEMENT	3.0	6.9	58.0	44.1
I/S SHEAR	11.2	9.9	116.1	46.9

9. This method is appropriate where the secondary system can be accurately modeled but this is often not the case in practice where a design specification for the secondary system is required. For this circumstance Gupta and Jaw have extended the coupled analysis to develop response spectra at the interconnecting supports for the secondary structure which they refer to as 'in-structure response spectra' (IRS). These spectra are plotted for varying secondary structure mass and damping.

10. Because these IRS are calculated in the normal way for response spectra, that is representing the response of a single degree of freedom oscillator at each point of connection and appropriate degree of freedom, they do not take account of any static constraint between the main and secondary structure. This is illustrated in Figure 1. The effect of static constraint on the response of the secondary system can be significant. Gupta and Jaw propose a method using correlation coefficients which define the correlation between the support displacements at the various connecting degrees of freedom. Theoretically the computation of these is straightforward but involves transmittal of large amounts of data to the user of the IRS. To overcome this a simpler approximate method is developed.

11. The Gupta-Jaw IRS method presents an efficient computational approach which, according to the comparative studies in reference (6) is of an acceptable accuracy, comparing favourably with conventional uncoupled methods and the IDK method discussed in the following section. However the computational procedures are complex and in practice are likely to be used as a "black box" procedure by design engineers. Limitations to the method are firstly that approximations are involved in the method for calculating the combined eigensystem and the correlation coefficients. From the studies reported in reference (6) the approximations appear to have little effect on the accuracy of the eigensytem but the approximate correlation coefficients which are frequency dependant appear to be less reliable especially where there is tuning of modal frequencies between the primary and secondary structure. The second limitation is that the effect of coupling is only considered in one direction, that is its influence on the reponse of the secondary structure. The effect of the secondary on the primary structure is not considered.



(a) RESPONSE SPECTRUM MODEL

(b) MODEL FOR MULTIPLY CONNECTED SUB-STRUCTURE

FIGURE 1 COMPARATIVE MODELS FOR FLOOR RESPONSE SPECTRUM COMPUTATION AND FOR MULTIPLY SUPPORTED SUB-SYSTEM

THE IGUSA-DER KIUREGHIAN (IDK) RESPONSE SPECTRUM METHOD

12. Igusa and Der Kiureghian (ref.7) develop a combined system of analysis for multiply supported secondary systems. Ground motion inputs can be in the form of response spectra, time history or power spectra. The complex eigensystem of the combined structure, primary and secondary, is derived from the individual eigensystems using a perturbation method. Like the Gupta-Jaw method an iterative approach is used and some second order approximations are made. The combined eigensystem can then be used to derive the response of the secondary system to the specified ground motion input.

13. Reference (7) gives an example where the method is used to calculate the seismic response of a 4 DOF secondary system attached to a 3 DOF primary system. Using secondary to primary mass ratios of from 0.001 to 0.05 errors in acceleration response of up to 14.3% and up to 6.9% on bending moment occur, using time history analysis of the combined system as datum.

14. Asfura and Der Kiureghian (ref.8) extend this method to to produce IRS solutions comparable with those in the Gupta-Jaw method. This method uses random vibration theory to generate 'cross oscillator, cross floor spectra, (CCFS) to account for pseudo static effects in the secondary system. Interestingly in connection with the use of random vibration theory reference (8) states, "A smooth ground response spectrum, such as that used in the design of nuclear power plants, represents the peak response of an oscillator to an ensemble of ground motions. The proper framework for analysis using such spectra, therefore, lies in the theory of random vibrations."

FREQUENCY DOMAIN RANDOM VIBRATION ANALYSIS OF MULTIPLY SUPPORTED STRUCTURAL SUB-SYSTEMS

Background

15. The theory of random vibration dynamic analysis of linear structures is well established, for example in references (9,10,11). The theory is generally based on the use of a stationary input so that a question arises on its applicability to seismic analysis. Studies of analysis treating the strong motion segment of ground motion as stationary (refs.12,13) have shown that a response can be calculated to an acceptable level of accuracy. This process uses numerical adjustments to the response which compensate for the limited duration of the strong motion duration.

16. An alternative methodology which can be used where the input is strongly non stationary is to use evolutionary power spectral input (refs.14,15) but this is not dealt with here on the grounds that adequate accuracy can be achieved with the assumptions of stationarity and introducing a correction factor for duration.

Ground Motion Input

17. Random vibration methods use a power spectrum of ground motion as input. This is a frequency based representation of the ground motion power. Vanmarcke (ref.16) gives an extensive discussion of the value of a power spectrum approach compared with the response spectrum alternative; some of the main points made in this discussion are, in brief:-

(a) Any phenomenon that is sensitive to motion duration such as inelastic action or low cycle fatigue is poorly predicted by response spectrum methods.
(b) An unknown degree of conservatism enters into the analysis of MDOF linear systems when the modal ordinates of design response spectra - unlikely to occur simultaneously- are combined.
(c) The use of peak ground acceleration which is widely used to scale response spectra is a poor indicator of ground motion severity.
(d) Response spectrum methods are unsuited to dealing with the spatial variability of ground motion.
(e) Stochastic representation - power spectra - are better able to deal with the foregoing problems (a) to (d). Additionally they offer improved accuracy in dealing with the influence of local geology and in relating the spectral properties of ground motion to source parameters and source to site distance.
(f) The use of random vibration methods offers the most efficient means of deriving the response in probabilistic terms - mean and distribution - which is of value in low cycle fatigue and reliability studies.

18. The power spectrum or power spectral density (PSD) of an acceleration record a(t) derives from the Fourier transform

$$A(\omega) = \left| \int_0^{\infty} a(t)\, e^{-i\omega t}\, dt \right| \qquad (1)$$

$A(\omega)$ = Fourier amplitude spectral value

ω = angular frequency (rads/sec)

t_o = duration (seconds) of the accelerogram

The PSD of the accelerogram is defined by:

$$G(\omega) = \frac{A^2(\omega)}{\Delta\omega} \qquad (2)$$

19. Equation (2) represents the one sided (positive frequencies only) PSD function which indicates how the "power" (energy per unit time) is distributed over the frequency range. Reference (16) shows the close relationship between the integral of the PSD function over the frequency range and the Arias Intensity which is a more efficient indicator of the capacity of a ground motion to cause damage than peak acceleration.

20. A commonly used (smooth) design PSD is the Kanai-Taijimi (ref.17) spectrum which is defined:

$$G(\omega) = \frac{[\,1 + 4\xi_g^2(\omega/\omega_g)^2\,]\; G_0}{[1-[\omega/\omega_g)^2]^2 + 4\xi_g^2(\omega/\omega_g)^2} \qquad (3)$$

where ξ_g = viscous damping (0.6 for firm ground)

ω_g = natural ground frequency (12.7 for firm ground)

G_0 = scaling factor

Equation (3) represents the response of a single degree of freedom system to a 'white noise' excitation. However the definition of a design PSD may equally well be derived non-analytically from analysis of a number of appropriate ground motion records. The scaling parameter G_0 may be derived from:

$$G_0 = \frac{0.141\, a_{max}^2\, \xi_g}{\omega_g (1 + 4\xi_g^2)^{0.5}} \qquad (4)$$

where a_{max} = peak value of time based record

21. Ground motions are commonly defined in practice by response spectra so that it is useful to note that methods exist for deriving compatible PSDs from specific response spectra (refs.13,18,19) and vice versa.

RECEPTANCE ANALYSIS OF SECONDARY SYSTEMS

Receptance

22. Solution in the frequency domain can be based on solving nodal equations of force, displacement, velocity or acceleration. Usually the most convenient parameters are force and displacement and these are used here in the form of receptance which is defined:

$R^A{}_{ij}$ = displacement at i due to a force at j for structure A

23. In the following, receptance is used in the dynamic sense, associated with a frequency ω . Calculations for receptance are straightforward once the eigensystem for the structure is known, only as many modes as necessary being calculated. The RMS response, σ_x, may be calculated from:-

$$\sigma_x^2 = \int_{\infty}^{\infty} \left| G_x(\omega) \right| d\omega = 2\int_0^{\infty} G(\omega) \left| H_{xo}(i\omega) \right|^2 d\omega \qquad (5)$$

where $H_{xo}(i\omega)$ = Complex frequency response function

$G(\omega)$ = power spectral density of forcing function

$G_x(\omega)$ = Output power spectral density function

Peak response is approximately 3 σ_x but may be estimated more accurately from σ_x and the moments of $G_x(\omega)$ (ref.20). In addition the distribution of peak response can also be obtained.

24. Ground motion is defined as stationary over a period of time τ. For low damping and low frequency structures equation (1) needs to be modified by a duration correction factor $B(\xi, \omega, \tau)$ so that:

$$\sigma_x^2 = 2\int_0^{\infty} \left| G_x(\omega) \right| B(\xi,\omega,\tau)\, d\omega \qquad (6)$$

where, for low damping

$$B(\xi,\omega,\tau) = 1+e^{-2\xi\omega_o t}\Big[\cos^2\omega_o t + \frac{\omega^2}{\omega_o^2}\sin^2\omega_o t \qquad (7)$$

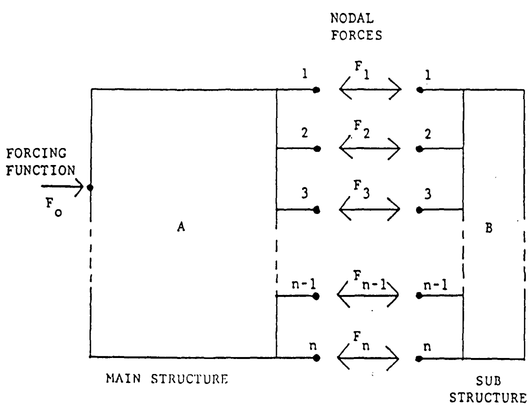
$$-e^{\xi\omega_o t}\Big(2\cos\omega_o t\,\cos\omega t + 2\frac{\omega}{\omega_o}\sin\omega_o t\,\sin\omega t\Big)\Big]$$

<u>Receptance Solution to linked structures</u>

25. Figure 2 shows the basic model of a primary system and attached secondary structure. The general solution for this model in the frequency domain can be shown to be:

$$\left[[R^A] - [R^B_{jj}]\right]\{F\} = -F_o\{R^A_{jo}\} \qquad (8)$$

where the suffix 'o' refers to the location of the forcing function.



DISPLACEMENTS AT NODE $i = x_i$

FIGURE 2 STRUCTURE AND SUB-STRUCTURE FOR RECEPTANCE ANALYSIS

Alternatively, solving in x

$$[[R^A]\ [R^B_{jj}]^{-1} - [I]]\ \{x\} = -F_o\{R^A_{jo}\} \qquad (9)$$

26. For further structures being added to A, B, or A and B, receptance values R^{A+B} for the new nodes can be calculated using (4) or (5) and the process repeated. Once a structure is defined the matrix to be stored is N x N x NF where N is the number of attachment degrees of freedom and NF the number of values of required. NF may be reduced for most practical cases by using the integration expressions given by Vanmarcke (ref.21).

<u>Discussion</u>

27. Comparisons of the three methods presented are illustrated in Table 2. All three approaches comply with the six IDK requirements (paragraph 6) except that in the Gupta and IDK methods interaction is only dealt with in terms of its effect on the sub-system and not on the primary. Frequency domain/receptance analysis (FDRA) appears to be the only one which can deal with multiple hierarchies of sub-systems.

28. At the user interface for singly supported sub-systems the Gupta and IDK methods provide the designer/supplier or 'user' with straighforward information in familiar terms of response spectra. The same option is available for the FDRA method. For a user prepared to use FDRA methods data would be supplied in the form of a frequency based power spectrum of excitation and receptance.

29. At the user interface for multiply supported sub-systems information supplied for the Gupta and IDK methods would be response spectra plus cross correlation data. For the FDRA method the data would comprise power spectra for each degree of freedom plus receptances and cross receptances for all degrees of freedom.

30. In terms of computing resources the comparison lies between the iterative procedures for producing the combined eigensystem which are embodied in the Gupta and IDK systems and the solution of the interaction equations in the FDRA method for each frequency required.

31. Advantages of the FDRA method over the other two are:
(a) A comprehensive model of the whole primary and all hierarchies of sub-systems can be built up.
(b) Sensitivity checks for each sub-system will establish whether its effect on the primary is trivial or not and in the event that it is trivial it can be dropped from the model.
(c) The over conservative enveloping used with response spectra can be dispensed with. Because the FDRA approach uses probabilistic methods uncertainy can be dealt with in a probabilistic manner instead of adding layers of conservatism.
(d) Defining ground motion by a PSD has advantages (described in paragraph (17)) over the reponse spectrum definition.
(e) Response can be described in a probabilistic manner - mean and standard deviation which is advantageous for reliability and risk assessment studies.

				User information	Inter-action	Earthqua input
Gupta/ Jaw	Primary system	Eigen-system	Combined complex eigensystem by iteration		No	Respons spectru
	Sub-system	Eigen-system		IRS correlation coefficients	Yes	
IDK	Primary system	Eigen-system	Combined complex eigensystem by iteration		No	Respons spectru PSD, ti history
	Sub-system	Eigen-system		IRS correlation coefficients	Yes	
FDRA	Primary system	Eigen-system	Interface node receptances.	Feedback from user to prime	Yes	PSD
	Sub-system	Eigen-system	Interface node receptances.	PSD (compat.IRS) receptances	Yes	

TABLE 2 comparison of three sub-system analysis methods

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10. The influence of shaking table characteristics on seismic qualification testing methodology

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SYNOPSIS. In seismic qualification testing using servo-hydraulic shaking tables, several factors limit the test response spectra which can be achieved. The most important of these are the performance characteristics of the table itself, particularly at low frequency.

INTRODUCTION

1. Electromechanical equipment in nuclear related facilities often has to be shown to be capable of functioning during and after earthquake shaking. Such seismic qualification can be done using analytical modelling, but in many cases involving complex equipment, physical testing is the only reliable qualification method. Testing generally involves shaking the equipment on a shaking table which is controlled to produce table motions representative of those which the equipment is likely to experience at its installed location.

2. The required shaking is usually specified in the form of response spectra. These are derived from numerical analysis of the response of the building structure to earthquake ground motions, taking into account interactions between primary, secondary and higher order structural systems within the building including, where appropriate, the equipment itself. The required response spectra represent an infinite set of acceleration time histories, each with different time domain characteristics but each having the same response spectrum. It is the job of the test house to produce shaking table acceleration time histories which envelop the required response spectra. This is not a simple task, being influenced by many parameters relating to the performance characteristics of the shaking table system and equipment under test. These factors lead to practical limits on the range of testing which can be achieved. A sound understanding of these limitations can help the design engineer to specify sensible required response spectra, particularly when dealing with secondary, tertiary and higher order structural interaction effects which can lead to over conservative response predictions.

3. The aim of this paper is to review some of the factors affecting achievable test spectra. The nature of response spectra is discussed following which the basic elements of a typical servo-hydraulic shaking table system are described. The important system characteristics are then identified and their influences on system performance discussed.

THE NATURE OF SEISMIC QUALIFICATION TESTING

4. In general, the aim of a seismic qualification test on an item of equipment is to prove that the equipment will function satisfactorily during and after experiencing an earthquake. Test procedures are detailed in various national standards (e.g. IEEE 344 (ref. 1), IEEE 323 (ref. 2)) and client specific standards.

5. In the UK, seismic qualification generally involves two main stages; exploratory testing and seismic testing.

Exploratory Testing

6. An exploratory test is one in which the test equipment is shaken either by low amplitude, frequency band limited, random accelerations or by low amplitude, sinusoidal motions sweeping in frequency over a specified bandwidth at a specified rate. The objective of these exploratory tests is to determine the equipment's natural frequencies and associated damping characteristics. These tests are of low amplitude since it is generally undesirable to risk damage to the test equipment prior to the second (seismic) stage of testing. Consequently, the damping measured in exploratory tests may be lower than that present in the higher level seismic tests when significant non-linear behaviour may occur.

7. In a typical exploratory test, input accelerations will be measured on the platform of the shaking table. Response accelerations, in the same direction as the input accelerations, will be measured on the test equipment. For random excitation, the table motions are adjusted typically to an rms level of 0.05g to 0.2g and the equipment shaken in a single degree of freedom at a time for around three to five minutes, data being recorded continuously over this period. The data are then processed using standard spectral analysis techniques (see for example ref. 3) to yield transmissibility plots which show how the amplification of acceleration at various locations on the test equipment varies with frequency.

8. Sine sweep testing involves a similar procedure except that the input signal is controlled to give a specified peak input acceleration at all frequencies, typically 0.05g to 0.2g. The sweep usually ranges from 1Hz to 40Hz at a rate of two octaves per minute. Sine sweep testing is a little more

difficult to carry out than random testing because of the need to monitor and adjust continuously the level of the driving signal to the shaking table to ensure the correct peak input accelerations are achieved, compensating as necessary for shaking table system resonances. Random excitation also needs to compensate for these resonances, but in general this can be done by shaping the frequency content of the input signal prior to the test, taking into account the shaking table's frequency response function.

9. Random excitation testing is usually quicker than sine sweep testing, since all frequencies are active at all times. Relatively cheap, digital spectrum analysers are now available which can generate the appropriate, compensated, random driving signal, and acquire, process and plot the response data in real time.

<u>Seismic testing</u>

10. Seismic testing forms the main stage of the seismic qualification procedure. It involves subjecting the test equipment to earthquake-like base motions representative of those it will experience at its particular installed location, or locations. These motions are usually specified in the form of a *Required Response Spectrum (RRS)*. This defines the maximum dynamic response to the input motion of any simple, damped, single degree of freedom structure having a natural frequency within the specified frequency range of the spectrum. Fig. 1 illustrates a typical velocity response spectrum. From this spectrum, an SDOF structure with natural frequency of 5Hz and damping of 5% critical would have a peak velocity response of 0.22 ms^{-1}. The definition and important characteristics of response spectra are discussed later. For UK conditions, the duration of shaking in a seismic test is usually specified at between ten and twelve seconds.

11. Seismic testing usually involves shaking the test equipment several times (typically five) with a multicomponent earthquake (i.e. one having uncorrelated motion in two orthogonal horizontal components and in the vertical component) which represents the maximum level of shaking at which the equipment should continue to operate satisfactorily without any action being taken by the operator. This is often termed the *Operational Shutdown Earthquake (OSE)* and is usually around 20-25% of the Required Response Spectrum. Functional tests of the equipment may be carried out during and after each shake as directed by the client.

12. The OSE tests are then followed by a larger shake at 100% of the RRS (with any additional margins as specified by the client), usually termed the *Safe Shutdown Earthquake (SSE)*. This is the maximum level of shaking following which the equipment should be capable of being shut-down safely.

In some cases, the SSE may be followed by an overtest of up to 160% of the RRS. Functional tests may be carried out as before. Normally, the shaking table accelerations in each component of shaking and appropriate response accelerations on the test equipment are recorded. Those from the table are processed to yield the actual response spectrum of the table, known as the *Test Response Spectrum (TRS)*, which must envelop the RRS. Similar processing may also be done for the equipment response data.

13. It is the responsibility of the test house to generate shaking table acceleration time histories which yield an acceptable TRS. This is a complex procedure which is greatly influenced by the dynamic characteristics of the shaking table itself and by the dynamic interaction between the table and the test equipment. The latter interaction is different for each item of equipment and so it is difficult to generalise the compensatory action which must be taken.

14. Obviously, the shape and frequency content of the RRS are the most important factors influencing the form of the table accelerations. Many items of equipment are used in multiple locations, each location having different RRS's. In such cases, it is common practice to construct a composite RRS which envelops the individual RRS's. This composite RRS is then used to define the table accelerations, thus reducing several potential tests to a single test. It should be recognised that this procedure can lead to high acceleration table motions which may be grossly conservative (ref. 4).

RESPONSE SPECTRA

15. As indicated above, the required response spectrum (RRS) is the most significant factor controlling a seismic qualification test. It is therefore important to understand the key characteristics of response spectra if rational testing is to be achieved. This section presents a brief overview of the derivation, characteristics and use of response spectra.

Definition of response spectra

16. Fig. 2 shows a simple, single degree of freedom structure consisting of a rigid transom beam of mass, M, and two massless columns, each of stiffness $K/2$. A viscous dashpot connects the transom to the ground, providing a damping characteristic, C. The sole degree of freedom of this oscillator is the horizontal displacement, x, of the transom relative to the ground. The oscillator has an undamped natural frequency, w_n (radians/second), and damping factor, z, expressed as a fraction of critical damping. If this oscillator is subjected to an earthquake ground acceleration, it will vibrate at its damped natural frequency. Fig. 3 shows how an oscillator (with a natural frequency of 5Hz and damping of 5% critical) responds to a

typical earthquake. The oscillator responds vigorously at certain times during the earthquake when the frequency content of the ground motions is close to the natural frequency of the oscillator. The oscillator's displacement response, is calculated by solving numerically the Duhamel Integral (ref. 5).

17. A displacement response spectrum for a given earthquake ground motion is constructed by calculating the absolute maximum relative displacement responses of a set of simple oscillators having different natural frequencies but the same damping ratio. These maximum responses are termed Spectral Displacements, S_d, and are plotted against frequency. A range of curves is usually produced for different damping ratios. Response spectra were originally introduced in the 1940's (ref. 5) as a simple means of calculating the seismic response of simple structures. The maximum spring force, F_s, in the columns of the oscillator is given simply by the product of the spring stiffness and the spectral displacement for the oscillator.

18. A pseudo-relative velocity response spectrum, S_v, can be defined as

$$S_v = wS_d \qquad \text{... (1)}$$

and a pseudo-acceleration response spectrum, S_a, as

$$S_a = wS_v = w^2S_d \qquad \text{... (2)}$$

19. The pseudo term is used since S_v and S_a calculated in this manner are not the same as the corresponding exact values calculated by solving the appropriate form of the Duhamel Integral. However, in the frequency range from 1Hz to 30Hz, the pseudo and exact spectral values are almost identical, and therefore the pseudo values are acceptable for engineering purposes.

Some important characteristics of response spectra

20. Fig. 4 shows how the example simple oscillator responds to a 1g amplitude sinusoidal input motion. The displacement of the oscillator gradually increases in an exponential manner, controlled by the level of damping, until steady state response is reached. If the frequency of the input motion coincides with the natural frequency of the oscillator, the steady state response will be the resonant response. It is easily shown that the number of cycles, N, to reach 99% of the steady state response is given by

$$N = -\ (\log_e 0.01)/(2\pi z) \qquad \text{... (3)}$$

where z is the fraction of critical damping. Thus an oscillator with 5% damping takes about 14 cycles to reach steady state (Fig. 4). This means that low frequency oscillators subjected to 10 to 12 seconds of random motion are less likely to reach resonance than high frequency oscillators.

21. At this point, it is worth examining some other important features of real response spectra. Studies of response spectra of actual earthquakes (ref. 5) have indicated general bounds of spectra relative to the peak ground displacement, velocity and acceleration. Referring to Fig. 1, at low frequencies the spectrum tends to the peak ground displacement. This is because the spring stiffness of the oscillator is very low and effectively isolates the transom mass from the ground motions. Thus, the absolute position of the mass is constant and therefore the peak relative displacement of the mass to the ground is simply the peak ground displacement. At high frequencies, the acceleration response tends to the peak ground acceleration since now the spring stiffness is very large and the oscillator effectively behaves as a rigid block on the ground. This acceleration limit is often termed the Zero Period Acceleration (ZPA). Over the intermediate frequency range, the dynamic response of the oscillator is significant and the statistical studies have indicated that for typical, non-stationary, random earthquake motions, spectral accelerations for a 5% damped oscillator are of the order of 2.5 to 3.0 times greater than the peak ground acceleration. This compares with a factor of 10.0 for the steady state, resonant conditions for a pure, sinusoidal input (Fig. 4).

22. Now consider a RRS which is reasonably broadband in nature, but has a high acceleration amplitude at low frequencies (Fig. 6). The broad frequency band will lead to spectrum compatible acceleration time histories which are reasonably random in nature and similar to real earthquake ground motions. But, in view of the low amplification factors discussed above, the time history must contain very high amplitude, low frequency components. For example, if a 5% damping RRS demands say 6.0g at 1Hz, which is not uncommon, the time history typically needs to contain about 2.0 to 3.0g at this frequency if the 6.0g spectral response is to be attained over 10 to 12 second period. The corresponding displacement is approximately +/- 200mm. This places great demands on the shaking table hydraulic system since a considerable amount of oil (possibly 400 litres per minute for a single axis of shaking) needs to be delivered to achieve such displacements. As has already been noted, at higher frequencies (e.g. above 30Hz), the spectral accelerations will tend to the zero period acceleration, which in this case is the peak table acceleration of 3.0g. Thus, there is limit to the ratio of the low frequency spectral acceleration to the ZPA.

23. Required response spectra having large spectral acceleration demands at high frequency are easier to achieve. In this case, the oscillator has ample time to approach resonance and the displacement demand, and hence oil flow, are small.

24. From the above discussion it will be seen that care must be taken to specify realistic required response spectra, particularly at low frequencies.

SERVO HYDRAULIC SHAKING TABLES

25. Most shaking tables used for seismic qualification testing are driven by servo hydraulic actuator systems, since hydraulic actuators offer the best solution to the combined requirements of large force, velocity and stroke. This section describes the important features of such systems.

26. Fig. 5 shows a typical simplified servo hydraulic actuator system driving a simple, single degree of freedom shaking table, or platform.

Actuator arrangement

27. The actuator consists of a piston rod contained within a cylinder. Each end of the rod is supported by a linear bearing. The piston rod has an annulus at its middle which separates the cylinder volume into two chambers. By applying a differential pressure across the annulus (and allowing oil to flow into and out of the two chambers), the piston rod can be made to move backwards and forwards. The oil flow is controlled by an electrical servo valve which connects the high pressure supply line (typically at 180-210 bar) and the low pressure return line of an hydraulic pump alternately to each chamber, causing the piston to move to and fro. Clearly, the force generated by the actuator is equal to the product of differential pressure across the annulus and the area of the annulus. A 50kN actuator will have an annular area of approximately 35 cm^2 and minimum working pressure of around 140 bar.

Hydraulic oil supply

28. Oil is supplied by a high pressure pump whose delivery is usually rated in litres per minute. A 400 litre per minute pump may cost in the region of £100,000 and therefore additional, short term flow capacity is often provided in the form of cheaper, nitrogen charged accumulators. These consist of a steel cylinder containing a rubber bag connected to the high pressure supply line. On the other side of the bag is pressurised nitrogen. When the hydraulic pump is off, the nitrogen forces all oil out of the accumulator and the nitrogen pressure is at or slightly below the minimum actuator working pressure. When the pump is switched on, the hydraulic pressure in the supply line increases, forcing oil

into the accumulator and compressing the nitrogen. When the supply pressure reaches its maximum working pressure, the nitrogen is at this pressure too. Should the pump alone not be able to supply enough oil flow to the actuators, the supply line pressure will drop, leading to a differential pressure across the oil in the accumulator which causes the nitrogen to force oil from the accumulator into the supply line. This tends to maintain the flow in the supply line until the accumulator discharges. As this happens, the nitrogen volume increases and its pressure decreases, leading to a gradual pressure drop in the supply line. Should the oil flow demand drop below the pump capacity again, the accumulator will start to recharge. Accumulator sets offer an economical method for increasing oil flow over a short period.

29. Oil flow capacity should be sized to ensure that the oil pressure will not fall below the minimum working pressure of the actuator (typically 140 bar), otherwise the actuator will not be able to generate full rated force for the duration of the test. Note, however, that if the supply line pressure can be maintained at the maximum working pressure (typically 180 bar) then the force generated by the actuator can be 30% greater than its rated force.

System compliances

30. An important factor influencing the dynamic performance of a servo-hydraulic system is compliance, or flexibility, in the mechanical and hydraulic parts. An hydraulic actuator contains a spring in the form of the compressible oil column within it. In series with this are other springs such as the flexible couplings between the actuator and the platform and reaction block, and the axial spring stiffness of the piston rod.

31. Considering a single actuator driving a mass, M, it can be shown that the hydraulic natural frequency, f_H, is given by

$$f_H = A/\pi \sqrt{(N/MV)} \qquad \ldots (4)$$

where A = piston area
N = bulk modulus of oil
V = enclosed volume of the cylinder

32. To minimise the effects of actuator resonance, it is necessary to have a high actuator frequency, which implies a large piston area and short stroke (and hence small cylinder volume). For seismic testing purposes, actuator resonances tend to lie within the test frequency range, due to the large stroke required.

Test specimen resonance

33. A further complication arises if the test specimen has natural frequencies in the test range. In this case, the resonating specimen can tend to drive the shaking table, reducing system performance. This is similar to primary-secondary structure interaction and is more significant the greater the mass of the test specimen. Fig. 6 shows how test specimen resonances can affect the frequency response of the shaking table, causing a 'peak and notch' effect in the amplitude response around the natural frequency of the test specimen. The solution to this problem is to introduce damping either by hydraulic means (e.g. a bleed orifice between the inlet and outlet parts of the actuator) or by an electronic compensation circuit. A significant problem here is that each test specimen has different, and sometimes non-linear, resonance characteristics and therefore it is not a simple task to develop a general control system capable of compensating for all test specimen / shaking table interactions. Most tests require some adjustment of the shaking table system to achieve optimum performance.

Performance envelopes

34. The shaking table performance is limited by actuator stroke (i.e. displacement) at low frequencies, actuator velocity at intermediate frequencies, and actuator force and payload at higher frequencies. The velocity limit is controlled primarily by the flow capacity of the servo-valves and also by the pump and accumulator flow capacities. At higher frequencies the table performance is acceleration limited, the peak acceleration being equal to the maximum actuator force divided by the payload mass.

Optimum table motions

35. To achieve severe required response spectra, the shaking table control motions must be consistent with the response spectrum and servo-hydraulic performance characteristics discussed previously. To extract the maximum performance from the table it is essential to ensure that the system hydraulic pressure remains as high as possible and does not fall below the minimum working pressure at which the actuator force rating is defined. This implies that the actuator velocities should be controlled to ensure that there is a uniform oil flow demand over the duration of the test. For an accumulator supplemented hydraulic system, the aim should be to just exhaust the accumulator at the very end of the shake. A uniform oil flow is achieved by minimising the ratio of the peak velocity to the root mean square (rms) velocity in each direction of shaking. This results in a velocity history similar to that of Fig. 7. The frequency content of this history is fairly constant with time, with the effect that low frequency oscillators have time to build up to the necessary response spectrum levels.

CONCLUSIONS

36. When generating response spectrum compatible time histories on a shaking table, it is important to appreciate certain key features of response spectra. Firstly, the response spectrum of an acceleration time history is the locus of the maximum absolute responses, plotted against frequency, of a set of simple oscillators having different natural frequencies but the same damping value. If an oscillator is to approach or reach resonance, and hence the response spectrum attain a maximum value, it must be exposed to base motions at its natural frequency for sufficient time. In the case of a typical UK earthquake motion lasting up to 12 seconds, to achieve high spectral accelerations at low frequencies (e.g. below 2.0Hz), the resulting time history must have a high acceleration amplitude, low frequency content for most, if not all, of its duration. Thus, the time history will have a high peak acceleration and, as a consequence, the response spectrum will have a high zero period acceleration. When specifying required response spectra, care must be taken to ensure that low frequency spectral accelerations are consistent with the original earthquake ground motions and that the maximum spectral acceleration is in sensible proportion to the zero period acceleration.

37. High acceleration amplitude, low frequency motions have large displacements which lead to heavy oil flow demands in servo-hydraulic shaking table systems. On the other hand, high amplitude spectral accelerations at higher frequencies are easier to achieve, since the oscillators have more opportunity to reach resonance. Corresponding displacements are small and hence the oil flow demand much reduced.

38. In general, maximum shaking table performance is achieved from time histories which have a low peak acceleration to rms acceleration ratio and have stationary frequency content over their durations. Such motions avoid sudden surges and place a steady demand on oil flows which a servo-hydraulic system can most easily to cope with.

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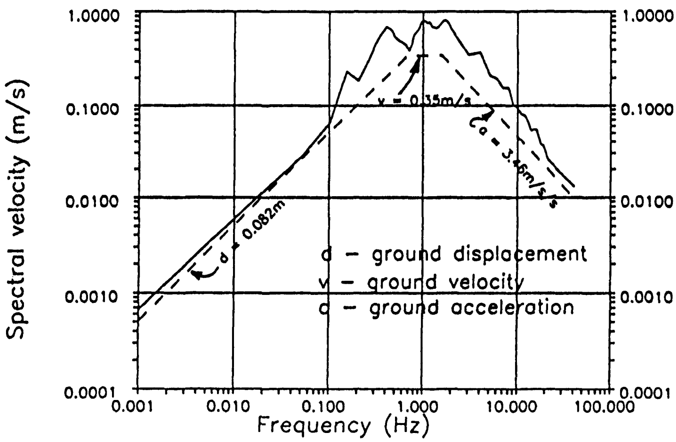

Fig. 1. Typical response spectrum

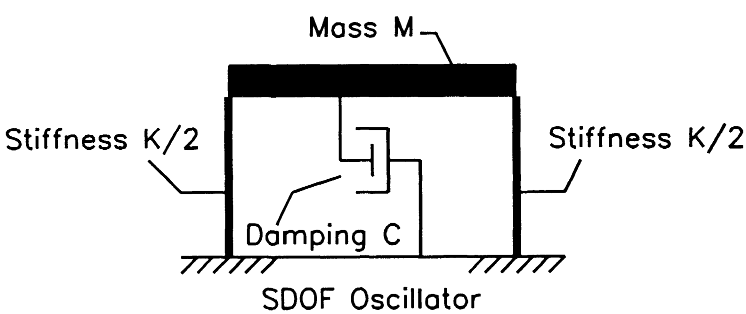

Fig. 2. Single degree of freedom oscillator

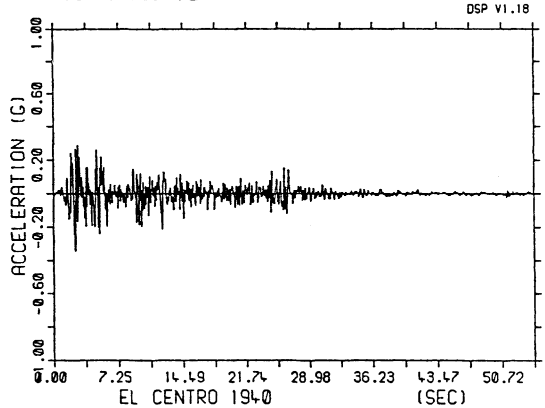

Fig. 3(a). El Centro 1940 earthquake

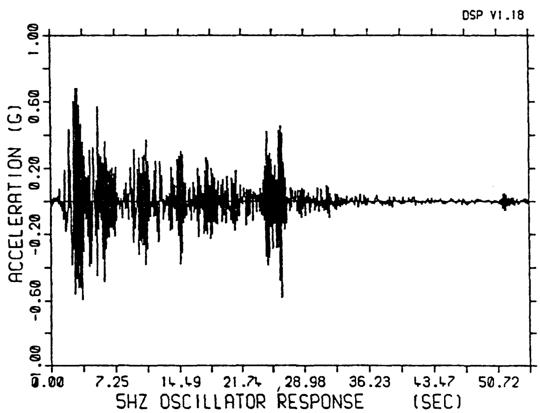

Fig. 3(b). Response of 5Hz oscillator (5% damping) to El Centro earthquake

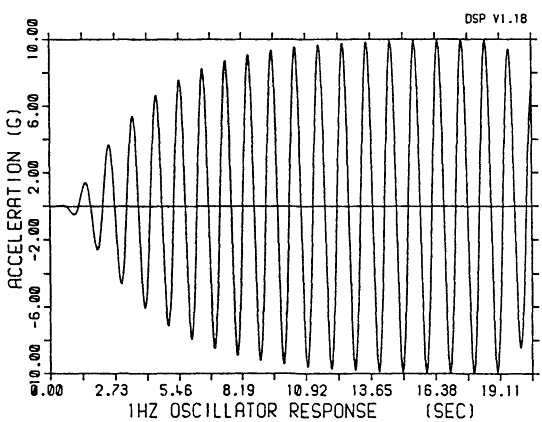

Fig. 4(a). Response of 1Hz oscillator (5% damping) to 1g amplitude, 1Hz sine wave

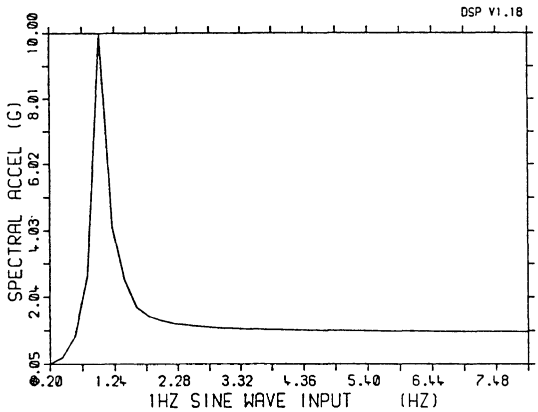

Fig. 4(b). Acceleration response spectrum of 1Hz sine wave (5% damping)

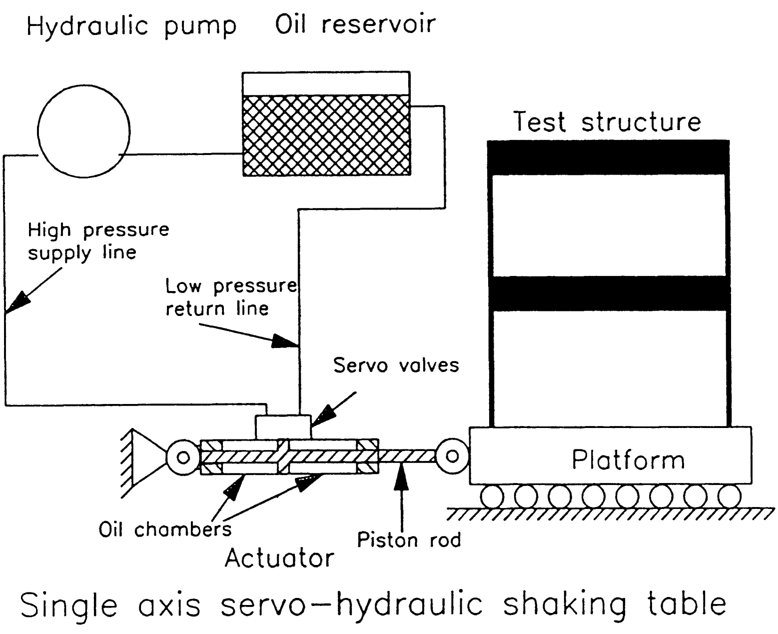

Fig. 5. Simplified shaking table system

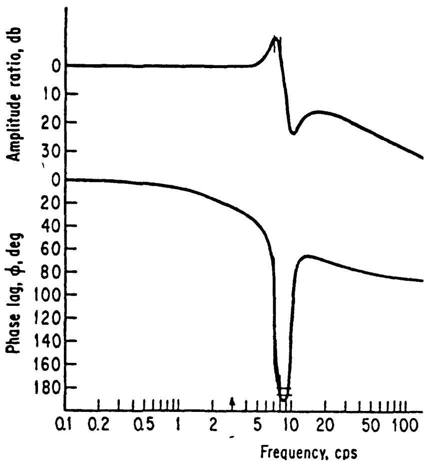

Fig. 6. Shaking table frequency response showing effect of test specimen interaction

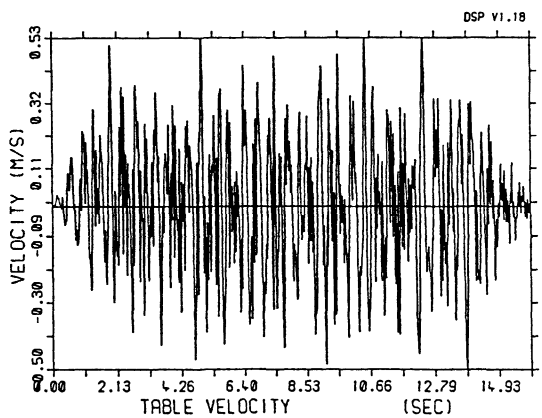

Fig. 7. Typical shaking table velocity time history

11. Seismic soil–structure interaction analysis of embedded multiple buildings using the hybrid continuum impedance approach

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SYNOPSIS. Seismic Soil-Structure Interaction (SSI) analysis has been performed for a site with deeply embedded multiple power block structures. The new method employed combines the computational advantages of the Continuum Impedance Approach together with the Substructure Deletion/Boundary Element Method to accurately predict the 3-Dimensional seismic response of structures.

ABSTRACT

1. Traditional seismic SSI analysis using Finite Element meshes for structures and surrounding soil proved to be quite expensive for consideration of higher frequencies and full 3D interaction effects. To limit this computational cost, various simplifying assumptions need to be made with respect to boundary definitions and mesh sizes, which affect the response results adversely.

2. The hybrid method described in this paper completely eliminates the need to use Finite Element meshes to model the foundation and soil, and in fact, provides a higher frequency seismic motions in a full 3D environment. This, in turn, has also eliminated all boundary-related problems.

3. This paper describes the theoretical background of the method employed in the hybrid CLASSI/ASD program for embedded single and multiple foundations, the advantages with respect to both computation and accuracy over conventional FEM methods, and the overall procedure for use of this method. Results of parametric study of seismic SSI analysis performed for a nuclear power plant in various soil conditions are present.

4. The special application of this program for arbitrary directional waves, as well as the reduction in responses due to wave scattering and embedment is found to be beneficial in reducing predicted seismic responses. The effects of including the adjacent structures in the SSI analysis on the overall response of the subject structure are found to be significant in softer soils.

5. This paper also provides a brief summary of the program's application for SSI analysis to various layered soil media and different embedment depths.

INTRODUCTION

6. The primary objective of this analysis is to calculate the seismic soil-structure interaction (SSI) response of a standard plant Reactor Building embedded in soil sites, including the effects of embedment, soil property variation and adjacent control and turbine buildings. The SSI computer program CLASSI/ASD is used to achieve this purpose.

7. Since CLASSI/ASD code uses basically the continuum impedance approach, it is believed that it will simulate the three dimensional wave propagation effects accurately. Furthermore, CLASSI/ASD uses the Substructure Method, which has the advantage that once the impedance and scattering problems have been solved, they do not have to be repeated if the properties of the structure are changed in the design process. Similarly, if the seismic environment is changed the impedance functions and dynamic structural properties do not have to be recalculated.

8. Six cases of soil structure interaction analysis for three site conditions are performed in this analysis.

ANALYSIS METHODOLOGY

General Theory

9. CLASSI/ASD uses the frequency-dependent Continuum Impedance Approach to model the essentially semi-infinite soil medium, and a Substructure approach which subdivides complicated soil structure interaction problems into more manageable parts.

10. The half-space is first analyzed using a Hybrid Integral Equation Formulation (or sometimes referred as Substructure Deletion Formulation) which combines the source distribution and the mixed distribution of singularities respectively over the top foundation surface and the embedded foundation surface. On the top foundation surface where the geometry is simpler, the free-surface Green's function which satisfies the free-surface boundary condition of a uniformly layered viscoelastic half-space is used in constructing the source integral equation relating the surface displacements and tractions.

11. On the soil volume occupied by the embedded foundation where the geometry can be represented by rectangular blocks, the Stoke's fundamental solutions (the full space 3-D Green's functions) of elastodynamics in an infinite elastic space are used in constructing the mixed integral equation relating displacements and tractions along the embedded foundation-soil interface. A matching condition is then followed by imposing the force equilibrium and displacement compatibility conditions over the top foundation surface. From this the force-displacement relationship (the impedance matrix) of the deleted half-space can be easily deduced.

12. Figure 1 presents a conceptual illustration of the Hybrid method implemented into CLASSI/ASD. The impedances for the embedded foundation in a layered viscoelastic half space is derived as a sum of two parts — (1) Impedances of a layered viscoelastic half space before the soil is excavated, and (2) Impedances of the volume of soil occupied by the embedded foundation.

Calculation of Impedance Matrix

13. The impedance matrix describes the frequency-dependent force-displacement relationship of the foundation(s). In CLASSI/ASD the foundation boundary surfaces of S_t (top surface) and S_e (embedded surfaces) are discretized by equal numbers of elements, N_t and N_e. The unknowns are assumed to be at the center of each element, and to have a constant value over the element. Integrations associated with the Green's functions over the element area are carried out by using the 4-point Gaussian quadrature formula. The harmonic time factor $e^{i\omega t}$ will be omitted hereafter for the clarity. The resulting sets of linear algebraic equations for the top surface and

the embedded surfaces of the foundation can be written respectively, in matrix form, as

$$\{f_t\} = \left[K^h_{tt}\right]\{u_t\} \quad (1) \qquad \text{and} \qquad \begin{Bmatrix} f_t \\ f_e \end{Bmatrix} = \begin{bmatrix} K^f_{tt} & K^f_{te} \\ K^f_{et} & K^f_{ee} \end{bmatrix} \begin{Bmatrix} u_t \\ u_e \end{Bmatrix} \quad (2)$$

where $\{f_t\},\{f_e\}$ = nodal forces acting on surfaces S_t and S_e respectively
$[k_{ij}]$ = impedance matrix of surface i due to forces acting on surface j
$\{u_t\},\{u_e\}$ = nodal displacements over surfaces S_t and S_e, respectively

superscripts h and f are used to differentiate between the half-space and the embedded foundation-soil interface.

14. The matching conditions are then satisfied by substituting equation (1) into equation (2). The force-displacement relationship on surfaces S_e is obtained subsequently as

$$\{f_e\} = \left(\left[K^f_{et}\right]\left(\left[K^h_{tt}\right] - \left[K^f_{tt}\right]\right)^{-1}\left[K^f_{te}\right] + \left[K^f_{ee}\right]\right)\{u_e\} \quad (3)$$

The impedance matrix for the embedded foundation at surfaces S_e is therefore given by

$$\left[K^e_{ee}\right] = \left[K^f_{et}\right]\left(\left[K^h_{tt}\right] - \left[K^f_{tt}\right]\right)^{-1}\left[K^f_{te}\right] + \left[K^f_{ee}\right] \quad (4)$$

The total rigid body foundation impedance matrix about a foundation reference point is therefore obtained as

$$\left[K(\omega)\right] = \left[\alpha\right]^T\left[K^e_{ee}\right]\left[\alpha\right] \quad (5)$$

where $[\alpha]$ = ($3N_e$x6) rigid body motion influence matrix representing the geometry of the foundation-soil interface w.r.t. the foundation reference point

Calculation of Scattering Matrix

15. Scattering, as defined in CLASSI/ASD, is the massless foundation response under the action of a unit harmonic seismic incident wave in the absence of the superstructure.

16. Using matrix notation, the resulting force acting on the foundation-soil interface due to unit amplitude input motion can be written as

$$\{f\} = \left[k^e_{ee}\right]\{u_f\} \quad (6)$$

where

$\{u_f\}$ = ($3N_e$x1) free-field motion vector along the embedment depth due to unit-amplitude control motion applied at grade level

17. Assuming that the foundation is rigid, massless, and perfectly bonded to the soil along the foundation soil interference S_e, the resulting generalized forces acting on the foundation reference point can be written as

$$\{F\} = [\alpha]^T\{f\} \quad (7)$$

18. Knowing $\{F\}$, the rigid-body foundation impedance matrix, $[K]$, as described in equation (5), the normalized frequency-dependent scattering vector, $\{S\}$, can be easily found by solving

$$\{F\} = [K]\{S\} \quad (8)$$

Calculation of the Soil Structure Interaction

19. The fundamental calculation in soil-structure interaction analysis using substructure method in the frequency domain is the calculation of the foundation motion by means of the Fourier synthesis given by:

$$u(t) = \frac{1}{2\pi} \int_{-\infty}^{\infty} U\ (\omega)\ e^{i\omega t}\ d\omega \tag{9}$$

In this equation, u(t) denotes the response in the time domain, while U(ω) represents the frequency response of the foundation displacement. The latter is calculated according to the governing equations of motion given by:

$$\left\langle -\omega^2 \left(\left[M_o\right] + \left[M_B\ (\omega)\right]\right) + \left[\ K\ (\omega)\right]\right\rangle \left\{U\ (\omega)\right\} = \left\{F^*\ (\omega)\right\} \tag{10}$$

where

[Mo] = (6x6) mass matrix of the foundation

[MB (ω)] = frequency-dependent mass matrix of the superstructure subjected to base excitation

{F* (ω)} = (6x1) generalized foundation driving force vector due to seismic input motion

20. The different interaction effects are clearly separated in equation (10). The interaction effects between the foundation, superstructure, and soil (impedance) are represented by the corresponding bracketed terms on the left-hand side of the equation. The effects of scattering of the input motion is included in the term on the right hand side. The total foundation motion, {U (ω)}, is therefore representative of each of the appropriate soil-structure interaction effects.

21. After the unknown, {U}, is determined for each frequency, the complete deformation and/or stress analysis of the superstructure may be accomplished by simply applying {U} as the base excitation parameter. Once all required responses have been calculated in the frequency domain, Inverse Fourier Transform techniques are used to calculate the equivalent response in the time domain.

Interactions Through the Soil Between Adjacent Structures

22. The procedures described in paragraphs 19-22 can readily be extended to analyze the interactions through the soil between adjacent structures. The governing equations of motion, in this case, are still given by equation (10) except that the bracketed terms on both sides of the equation must be redefined. For structures that are not connected to each other and rest on separate foundation mats with N foundations, the mass matrices, $[M_0]$ and $[Mb(\omega)]$,consists of a total of N block diagonal matrices, each of which is (6x6) in size. The coupling effects between the foundations appear in the (6Nx6N) foundation impedance matrix, which has the following form:

$$[K] = \begin{bmatrix} [K_{11}][K_{12}] & \cdots & [K_{1N}] \\ [K_{21}][K_{22}] & \cdots & [K_{2N}] \\ \vdots \quad \vdots & & \vdots \\ [K_{N1}][K_{N2}] & \cdots & [K_{NN}] \end{bmatrix} \tag{11}$$

The foundation motion, {U}, is now a 6N vector composed as

$$\{U\} = \left\{\{U_1\}^T \ \{U_2\}^T \ \cdots \ \{U_N\}^T\right\}^T \tag{12}$$

The driving force vector is also expanded to (6Nx1) and has the form

$$\{F^*\} = \left\{\left\{F_1^*\right\}^T \ \left\{F_2^*\right\}^T \ \cdots \ \left\{F_N^*\right\}^T\right\}^T \tag{13}$$

for each of the wave forms considered.

ANALYSIS CONDITIONS

Design Ground Motion

23. The design ground motion is defined by three artificial statistically independent acceleration time histories. They represent three othogonal directions of the earthquake motion (H1 and H2 are two horizontal components, VT is the vertical component). They are also assumed to be vertically propagating plane shear and compressional waves. Scaling factors are used to yield a 0.15g maximum acceleration for all three directions in the operating basis earthquake (OBE) analysis. Each time history has a duration of 22 seconds and is digitized at 0.01 second time steps. The global X and Y axes correspond to plant 0° - 180° and 90° -270° directions, respectively. Z axis is the vertical direction. A 2% damping response spectra of these acceleration time histories are also calculated. These input motions are assigned to grade level in the soil column analysis using the SHAKE computer program (Ref. 1) as well as in the SSI analysis using CLASSI/ASD.

Soil Conditions

24. Three design soil profiles were used for the analysis, namely UB45, VP3-45 and VP5-45 representing soft, medium and stiff soil characteristics, respectively. For all three sites, the soil deposit depth is considered to be 45.73m (150 ft) and the ground water table is located at 0.61m (2 ft) below grade. Soil column analysis using the SHAKE program was performed in order to take into account the soil nonlinearities in the seismic analysis. Analysis results in terms of the iterated strain-compatible soil properties and the computed deconvoluted motions (maximum accelerations) within soil layers are used to define shear wave velocity, damping ratio, Poisson's ratio and the free-field motion scaling factors for use in the subsequent CLASSI/ASD analysis. Table I. exhibits the soil properties considered in SSI analyses. The soil material damping considered in CLASSI/ASD is assumed to be that of hysteretic type. The complex shear modulus is therefore defined as $G^*=G(1 + 2i\,\xi)$, where ξ is the critical damping ratio of the soil.

25. Table II. summarizes the maximum zero period accelerations at the top of each layer in each of the three soil properties in the free-field subjected to 0.15g-OBE-H2 control motion at grade. As can be seen, the ZPA profiles are consistent with the expected responses. That is, in general, the effects of deconvolution decreases with increasingly stiff soil.

Free Field Motion

26. The description of the free-field motion at a site, involves several aspects. In the first place, a complete description of the motion at a point, usually on the ground surface must be provided.

27. For embedded foundations, the variation of the wave motions over the ground surface as well as into the soil along the foundation-soil interface must

also be given. The assumption of vertically propagating shear or compressional waves, however, drastically simplifies the problem. In this case, if the motion at a point on the ground surface is given, then all other points on the ground surface would experience simultaneously the same motion.

28. For the purpose of the present analysis, the variation of free-field motion with depth is represented by a scale factor to the input motion at ground surface using the results of SHAKE's deconvoluted motion (maximum acceleration) at each soil layer along the embedment depth. This assumption is essentially equivalent to that considered by Chen [Ref. 2] in the case of vertically propagating shear waves. Furthermore, since the non-linear soil properties are obtained by an identical analysis, the assumption is deemed to be appropriate.

SSI Analysis Cases

29. A total of 6 cases are considered in the present SSI analysis. They were chosen such that the effects of the three selected soil profiles (UB45, VP3, VP5) as well as the adjacent control and turbine buildings to the seismic response of the Reactor Building can be studied. The 3 components of the ground input motions are input simultaneously in each of the analysis case since they are statistically independent.

ANALYSIS MODELS

Superstructure Models

30. The site plan of a standard plant is shown in Figure 2. The plan orientations are identified by 0°-180° and 90°-270° directions. Adjacent to the Reactor Building (R/B) along the 0°-180° direction is the Control Building (C/B) and Turbine Building (T/B). These buildings are supported on separate basemats and are embedded to different depths as shown in Figure 3. The C/B and T/B are included in 3 analysis cases as described in paragraph 30 with the purpose to evaluate the effects of structure-to-structure interaction on the reactor building response as function of soil stiffness.

31. The Reactor Building complex is represented by a lumped mass concentric stick model as shown in Figures 4a and 4b. The components included in Figure 4a are the building walls, reinforced concrete containment vessel (RCCV), reactor shield wall (RSW), reactor pedestal, and basemat. Coupling between those structural components is represented by spring elements. Figure 4b shows the model for the reactor pressure vessel (RPV) and its internal components along with the connections to the building structures. The Reactor Building model basically consists of two uncoupled models in the X-Z and Y-Z planes since the lateral-torsional coupling has been found insignificant. The only differences in terms of schematic representations between the X-Z and Y-Z plane models are (1) the two sticks representing the building walls above E1.18.5 (60.7 ft) in the X-Z plane combine into one stick in the Y-Z plane, and (2) the rotational spring between the RCCV top slab (node 90) and the basemat top (node 88) is present only in the X-Z plane.

32. The primary objective of this analysis is to obtain seismic response for the Reactor Building. The Turbine Building response is of no concern but its effect on the Reactor Building through the structure-to-structure interaction is. For this reason, the model for the Turbine Building is rather simple. The turbine pedestal is represented by a single lumped mass stick and building is represented by a 3-lumped mass stick with the foundation mass lumped at the base. The model is assumed rigid in the vertical direction. The horizontal

frequency is 1.27 Hz for the pedestal and 5 Hz for the building. The total weight for the Turbine Building complex considered in the model is about 142,800 tons (314,000 kips).

Foundation Models

33. The foundation is assumed to be rigid. Although no foundation is perfectly rigid, the stiffening effects of the superstructure does, in reality, justify the use of such an assumption. Specifically, each foundation is modelled separately to its individual embedment depth. The top foundation surface is discretized using 25 rectangular subregions while each of the foundation-soil interfaces is discretized using 5 rectangular subregions. Therefore, a total of 50 elements (150 degrees of freedom) is used in the numerical discretization for each foundation as shown in Figure 5. Dasgupta [Ref. 6] demonstrated that the well posedness of the Substructure Deletion formulation can be guaranteed when the number of degrees of freedom associated with the top foundation surface is equal to those associated with the foundation-soil interfaces.

RESULTS AND DISCUSSION

34. The two major parameters which were varied among the 6 cases were (1) the 3 soil profiles (UB45, VP3, VP5) described and (2) the single (R/B only) foundation and multiple (R/B, C/B, T/B) foundation.

Results of the Soil Structure Interaction

35. The maximum Zero-Period Accelerations at selected critical points in the Reactor Building Superstructure are presented for all 6 cases as tabulated in Table III. The nodes selected correspond to the locations are (refer to Figures 4a and 4b), 88—Top of Foundation Basemat; 18—RPV Internals-Shroud Head; 33—RPV/Mainsteam Nozzle; 70—Top of RSW; 71—RPV Support; 89—Top of RCCV; 92—Diaphragm Floor; 95—Top of R/B; 100—Upper Pool Slab; 104—Lower Portion of R/B; 107—Refuelling Floor. It is evident that the amplitude of the spectral acceleration at this point is significantly reduced, especially at the high frequency range as compared to that of the corresponding free-field motion. This is primarily due to the soil-structure interaction effects.

36. Similar comparisons between the free-field input motion and each of the corresponding foundation motions at the top of the basemat reveal comparable reductions in the foundation motion due to soil structure interaction effects.

Effects of Foundation Soil Property

37. Cases 1, 2, and 3 were performed primarily to investigate the effects of the variations in soil properties for the UB45 (soft) site, VP3-45 (medium) and VP5-45 (stiff) sites on the single foundation R/B structural response. Figure 6 provides a comparison of the X-direction response of the R/B for each of the UB45, VP3-45 and VP5-45 soils for the single embedded R/B foundation system. Similarly, cases 4, 5 and 6 were performed to investigate the effects of the variations of soil properties for the UB45, VP3 and VP5 sites on the X-direction response of the R/B when multiple foundations (R/B, C/B, T/B) are considered.

38. It has been seen that the maximum X-direction Zero Period Accelerations increase, in general, with increasing soil stiffness. This trend was expected, since as was observed in the SHAKE analysis, the effect of deconvolution decreased with increasing soil stiffness. Therefore, with all

other things being equal, the amplitude of the deconvolution motions are, in general, larger for stiffer soils.

Effect of Adjacent Buildings

39. Cases 4, 5 and 6 were performed primarily to investigate the interaction through the soil between adjacent structures. Specifically, the effects on the dynamic response of the R/B due to the presences of C/B and T/B for different soil properties are considered.

40. The maximum X-direction accelerations in the soil and superstructure for both the R/B along and the R/B, C/B and T/B multiple-foundation system for UB45, soil profile were plotted. For the softest soil profile, the effect of including the C/B and T/B tends to increase the response at top of the R/B by approximately 37%. However, increases in the responses of the RPV, Internals and RSW/PED are much higher. Also, by contrast to the comparison for the soft soil (UB45), the effect on the response of the R/B for the multiple foundation analysis for soil profiles VP3 and VP5 is, in general, considerably lower than that of the single R/B.

41. This is primarily due to (1) the observation that the maximum accelerations increased more significantly with increasing soil stiffness for the single R/B foundation than for the multiple foundation, and (2) the effect of the structure-to-structure interaction is expected to be less significant in the stiffer soil sites.

42. Note also that the maximum X-direction acceleration at the top of the basemat (Node 88) was found to be consistently larger for the multiple foundation system than that for the single R/B foundation. This is attributed to much higher mass of the multiple foundation system (total weight: 359,000 tons) compared to that for the R/B along (total weight: 192,000 tons).

43. As to the comparison of the acceleration response spectra, it was observed that the multiple foundation analysis exhibits a high peak at a frequency of 5.0 Hz at the foundation basemat level for the soft soil profile (UB45 - CASE 4). This is due to the effects of structure-to-structure interaction, or the so-called through-the-soil interaction. Specifically, two major factors contribute to these effects. (1) The interaction effect due to the structural impedance of the adjacent T/B superstructure which is of comparable inertia to the R/B. Since the dynamic characteristics of the R/B superstructure demonstrates that there is no fixed-base model with a frequency close to 5.0 Hz, evidently these strong responses are caused by the existence of the T/B, which having a fundamental frequency at 5.0 Hz, manifests itself in the form of relatively large peak spectral acceleration in the foundation response of the R/B at the same frequency. (2) The interaction effect due to the coupling between rocking and horizontal responses through the foundation impedance. This interaction effect is due to the X-direction alignment of the multiple foundations and the simultaneous inputs of the horizontal and vertical ground motions considered in the present analysis.

44. To summarize, as observed from the present analysis results, the effect of the adjacent buildings on the response of the R/B consists of the following important factors, (1) direction of alignment between adjacent buildings, (2) structural inertia of each building, and (3) stiffness of the underlying soil medium.

SUMMARY AND CONCLUSIONS

45. Presented herein are the results of the seismic soil-structure and structure-to-structure interaction analysis of the standard plant Reactor Building embedded in 3 selected soil sites. Basically the effects of soil structure interaction on the dynamic response of the Reactor Building due to 3 different foundation soil properties as well as the adjacent control and turbine buildings are investigated through a series of parametric studies.

46. The main conclusions drawn from the present study for this specific soil-structure system are:

1. in using the Substructure Method of Analysis, it is very important to obtain an accurate foundation response (at the basemat level) since the evaluation of the entire superstructure responses are based on the computed foundation motion.
2. the presence of the foundation embedment significantly reduces the foundation responses in all of the 3 global X-, Y-, and Z-directions, especially at high frequencies.
3. the effect of the soil stiffness is quite important to the response of the Reactor Building for both single and multiple foundation system. In general larger responses are predicted for harder soil
4. the effect of including the C/B and T/B, in general, tends to increase the response of the R/B in all of the global X-, Y-, and Z-directions. For the case of the soft soil site (UB45), the increase in the global X-direction response is especially noticeable. This is solely due to the through-the-soil interaction effects. For the medium (VP3) and stiff (VP5) soil sites, however, reductions in the maximum Zero Period Accelerations (ZPA) are seen in most of the horizontal -X degrees of freedom in both Reactor Building walls and RCCV above grade level.
5. when considering the effect of structure-to-structure interaction, not only the direction of alignment of these structures and their corresponding structural inertias, but also the variation of the soil stiffness interaction effects are more pronounced in the softer soil sites.

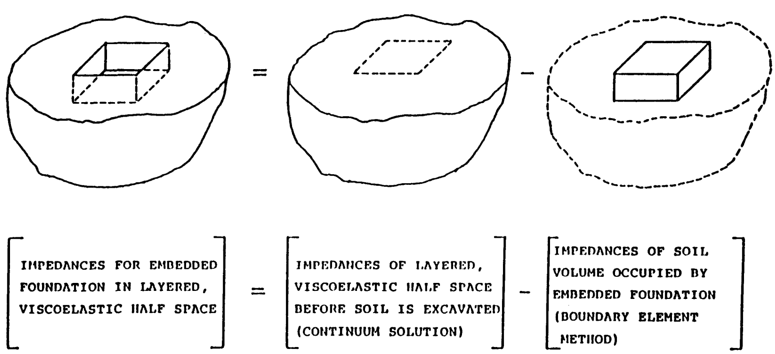


FIGURE 1
CONCEPTUAL ILLUSTRATION OF HYBRID METHOD IN CLASSI/ASD
USING SUBSTRUCTURE DELETION AND BOUNDARY ELEMENT METHOD

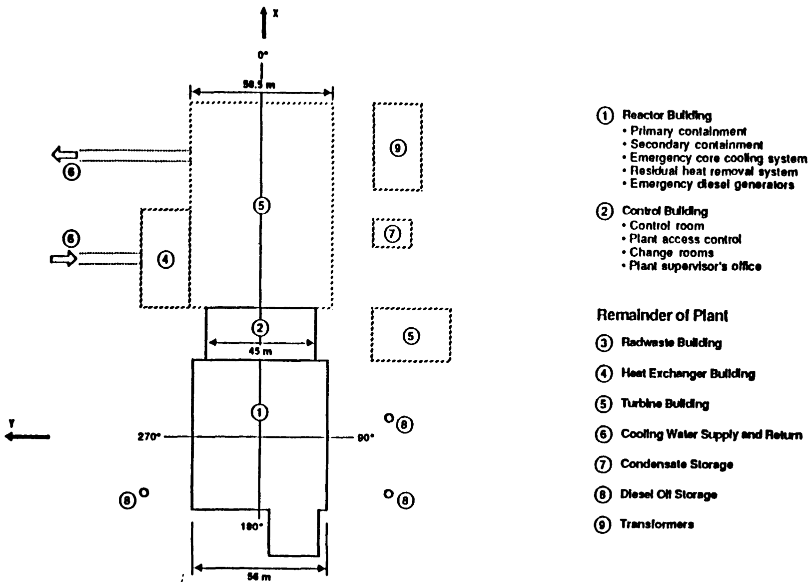


FIGURE 2
TYPICAL SITE PLANT

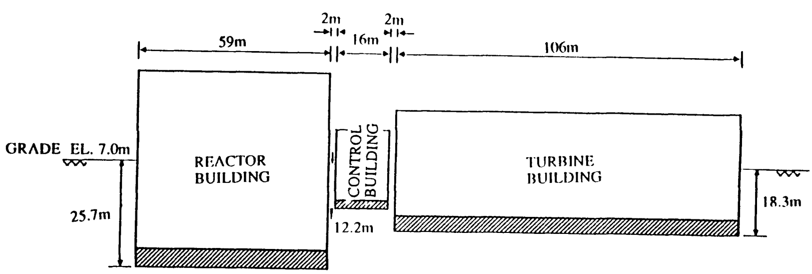
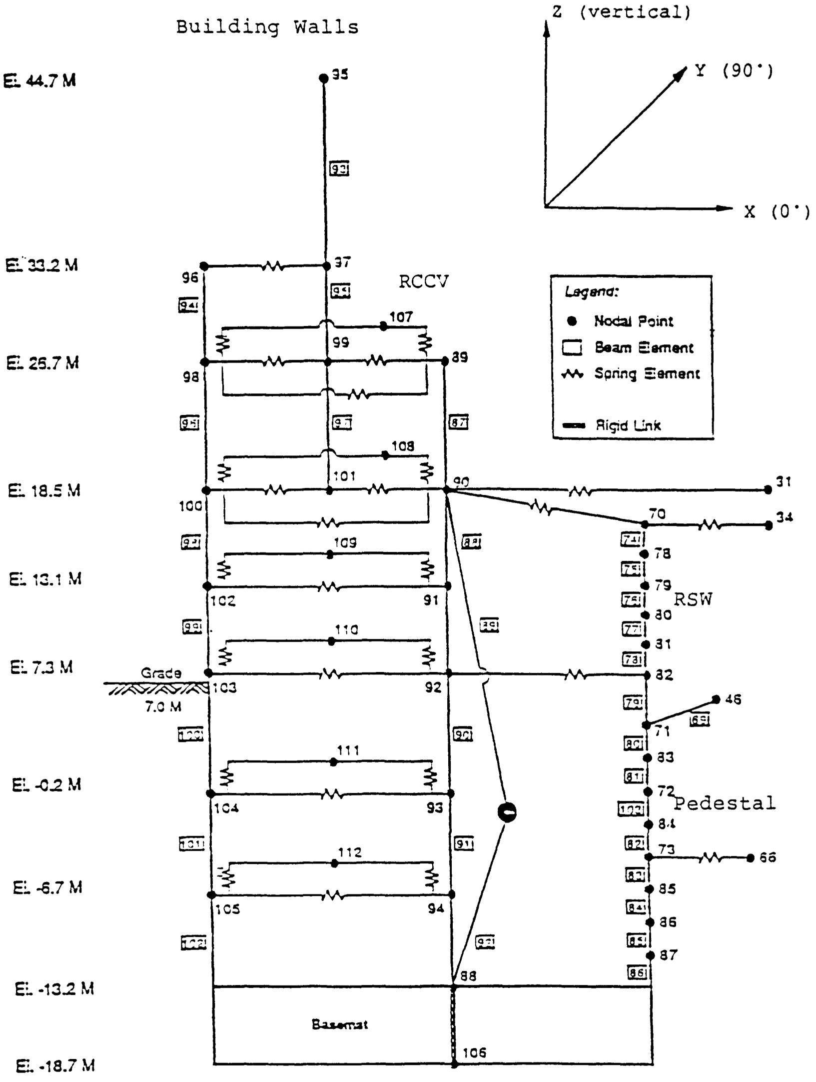


FIGURE 3
0°-180° SECTION VIEW

TABLE I
SUMMARY OF SOIL PROPERTIES FOR SSI ANALYSIS

LARGE ELEVATION (M)	UNIT WEIGHT TON/M^3)	CRITICAL DAMPING RATIO			POISSON'S RATIO			SHEAR WAVE VELOCITY (M/SEC)		
		UB45	VP3-45	VP5-45	UB45	VP3-45	VP5-45	UB45	VP3-45	VP5-45
0 - .61	1.92	.008	.006	.005	.38	.38	.38	301	549	891
.61 - 3.05	1.92	.022	.011	.007	.48	.42	.38	285	541	884
3.05 - 6.10	1.92	.037	.018	.010	.48	.42	.38	270	532	875



Notes: (1) In Y-Z plane, the two reactor building sticks above El. 18.5m combine into one single stick.

(2) The rotational spring between nodes 90 and 88 is present only in the X-Z plane.

FIGURE 4a
FINITE ELEMENT CENTER STICK MODEL OF REACTOR BUILDING WALLS, REICFORCED CONCRETE CONTAINMENT VESSEL, SHIELD WALL, PEDESTAL AND BASEMAT

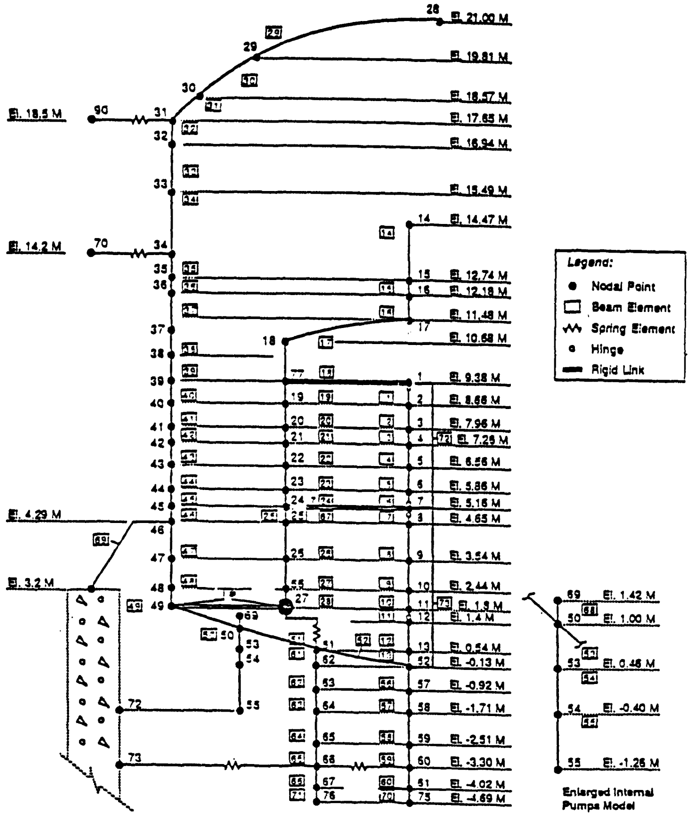


FIGURE 4b
FINITE ELEMENT CENTER STICK MODEL OF
REACTOR PRESSURE VESSEL (RPV) AND INTERNALS

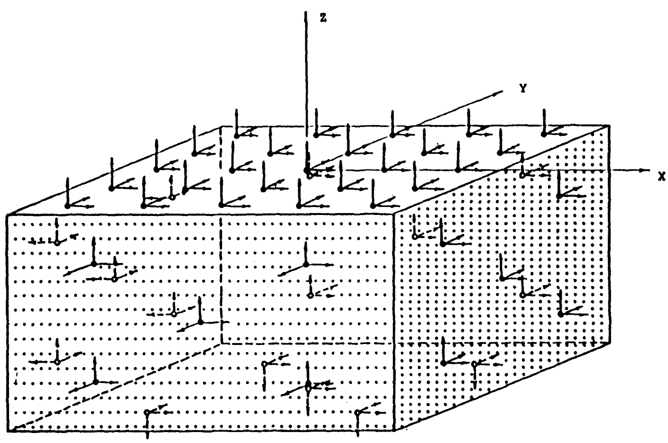


FIGURE 5
BOUNDARY ELEMENT REPRESENTATION OF A TYPICAL
EMBEDDED RECTANGULAR FOUNDATION

TABLE III
COMPARISION OF MAXIMUM ACCELERATIONS OF A PLANT REACTOR BUILDING FOR CASES STUDIED

STRUCTURE	ELEVATION (m)	NODE	DOF*	MAXIMUM ACCELERATION (g)					
				CASE 1	CASE 2	CASE 3	CASE 4	CASE 5	CASE 6
R/B	44.7	95	1	0.2387	0.4445	0.5508	0.3272	0.3747	0.6059
		95	2	0.2539	0.5198	0.6085	0.3634	0.5976	0.5834
		95	3	0.2057	0.1957	0.2235	0.2386	0.2189	0.2506
	18.5	100	1	0.1687	0.2822	0.3584	0.2377	0.2206	0.3252
		100	2	0.1873	0.2934	0.3181	0.1926	0.3578	0.2971
		100	3	0.1499	0.1685	0.1939	0.1811	0.1756	0.2244
	-0.2	104	1	0.1195	0.1596	0.1968	0.1675	0.1679	0.2263
		104	2	0.1252	0.1720	0.1666	0.1557	0.1966	0.1666
		104	3	0.1086	0.1368	0.1648	0.1231	0.1367	0.1876
	26.7	107	3	0.2917	0.4090	0.4957	0.4496	0.4297	0.5973
RCCV	26.7	89	1	0.1920	0.3454	0.4620	0.2888	0.2653	0.4353
		89	2	0.2288	0.3834	0.4370	0.2737	0.4622	0.4100
		89	3	0.1637	0.1851	0.2155	0.2043	0.2051	0.2504
	7.0	92	1	0.1408	0.2124	0.2577	0.2062	0.2104	0.2790
		92	2	0.1510	0.2262	0.2276	0.1587	0.2775	0.2237
		92	3	0.1370	0.1654	0.1968	0.1698	0.1746	0.2268
RSW/PED	14.2	70	1	0.1849	0.3008	0.3768	0.3786	0.2997	0.3956
		70	2	0.1831	0.2832	0.3233	0.2154	0.3596	0.3317
		70	3	0.1191	0.1386	0.1837	0.1651	0.1430	0.2397
	3.2	71	1	0.1301	0.1838	0.2273	0.2016	0.1910	0.2516
		71	2	0.1383	0.2008	0.1948	0.1626	0.2390	0.1941
		71	3	0.1121	0.1328	0.1744	0.1512	0.1374	0.2099
	-13.2	88	1	0.0967	0.1375	0.1410	0.1556	0.1537	0.1990
		88	2	0.1051	0.1356	0.1357	0.1582	0.1256	0.1324
		88	3	0.1052	0.1246	0.1476	0.1185	0.1227	0.1664
RPV	10.68	18	1	0.1810	0.2948	0.4021	0.4557	0.4145	0.4813
		18	2	0.1734	0.2687	0.3657	0.2029	0.2989	0.3651
		18	3	0.1261	0.1437	0.1931	0.1964	0.1487	0.2884
	15.49	33	1	0.2193	0.3348	0.4334	0.5299	0.4130	0.4916
		33	2	0.2007	0.3171	0.3730	0.2601	0.3833	0.3996
		33	3	0.1178	0.1373	0.1820	0.1642	0.1415	0.2377

* DOF 1 = X-DIR TRANSLATION; DOF 2 = Y-DIR TRANSLATION; DOF 3 = Z-DIR TRANSLATION

TABLE II
MAXIMUM ACCELERATION (ZPA) PROFILES IN FREE-FIELD FOR SELECTED SOIL SITES

LAYER ELEVATION (M)	MAXIMUM ACCELERATION (ZPA)		
	UB45	VP3-45	VP5-45
0	.154	.154	.154
.61	.154	.154	.154
3.05	.149	.151	.153
6.10	.144	.150	.150

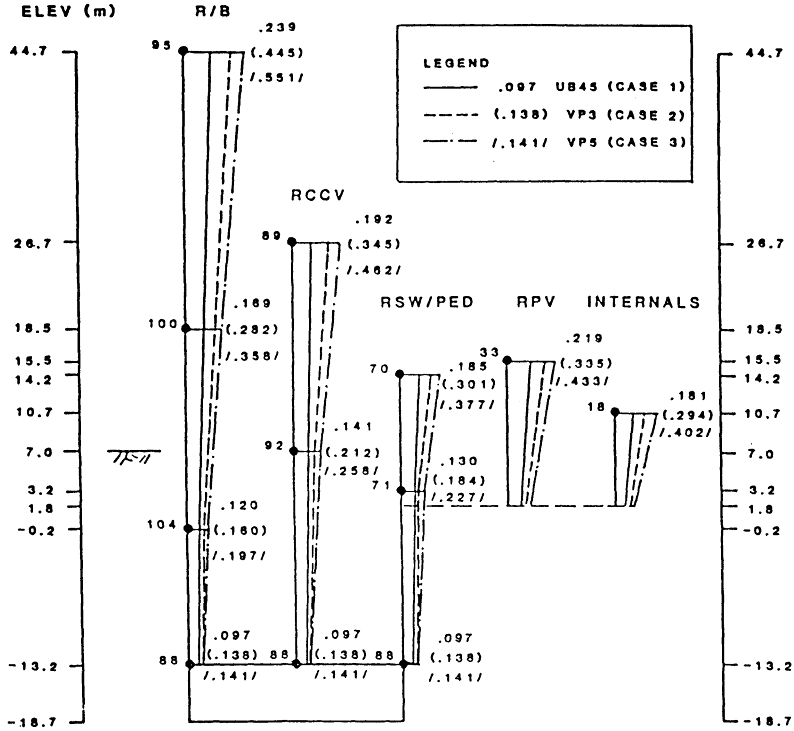


FIGURE 6
EFFECTS OF SOIL CONDITIONS ON MAXIMUM X-DIRECTION
ACCELERATIONS OF REACTOR BUILDING - R/B ONLY

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Discussion

R. L. BRUCE, ABB Impell Ltd, Warrington
I would like to congratulate Professor Key on advocating a power spectral approach, which offers many advantages over the response spectral approach. However, the accepted problem with the power spectral method is the difficulty in representing the non-stationarity of the input and output motions. I can see three ways in which the non-stationarity can be modelled in this method. Information from the computed output spectra coupled with an assumed envelope for the response can be combined with statistical extreme value theory to predict maxima. Secondly, Monte Carlo simulation can be applied to the power spectrum. The resulting time history can then be enveloped and the maximum extracted. Thirdly, the power spectrum used in the paper can be replaced by the complex valued Fourier spectrum. This retains the phase information, and as a result models the envelope of the input motion which is carried through into the response.

Would Professor Key comment on which of these methods he advocates (if any), and whether he sees any mileage in the other methods?

C. A. TAYLOR, University of Bristol
Regarding the question of generating realistic sets of phase angles for use in Fourier spectrum based analyses, one solution is to generate response spectrum compatible time histories and to use the phase contents of these. There is a direct relationship between velocity response spectra and Fourier spectra; the zero-damped velocity response spectrum is the Fourier amplitude spectrum. Care must be taken to ensure that peak acceleration to peak velocity ratios etc. are sensible by selecting an appropriate set of phase angles.

J. R. MAGUIRE, Lloyd's Register, Croydon
I note from Paper 11 that Class I verification studies have been carried out. For this particular boiling water reactor, have any full-scale measurements been carried out, post-construction, to validate the analysis? If not, what alternative validation has been carried out?

B. COURTIER, W. S. Atkins Consultants Ltd, Epsom
The cross-section of the station analysed by Mr Unemori showed a very deep excavation. Such an excavation in soft soils would change the soil properties. Did Mr Unemori investigate the effect of the stress history on the soil properties.

D. SHEPHARD, NNC Ltd, Knutsford
I observe that one of the significant advantages of the response spectrum approach is that it enables the primary system analysis to proceed in the absence of detailed knowledge of the secondary system. Professor Key's recommended revised approach takes account of both primary and secondary systems simultaneously; how could this be reconciled with the practical situation where designs endure while the analysis is in progress?

C. I. ROBERTSON, Sir William Halcrow and Partners, Rosyth
The Authors of Paper 8 make reference to conservatism arising from the combination of responses to the different spatial components of the earthquakes, before the issue of secondary response spectra. How would the Authors like this matter to be addressed in the future?

J. H. MILLS, Allott & Lomax, Manchester
A comment and two questions. The comment is that Newell's and Key's papers confirm my prejudices that earthquake loading, and in particular floor response spectra, are a self-inflicted wound. It is a wound that the industry can do without, and one that the industry must solve.

On the theme of interfaces, the recommendation to use the simplest model possible brings with it the introduction of a further interface within the analysis process itself. Would the Authors of Paper 8 please comment?

Do Paper 10's comments on the difficulties with servo systems apply to shake tables in general, or the Bristol Table in particular?

G. N. TROTT, W. S. Atkins Consultants Ltd, Warrington
Could Mr Unemori please confirm how the strain-dependent soil properties are incorporated into the CLASSI-ASD analysis? Are the primary non-linearities posed on an equivalent linear approach using a one-dimensional program such as SHAKE? Further, have the secondary non-linearities due to structure-soil interaction been incorporated into the analysis?

J. P. NEWELL, Paper 8
Our Paper explains that conservatisms in the seismic analysis and design process arise at interfaces between different organizations involved, between engineering disciplines and at convenient points in the analysis procedure. We advocate the use of the simplest analysis approach that will provide a realistic estimate of seismic response, and this may include the elimination of unnecessary conservatism arising at interfaces. The choice of an appropriate seismic modelling method for a particular project will depend not only on those considerations but also on factors such as cost and programme requirements.

D. KEY, Paper 9
In reply to Mr Bruce, the three methods proposed for dealing with non-stationarity are all valid approaches which lead to fairly extensive computation. The alternative proposed in the Paper (equations (6) and (7)) requires negligible computer resources and is well within acceptable limits of accuracy. However, as computating costs reduce and speeds increase it may become desirable to use one of the methods you propose, especially if the probability distribution of response is required.

The solution to Mr Shepherd's problem is in the design management process. If reasonable margins for primary-secondary interaction are allowed, based on sensitivity calculations (which follow directly from the proposed method of analysis), the combined primary-secondary model can be monitored as the design evolves. For the final design, all the actual margins will be known.

12. Design and construction of the primary containment for the Sizewell 'B' PWR

J. IRVING, BSc, FICE, PWR Project Group, Nuclear Electric plc, and
R. CROWDER, BSc, ACGI, MICE, Special Projects Division, Taywood Engineering Ltd

A) SYNOPSIS. The primary containment structure for Sizewell 'B' PWR is the result of a number of years intensive development and design by Nuclear Electric (formerly Central Electricity Generating Board) and their design consultants Nuclear Design Associates (NDA). This paper identifies the significant features of the design, and the main stages of construction achieved thus far.

B) INTRODUCTION

1. Development of the PWR has been active in the UK since 1979, and in 1981 a decision was taken to adopt the standard nuclear unit power plant system (SNUPPS) concept developed in the United States for prototype plants at Calloway (Missouri) and Wolf Creek (Kansas).

2. From the very early days it was a condition of using the Westinghouse plant that the UK adaptation of the system should be "licensable in the country of origin". The UK licensing authority, the Nuclear Installations Inspectorate (NII) therefore required that the design and construction specification for the primary containment should be broadly based on US codes with particular reference to the ASME Boiler and Pressure Vessel Code Section III Division 2.

3. The Sizewell PWR primary containment is defined as the outer shell structure and base slab of the reactor building and is designed for a life of 40 years. It must fulfil a number of important structural functions during this period whilst subjected to the loads and loading combinations set out in table 1.

4. The containment is the last principal barrier between a possible, though highly unlikely, reactor accident and the outside environment. It is therefore essential that this structure should be of the highest quality of design and construction. The primary containment is classed as a Category 1 structure, and as such, must withstand a Safe Shutdown Earthquake (SSE), and subsequently permit the reactor to be safely shut down and cooled. Concurrent with the SSE, the structure must also be capable of withstanding and

containing the effects of a design basis fault (ie loss of primary circuit coolant).

C) CONTAINMENT STRUCTURAL DESCRIPTION

5. The primary containment takes the general form depicted in Fig 1, with a prestressed concrete cylinder and dome founded on a reinforced concrete base. The cylinder shell is of 45.7m ID with a wall thickness of 1.3m and it is surmounted by a hemispherical dome of 1.0m thickness. The reinforced concrete base of some 3.85m thickness is essentially unpenetrated, but has a central depression to accommodate reactor instrumentation lines. An annular prestressing gallery is located around the periphery of the underside of the base. Major penetrations are located in the cylindrical wall of the containment where the major advantage of prestressed concrete is that this part of the structure and the dome will be essentially free from through cracks under accident conditions. The containment is sized so that the net free volume limits the internal pressure rise to a design pressure of 0.345 MPa (maximum fault pressure +10% margin).

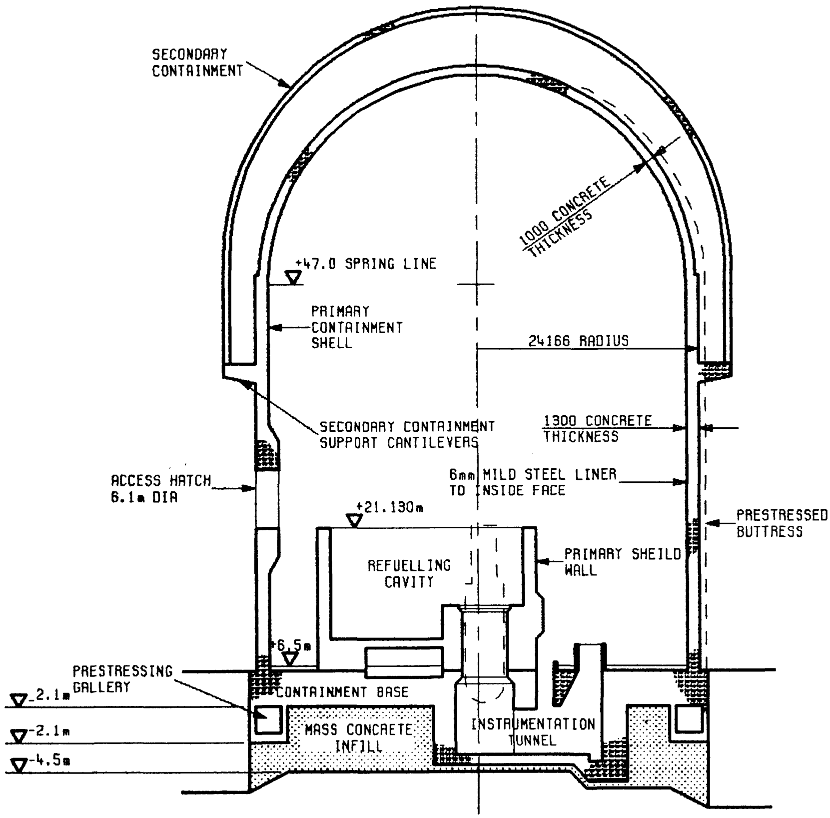

Fig 1. Vertical section through primary containment

6. Prestress is applied in the meridional direction by 74 No. 'up and over' tendons each of 11.1 MN ultimate capacity. These are anchored equi-spaced on the underside of the base, run vertically up the wall, and cross the dome on 'small circles' forming an orthogonal pattern. Hoop prestress is applied by 107 No. hoop tendons starting near the bottom of the cylinder wall and extending to a 45^{o} angle over the dome. The hoop tendons are anchored on three vertical buttresses equi-spaced at 120^{o}, with each tendon passing through 240^{o} between the anchored ends.

7. The hoop and meridional tendons impose a prestress 'pressure' equivalent to 1.2 x design pressure at the mid-buttress and dome apex positions where friction losses are a maximum, and at containment late life when other prestressing losses would be at a maximum.

8. The prestressing system employed is the Freyssinet/PSC 37 K 15, in which each tendon comprises 37 No. 15.2mm diameter 'compacted' strands. Anchorage is achieved by the use of tapered wedges with teeth to grip the strands located in similarly shaped holes in a special bearing plate. The tendons are housed in spirally wound ungalvanised mild steel ducts of 130mm internal diameter. Corrosion protection is provided by a combination of oil and grease compounds on the various elements. This arrangement is preferred to cement grouting as it provides the opportunity to periodically inspect sample tendons for corrosion or to remove and replace them completely. It also allows load checking and load adjustment of the tendons to be carried out if required during the life of the structure.

9. The tendon pattern is modified as necessary to accommodate the large number of penetrations through the containment wall. The largest of these is the 6.1m diameter equipment access hatch. Two 3.25m diameter penetrations provide for personnel access. Numerous other smaller penetrations cater for electrical, control and instrumentation, and steam and feed penetrations to the boilers, as well as penetrations for the reactor safety systems. Closures of steel plate construction are provided to the main hatch and access penetrations.

10. The internal surface of the primary containment is covered by a 6mm thick mild steel liner which serves as a leak-tight membrane during the life of the structure. The function and design of the liner are described in more detail in section G.

11. In addition to the prestress, bonded reinforcement is provided in the walls and dome of the structure for general crack control purposes and to cope with local highly stressed zones. The bonded reinforcement, unlike the liner, is taken

into account in assessing the ultimate load capability of the structure.

12. Though not specifically the subject of this paper, a secondary containment structure surrounds the primary containment with a nominal 3m interspace maintained below atmospheric pressure. By this means any leakage from the primary containment will be contained within the reactor building. The secondary containment structure is of 0.3m thick lightweight concrete and the dome portion is supported on cantilevers which project from the primary containment near the top of the cylinder wall. These cantilevers are designed as discrete elements with radial joints so that hoop stiffening effects at this location on the primary containment are avoided.

13. At a relatively late stage in the design, it was decided to replace the original internal steel bracket arrangements for support of the polar crane with a novel reinforced concrete corbel design (ref 8). These corbels are located 3.95m below the spring line and are 2.0m deep at the throat and protrude 0.24m into the containment. With this design, the polar crane rail can be mounted directly on top of the corbels and the circular crane rail girder omitted. Like the cantilevers supporting the secondary containment, the corbels are divided into 48 discrete sections to eliminate hoop stiffness effects.

D) DESIGN CODE

14. Although British Standard BS4975 is available for use in the design of prestressed concrete pressure vessels for reactor systems, there is no UK code for the design of concrete containment structures. Therefore, a special document entitled 'Design and Construction Rules for PWR Primary Containment' was established for Sizewell 'B', which covers both the concrete structure and liner (ref 1). This now stands as the 'design bible' for UK primary containments, although basically derived from the US Code - ASME III Division 2 (Subsection CC). Broadly, the document comprises three parts :

- Part 1 covers definitions, loads to be considered, loading combinations and design allowables.

- Part 2 is the technical specification for main structural components viz concrete, prestress, reinforcement.

- Part 3 shows how ASME has been changed in arriving at the UK equivalent.

15. The principal differences between ASME and the UK D & C rules are :

	Load combinations	Loading category: Construction	Normal	Exceptional	Ultimate	D	L	F	W	W′	E	E′	P_1	T_1	P	T	P_2	T_2	R_1	R_2	R_3	R_4	R_5	Y	Z
1	Construction	•	–	–	–	1·0	1·0	1·0	0·67	–	–	–	–	1·0	–	–	–	–	–	–	–	–	–	–	–
2	Structural overpressure	–	•	–	–	1·0	1·0	1·0	–	–	–	–	–	1·0	1·15	–	–	–	–	–	–	–	–	–	–
	Integrated leak rate	–	•	–	–	1·0	1·0	1·0	–	–	–	–	–	1·0	1·0	–	–	–	–	–	–	–	–	–	–
3	Normal operation	–	•	–	–	1·0	1·0	1·0	–	–	–	–	1·0	1·0	–	–	–	–	1·0	–	–	–	–	–	–
4	Normal operation + Normal climatic	–	–	•	–	1·0	1·9	1·0	1·5	–	–	–	1·0	1·0	–	–	–	–	1·0	–	–	–	–	–	–
5	Normal operation + operational shutdown earthquake (OSE)	–	–	•	–	1·0	1·3	1·0	–	–	1·5	–	1·0	1·0	–	–	–	–	1·0	–	–	1·0	–	–	–
6	Normal operation + exceptional climatic	–	–	•	–	1·0	1·0	1·0	–	1·0	–	–	1·0	1·0	–	–	–	–	1·0	–	–	–	–	–	–
7	Normal operation + safe shutdown earthquake (SSE)	–	–	•	–	1·0	1·0	1·0	–	–	–	1·0	1·0	1·0	–	–	–	–	1·0	–	–	–	1·0	–	–
8	Design basis fault a	–	–	•	–	1·0	1·0	1·0	–	–	–	–	–	–	1·5	1·0	–	–	–	1·0	–	–	–	–	–
	b	–	–	•	–	1·0	1·0	1·0	–	–	–	–	–	–	1·0	1·0	–	–	–	1·25	–	–	–	–	–
	c	–	–	•	–	1·0	1·0	1·0	1·25	–	–	–	–	–	1·25	1·0	–	–	–	1·0	–	–	–	–	–
9	Design basis fault + safe shutdown earthquake	–	–	•	–	1·0	1·0	1·0	–	–	–	1·0	–	–	1·0	1·0	–	–	–	1·0	–	–	1·0	–	–
10	Normal operation + pipe rupture other than DBF	–	–	•	–	1·0	1·0	1·0	–	–	–	–	–	–	–	–	1·0	1·0	–	–	1·25	–	–	–	–
11	Normal operation + missiles	–	–	•	–	1·0	1·0	1·0	–	–	–	–	1·0	1·0	–	–	–	–	1·0	–	–	–	–	1·0	–
12	Normal operation + other hazardous loads or malfunctions	–	–	•	–	1·0	1·0	1·0	–	–	–	–	1·0	1·0	–	–	–	–	1·0	–	–	–	–	–	1·0
13	Ultimate pressure load	–	–	–	•	1·0	–	1·0	–	–	–	–	–	–	K	–	–	–	–	–	–	–	–	–	–

Symbols used on this drawing

D Dead loads (permanent gravity loads)
E Earthquake loads (DSE)
E′ Earthquake loads (SSE)
F Prestressing loads (calculated at all relevant times to be considered)
L Live loads (imposed loads)
P Design pressure
P_1 Operating pressure
P_2 Internal pressure loads due to accident conditions other than DBF
R_4 Local loads due to earthquake (OSE)
R_5 Local loads due to earthquake (SSE)
R_1 Local loads under normal operation
R_2 Local loads due to pipe rupture under DBF conditions
R_3 Local loads due to pipe rupture other than that caused by DBF conditions
T DBF temperature distribution
T_1 Operating temperature distribution
T_2 Internal temperature loads due to accident conditions other than DBF
W Wind loads (normal)
W′ Wind loads (exceptional)
Y Local loads due to missiles
Z Loads from any other hazards or malfunctions to be considered

Table 1. Table of loads and loading combinations

(a) The definitions are more in line with the UK prestressed concrete pressure vessel code BS4975.

(b) Again in line with BS4975, an ultimate load requirement is specified for the containment.

(c) Design allowables are expressed in UK terms eg., concrete as 28 day cube strength, and reinforcement yield for UK steel.

(d) For the liner, guidance is given for assessment of anchor failure taking account of non-linear behaviour under temperature loading. Also a limit is placed on liner tensile strain and advice is given for coping with peak strains in very localised areas.

E) STRUCTURAL ANALYSIS AND DESIGN

16. The behaviour of the primary containment under the specified design basis loadings (see table 1) has been the subject of a number of different analyses (refs 2, 3 and 7. The loads include exceptional loads which have a probability of exceedance less than 10^{-4} per annum. These include the Safe Shutdown Earthquake (SSE) corresponding to a peak horizontal acceleration in the free field of 0.25g and the design pressure (P) which is set at a level some 10% above the peak pressure resulting from a design basis fault. For Sizewell, the design pressure is 0.345 MPa (50 psig). Some fifty different analysis reports, were certified and endorsed by the Independent Inspection Agency (IIA) and made up the containment design report submitted to the NII. The bulk of the structural analyses were performed using four computer codes as set out in the following table 2 :

Table 2

Scope \ Type	Elastic	Dynamic	Non-Linear
Axisymmetric	Pafec		Adina
3D	Pafec	MSC/Nastran	
Settlement	Setmod		

17. Dynamic analysis. The main dynamic analysis was for seismic loading, using the MSC/NASTRAN code. An early fixed base analysis model comprised a lumped mass beam representation of the containment base raft, the mass concrete below and the internal structures, with the shell represented as a super element. At a later stage, a more refined seismic analysis model was carried out with the secondary containment modelled as 2D plate elements.

18. Shell analysis (elastic). A global elastic shell analysis was performed using the finite element code PAFEC as an axisymmetric model which extends into the ground to some 50m depth. The internal structures were partially represented and the base topping slab is represented by material of low modulus so as not to add unduly to the base stiffness.

19. Penetrations were omitted from this global model as were the effects of the secondary containment. In the base, the asymmetric reactor cavity/instrumentation tunnel was simplified as an axisymmetric structure.

20. The axisymmetric model was used particularly to analyse the stress situation at the wall/base junction. At this location, the wall inner face design was controlled by the design basis fault loading combination which includes a factor of 1.5 on accident pressure. The outside face design, where concrete compression is critical, was controlled by structural over pressure test conditions (1.15P). Higher up the barrel, the design was controlled by the loading combination involving accident pressure (1.0P) combined with SSE. Beyond some 20m height from the base and over the containment dome, only nominal reinforcement was required, 0.21% each way each face.

21. Assessment of local effects, for example in the stiffened area around the equipment access hatch, was achieved by a 3D finite element analysis.

22. Base analysis (elastic). The effects of symmetrical mechanical loads and temperature on the base were generally determined from the global axisymmetric model used for the shell. However, it was considered that this model would not suffice to assess the effects of asymmetric loads on the structure, nor provide a realistic assessment of local stresses in the reactor cavity and instrumentation tunnel. Thus a 180° 3D model of the base and soil was first assessed with the shell and dome represented by 2D plate elements and with the base itself simplified to a flat circular plate (fig 2). The results from this model were then used to derive pressure and spring constants at the base/soil interface for asymmetric loads for use on further 3D models without soil representation. The constants for axisymmetric loads were derived from the global axisymmetric analysis.

23. Two further models of 180° extent were developed. Both included the shell wall up to approximately mid-barrel height only, but with a full representation of the prestressing gallery and instrumentation tunnel. The continuity of the barrel and associated loads above mid-height was represented by boundary conditions at the cut surface. These models were analysed for the load cases selected as likely to be the most onerous. In general terms, it was found that the upper and lower mats of reinforcement in the base were controlled by the

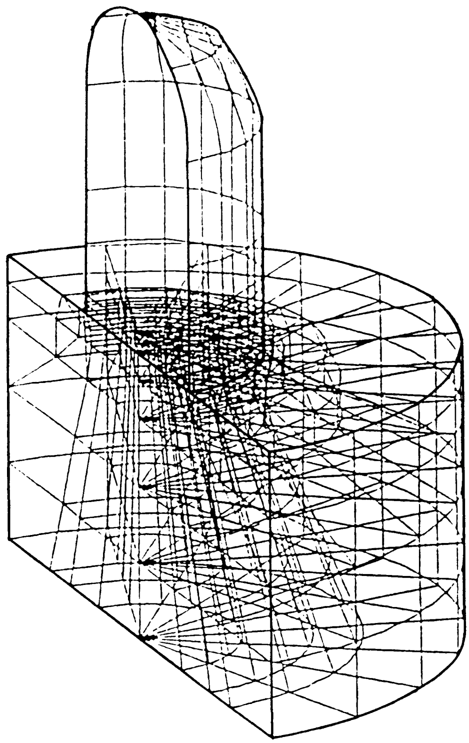

Fig 2. Global 3D model of base and soil

loading combination of DBF + SSE as was the reinforcement in the walls of the prestressing gallery and reactor cavity. This proved to be a significantly severe loading condition yielding a base reinforcement density of around 250 kg/m^3 of concrete rising to 650 kg/m^3 in some areas of the below base structures.

24. Non-linear analysis. In order to determine the ultimate load behaviour of the primary containment and the ultimate load factor on design pressure, a non-linear analysis (materially non-linear rather than geometrically non-linear) was carried out using the code ADINA-TW. The model included elements in both the walls and the base and also the prestressing gallery, although asymmetric features such as penetrations and prestressing buttresses were omitted. It should also be noted that ADINA-TW modelled not only the structural concrete but contained elements representing the principal reinforcement mats and prestressing. The containment liner was conservatively omitted from the analysis, despite being a significant strength member.

25. The analysis carried out indicated an ultimate structural condition determined by hoop tendon failure (the tendon having

reached its specified ultimate limit of 1% strain) at approximately mid height of the cylindrical shell wall. The 1% strain figure was reached at a pressure of 0.72 MPa giving a factor of 2.09 on design pressure.

26. Whilst the analysis contained 'probable' quantities of reinforcement in global areas, the results were also useful in establishing the build up of cracking in tensile zones and provided further justification for reinforcement disposition.

F) 1/10TH SCALE MODEL TEST

27. As mentioned in section D the UK Design and Construction rules require the design assessment to show that the containment can withstand an ultimate pressure of not less than 2 times the design pressure of 0.345 MPa (50 psig) at ambient temperature. As part of the validation process for ADINA-TW required by the Nuclear Installations Inspectorate, a 1/10th scale model of the Sizewell containment was built and tested to allow actual measurements to be compared with theoretical predictions for the model (refs 4 and 5).

28. The two principal objectives of the model were to : -

(a) Provide a means by which the ultimate analysis code being used on the full scale Sizewell 'B' structure could be validated in the linear and non-linear regions of structural response.

(b) Determine the ultimate structural capability of the model and its failure mode and establish the relevance of these to the full scale structure.

29. The principal dimensions of the model are shown in fig 3. The model had a flat base slab 0.42m thick and did not include the keyhole or stressing gallery projections; access for vertical tendon jacking was obtained by the base slab overhanging the supporting concrete plinth. Support conditions were thus not completely analogous to the full size structure.

30. The model base was thicker than the minimum scaled dimension from the full size structure in order to achieve similarity of rotational stiffness at the wall-base junction. However, this thickness did not fully compensate for the stiffening effects of the prestress gallery, reactor cavity and internal structures, all present on the full scale structure.

31. Structurally significant penetrations such as the equipment access hatch and the personnel airlocks and associated wall thickening were modelled.

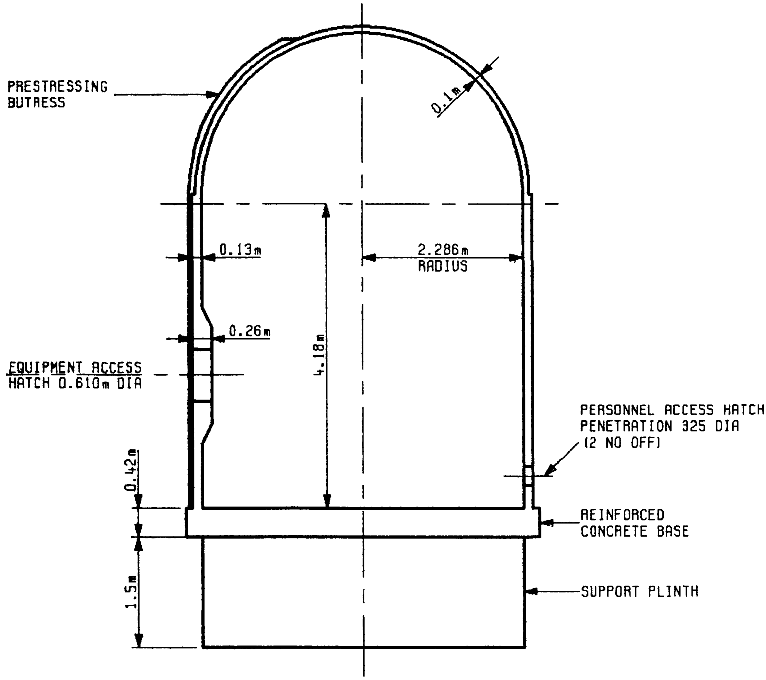

Fig 3. Section through 1/10th scale model

32. The 6mm thick steel liner which acts as the leak tight membrane in the full scale containment was not modelled. Pressurisation of the model was achieved instead by a water filled neoprene bag. Ancillary tests demonstrated the ability of the bag material to span crack widths in excess of those forecast.

33. A grade 45 microconcrete with maximum aggregate size 5mm was used throughout and the mix was designed to have compressive and tensile properties as near as possible to those of the full size containment. High yield reinforcement was used but one-for-one modelling was not practical; however scaled areas of rebar were provided (fig 4). Bars were joined by means of welding.

34. Prestress for the model was provided on a one-for-one simulation using 12.9mm and 8.0mm superstrands. Plastic sheathed greased strand was used which effectively allowed tendons to be fixed directly to the model without the use of preformed ducts (other than through the base). Values of friction coefficient were determined by ancillary tests.

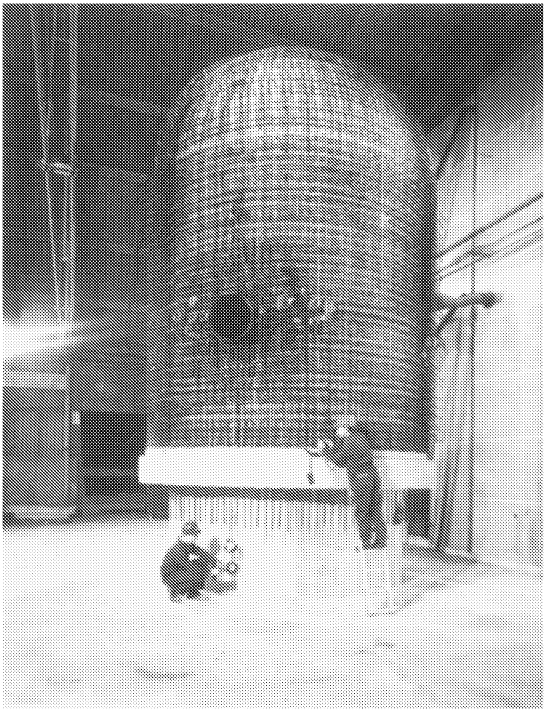

Fig 4. 1/10th scale model under construction

35. Tendons were anchored using standard barrel and wedge anchors bearing onto steel plates epoxy mounted on the surface of the model, and were shimmed after stressing to counteract the relative increase in wedge 'pull in' loss due to shorter tendons in the model than in the full size containment.

36. Instrumentation was provided on the model to allow its actual behaviour to be measured during the application of prestress and under pressurisation. Around 600 instruments were incorporated covering deflections, concrete and reinforcement strains, prestressing tendon loads and strains, and crack development.

37. The strain and deflection measurement instrumentation was generally arranged on three full cross sections and one half section through the containment.

38. The choice of model parameters, the design, and the instrumentation and test procedures were all subject to scrutiny by a Peer Review Panel.

39. The tests were carried out in July 1989 and the results can be summarised as follows.

(a) Elastic tests. At 1.15 times the design pressure there was no visible cracking of the barrel or the dome but hair line cracks were apparent at the edges of the base after the first elastic test. The measured uplift at the outer edge of the base was almost twice the predicted value. With subsequent elastic tests, base cracking extended vertically and horizontally from the bottom edge. Full elastic recovery was obtained after each elastic test.

(b) Ultimate test. The model failed at 2.43 times design pressure. At this pressure there were vertical cracks in the top half of the barrel, on either side of the buttresses. These cracks were consistent with radial expansion in the barrel and a number of the hoop tendons had reached the 1% strain failure criterion as defined by the D & C rules. Predicted results compared very well with measured values. The failure was progressive and ductile within the base mat and associated with yield of the lower mat of reinforcement and some spalling of concrete. After depressurisation and subsequent removal of the rubber bag the inner surfaces of the model, including the base, showed few signs of cracking. This is consistent with the fact that the bag remained intact with no escape of water.

40. From the results it is evident that the test satisfactorily achieved the two principal objectives.

G) LINER DESIGN

41. The liner design was substantiated by NNC (ref 6) with detailed design analysis carried out by the main liner contractor Cleveland Bridge & Engineering. The liner is designed for its principal duty of serving as a leak-tight membrane for all operating conditions and up to and including design basis fault. It also has a temporary duty of acting as an internal shutter to the concrete during wall and dome construction. There is no ultimate load requirement for the liner.

42. Under pressure loading the liner is supported by the backing concrete. The general design intent is to maintain the liner under uniform conditions of biaxial compression or tension. Stiffeners are incorporated connecting the liner to the concrete with the capability of transferring local plate forces.

43. The most significant loading imposed on the liner arises from the design basis fault where the liner is immediately subjected to a temperature rise of 200°C, whereas the

temperature rise of the backing concrete occurs much later. In this condition a maximum compressive strain of 3,200µS is induced which on cooldown can generate a residual tensile strain of 1,500µS. The liner material is therefore specified to have a tensile strain limit of twice this value, viz 3,000µS.

44. The arrangements for the liner tie or stiffening systems vary with location in the containment. In the reinforced concrete base the liner plates are welded to the top flanges of a steel grillage embedded in the top of the structural concrete. In the cylindrical barrel of the containment, vertical angle stiffeners are welded to the back of the liner at 345mm centres with additional stiffening rings outside these and also furnished with stud connectors in certain locations. In the dome, stiffening is provided by an orthogonal grid of tee and angle sections.

45. The maintenance of the generally biaxial stress conditions in the liner membrane ensures little or no shear on the anchors to the main body of the liner. At corners or discontinuities the anchorage system has been enhanced to transfer the plate loads into the concrete. To minimise loads a thin 6mm plate was selected consistent with relative ease of fabrication, together with a low yield mild steel (325 MN/m^2 specified for Sizewell). Forces arising from potential buckling of the plate are restrained by the anchorage system, the performance of which has been determined empirically from full scale testing, which provided the load-deflection characteristics of the multiple anchor systems. Fatigue of the liner is not considered as a design problem, apart from areas local to discontinuities, since the major loads experienced by the liner are of low probability and of a non-cyclic nature.

H) CONSTRUCTION APPROACH AND PROGRESS

46. Construction of the Sizewell 'B' containment is being undertaken at site by John Laing Construction Limited, the main civil works contractor. The sequence of principal construction activities is summarised in table 3 together with the already achieved start dates and durations. Following excavation to formation level in February 1988, the mass concrete base and structural concrete have followed as successive stages. The major items from the completed activities are summarised below

<u>Mass concrete.</u> It was decided at an early stage to excavate below the containment base to a single common level below the underside of the reactor cavity. Mass concrete (without reinforcement) was then placed up to the level of the underside of the main structural base with major boxouts provided for the reactor cavity/instrumentation tunnel and the prestressing

gallery. Whilst no account was taken in the design of the structural strength of the mass concrete, its inertial contribution to structural stability under seismic loading was included. The walls of the reactor cavity/instrumentation tunnel and the prestressing gallery were then constructed using the mass concrete as a back shutter.

Table 2. Sequence of principal construction activities

No	Item	Start Date	Complete Date
(a)	Completed activities		
1	Mass Fill Concrete	Feb 88	Jun 88
2	Construct Reactor Instrumentation Tunnel (first permanent structural concrete)	Jun 88	Nov 88
3	Construct Base Slab	Oct 88	May 89
4	Weld Liner Base	Jun 89	Nov 89
5	Liner Topping Concrete Base	Nov 89	May 90
(b)	Under Construction		
6	Weld Liner Walls to +47.70 top of tier 15	Jun 89	
7	Construct Containment Walls to +46.40 top of lift 15	Jan 90	
(c)	Future Activities		
8	Construct Polar Crane Support Corbel		
9	Erect Polar Crane		
10	Erect & Weld Dome Liner		
11	Construct Dome (including lift 16)		
12	Weld & Construct Infill Section Construction Access		
13	Install Tendons		
14	Pre-stress Containment		
15	Structural Over-Pressure Test & Integrated Leak Rate Test		

Main base structural concrete. The original intent had been to cast the whole base (7,500 m^3) as a single, continuous pour. However, analytical investigation showed that the thermal movement of such a large pour might cause unacceptably high stresses at the junction of the base with the prestressing gallery and instrumentation tunnel. Further investigation led to the concept of an initial disc pour some 0.8m deep which would generate lower thermal movement, but which could relieve residual thermal stress with controlled cracking. The main 3.0m deep pour for the base then followed some 28 days later, and subsequent examination of this pour and the below base structures showed this to be an entirely satisfactory construction method.

Liner erection.

(a) Base:- The liner grillage members comprise joist sections arranged to allow subsequent installation of liner plate in suitably sized panels. The grillage was carried by special structural steelwork supports erected on top of the 0.8m concrete disc. During casting of the remaining 3m deep pour, concrete was taken to a level some halfway up the grillage webs, so enveloping the bottom flange. When the main concrete had hardened, but was still 'green', the final topping layer was placed level with the top flange of the grillage. The liner base plate panels were then welded to the top of the flanges incorporating, at the same time, the leak detection channels.

(b) Barrel:- The wall liner (fig 5) was built from 15 tiers of panels each generally 2.8m high with 16 panels per tier; each panel was about 9m long.

Vertical joints between the tiers were staggered by half a panel, and 15mm of trim was provided on top of tiers 6, 10 and 15 which could be used to whatever extent necessary in order to maintain levels within tolerance as the build proceeded.

The bottom of the wall was welded to the floor grillage and subsequently to the floor plates.

Tier 14 included an enlarged section of 25mm thick plate for the concrete corbel supports for the polar crane rail and to accommodate the through liner couplers.

Stability of the wall liner during construction was maintained by a system of temporary wind girders, (design wind velocity 42m/sec).

During construction of the wall, a temporary opening for construction access has been left (tiers 1-6). This was formed in three stages to provide an access approximately 9m wide by 15.7m high. After basic completion of the liner, a number of areas were removed to be replaced by prefabricated shop-built panels complete with embedments.

Embedments were generally set in 12mm or 25mm thick plate, as were tiers 13 and 14 incorporating the polar crane rail support brackets. Where 12mm thick embedments were to be incorporated in 6mm thick panels during works fabrication, this

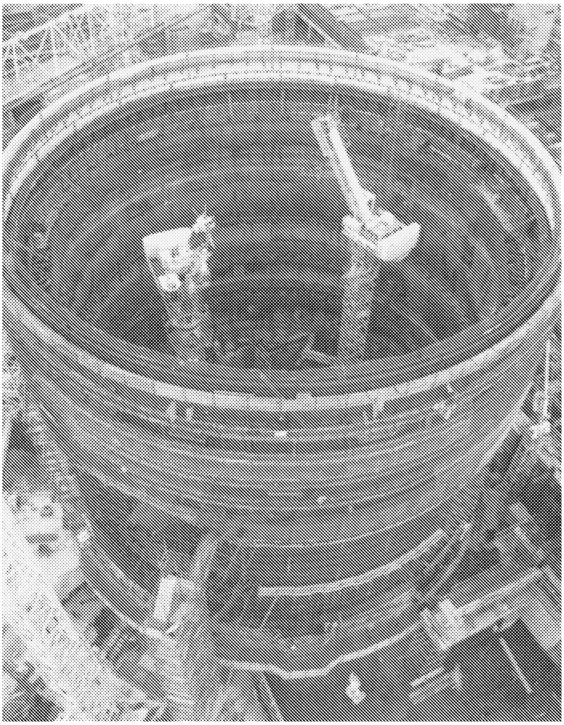

Fig 5. Wall liner erected to spring line.

was achieved by machining 12mm plate down to 6mm in order to avoid distortions which could arise from welding to thin plate.

Additional internal strongbacks were installed where there was discontinuity of external stiffening due to embedments etc., in order to provide the necessary strength to resist the hydrostatic pressure of concreting.

(c) Dome liner:- This is fabricated in three 'onion rings' on a hardstanding area (fig 6) adjacent to the reactor building. Each ring will then be lifted into position and welded prior to completion of the dome concrete.

Wall concreting. In general a maximum lift height of 2.8m was selected taking into account the ability of the liner, which forms the inner shutter, to withstand temporary wet concrete loadings.

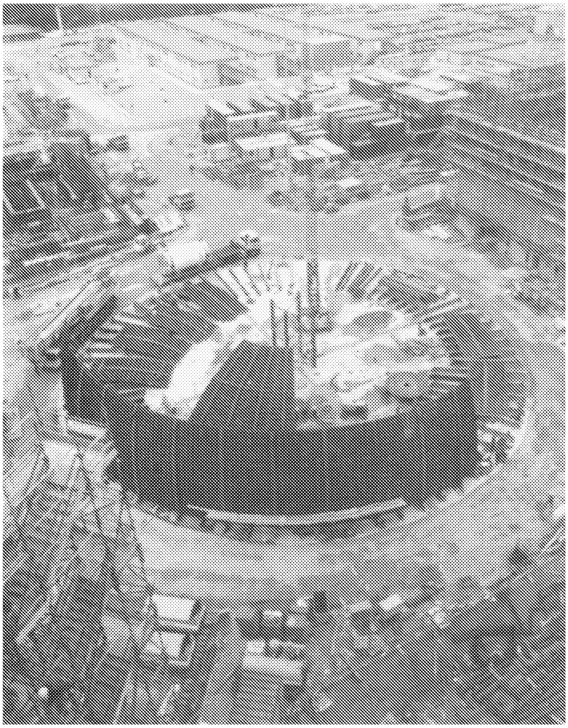

Fig 6. Dome liner under fabrication

During construction of the wall, a large temporary opening is provided for construction access through the liner, and will be sealed before prestressing the containment. The presence of this opening, together with multiple penetrations at lower levels of the walls and density of reinforcement has led to concreting proceeding in five sectors to the top of the access opening. Above this level, concrete will be placed either in complete rings or as two approximately semicircular segments to the springline of the dome, flying past the secondary enclosure support cantilevers, which will be cast later, making use of coupled reinforcement for continuity.

I) CONCLUDING REMARKS

47. Although the PWR is new in the UK, the primary containment has been under design and development for the past ten years. The concept is based upon design and construction of reactor containments which have been operating successfully in the USA for a number of years.

48. It is considered that the development, design and construction processes to which the primary containment of the first of the UK PWR power stations have been subjected will ensure a fully safe and reliable structure.

J) ACKNOWLEDGEMENTS

49. This paper is published with the permission of Nuclear Electric plc.

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13. Stainless steel containment linings for nuclear processing facilities

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SYNOPSIS BNFL, as part of the Nuclear Fuel cycle, have a requirement to store large quantities of material which require cooling. Where the coolant demand is high, BNFL have used large storage ponds where water is allowed to circulate around the material. It can be seen then that there is a requirement for fuel storage ponds to demonstrate high degrees of containment. Two independent containment systems are used to prevent any escape to the environment.

CONTAINMENT SYSTEMS

1. The first or primary containment can be either water retaining concrete painted with a decontaminable material or alternatively where different operational criteria apply, a stainless steel lining. The subject of this paper. (See Fig.1.)

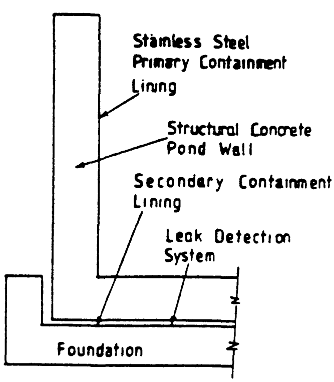

FIG 1. POND WITH PRIMARY AND SECONDARY CONTAINMENT LINING

2. The secondary containment consists of a carbon steel lining protected with a plastic anticorrosion layer. This can form part of a leakage detection system. This system is described more fully and in the context of the structural system in Ref 1.

LINING SYSTEMS

3. BNFL Engineering have developed a stainless steel primary containment lining system for use in cooling ponds. The design and analysis of the system is based on research and testing carried out by BNFL to establish the behaviour of the system.

4. It is important that the lining is not stressed beyond predetermined allowable limits during normal operating and extreme hazard conditions. The extreme hazard conditions identified by BNFL Engineering are cooling system failure and earthquakes.

5. The allowable limits for the containment system have been adopted from the ASME pressure vessel code Ref 2. The design and analysis of the lining and anchorage system is project specific the loading regime for each case is identified and applied to the system to determine its integrity.

ANCHORAGE GRILLAGE AND LINING DETAILS FOR STAINLESS STEEL PRIMARY CONTAINMENT

6. An anchorage grillage is cast into the walls of the pond or into a Structural screed anchored to the pond base. The grillage is anchored into the concrete by a system of reinforcing bars. The anchorage members are generally at 1.255m centres in both directions except around encast plates and other obstructions where other details are employed. The grillage is welded into a continuous frame by means of butt welds or, where access is restricted fishplates. During casting of the Walls the grillage is held in position by bolting to the shutters. The base grillage is held in position by fixing it to stools which are bolted to the base concrete. When the concrete is cast and the structural screed placed, the top surface of the anchorage is left flush with the concrete surface. (See Fig. 2.)

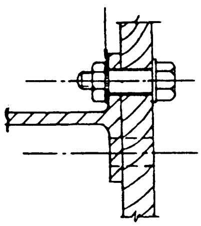

FIG. 2. WALL ANCHORAGE SUPPORT DETAIL

7. <u>Lining Details</u> The lining consists of 3mm thick type 304L stainless steel sheet. The sheets are cut to size, edges prepared for welding then tacked to the grillages leaving a gap which coincides with the centres of the anchorage grillage. The lining is completed by filling the gap with weld material which fuses the sheets together and welds the sheets to the anchorage. (See Fig. 3.)

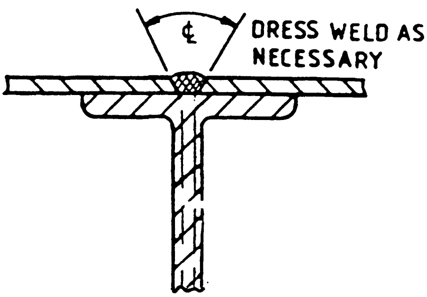

FIG 3. LINING TO ANCHORAGE DETAIL

8. At internal corners the lining sheets are folded with a 50mm internal radius. These folded sections are welded to the anchorage adjacent to the corners. The folded sections are joined to each other by using a backing flat which is welded at its ends to the encast anchorage and is located in a groove formed in the concrete. (See Fig. 4.)

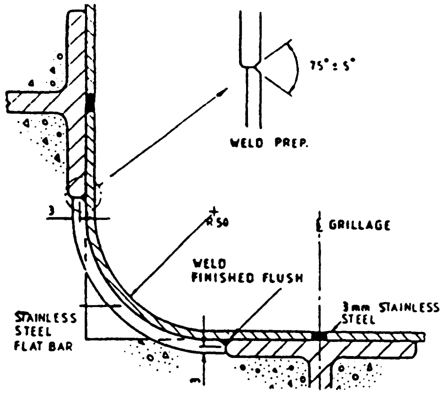

FIG 4. LINING AND ANCHORAGE CORNER DETAIL

9. Where the three internal corners meet a prefabricated corner section is used. (See Fig. 5.)

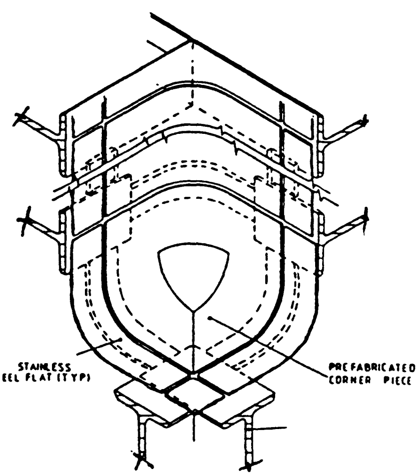

FIG 5. LINING AND ANCHORAGE DETAIL AT THREE INTERNAL CORNERS

10. The air gap formed behind the lining is vital as it allows the lining to deflect freely in the membrane plane and allows strain relief to take place.

LOAD BEARING ENCAST WALLPLATES DETAILS

11. No connections or attachments are made to the lining sheets. Any item requiring support is carried by encast wallplates.

12. A wall plate consists of a stainless steel face plate with anchorge sections welded to the back of it. The face plate is sized such that connections can be made to it as required to suite the dimensions of the item that is required to be supported from it, together with an allowance for construction tolerances. The faceplate thickness is designed to suite local stresses incurred in transferring the forces from connections to the anchorages. (See Fig. 6.)

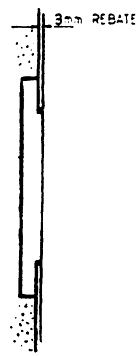

FIG 6. ENCAST WALL PLATE DETAIL

13. The anchorages are designed to safely transfer the loading forces to the surrounding concrete structure. The design principles for the anchorages are in accordance with those described in Ref 4.

ANALYSIS

14. In order to devise a stainless steel lining system for storage ponds it is necessary to define the behavioural characteristics of what is essentially a concrete box lined internally with stainless steel. To define these characteristics it will be assumed that the lining is integral with the concrete surface.

15. Thermal Incompatibility Stainless steel has a higher thermal expansion coefficient when compared to either normal carbon steels or concrete. The expansion coefficient for stainless steel is approximately 1.6×10^{-5} strain/^{o}C whereas both carbon steel and concrete are typically 1.2×10^{5} strain/^{o}C. Any thermal changes in the concrete will result in differential strain inducing loads in the stainless steel.

16. Flexural Eccentricity (out of plane behaviour) Because the stainless steel lining is essentially at the extreme fibre of the structural elements, strains induced in the liner will be greater even than the inner reinforcement. Bearing in mind that the reinforcement is to be optimised in the design process, generally in BNF to ACI 318 (Ref 3) limits, the strains in the liner will exhibit values that will be unacceptable because the stainless steel is less robust than the reinforcement steel.

17. In Plane Behaviour Tensile or compressure strains from mechanical loading affect both the concrete and liner equally and generally is not a significant characteristic.

18. Performance Requirements The concrete structures is not considered to be part of the containment system, but has to remain intact during the prescribed loading conditions, hence the use of ACI 318 in the design process. However, due to the containment requirement of the lining, the performance limits for the lining are defined as those used in ASME IV Div 2 which are more restrictive than ACI 318.

19. Discussion From the description of the behaviour of a lining system that is integral with the concrete superstructure, it can be seen that the lining will undergo severe distress when compared to the limits set by ASME III. To prevent this some form of relaxation anchor system must be devised where the loads from the concrete superstructure cannot pass to the lining system. BNFL have investigated, by testing many configurations, a series of anchor systems that are mechanically robust and yet exhibit highly ductile behaviour. This means that the lining and concrete structure will in part act independently of each other. A delicate balance must be struck between having a system which has ductility and low

strength and its converse with rigidty and high strength. A low strength system will work in ponds of a uniform nature with no cast-in items but the introduction of cast in items will yield high local stresses in the liner which can only be relieved by the introduction of more rigid attachments elsewhere.

TESTING

19. Approximately one hundred and thirty tests were carried out at Lancaster University to determine the in plane physical characteristics of many anchorage systems some of which were very rigid where others highly ductile. Figure 7 shows a typical load/deflection curve. The tests involved anchorage test pieces cast into concrete where upon loads were applied to arrive at their load/deflection characteristics. This was an evolutionary process in order to achieve the desired characteristics the results of each test being used to decide subsequent configurations. Each test was carried out five times to ensure consistency of results. In addition to the anchorages, the stainless steel used in the lining was also tested to give its non-linear stress characteristics which were used in the analysis (See Fig. 8.)

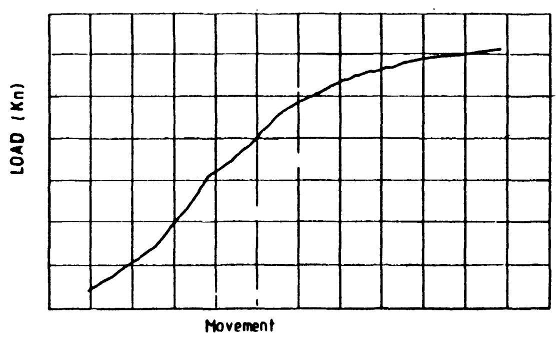

FIG 7. TYPICAL LOAD / DEFLECTION CURVE FOR ANCHORAGE

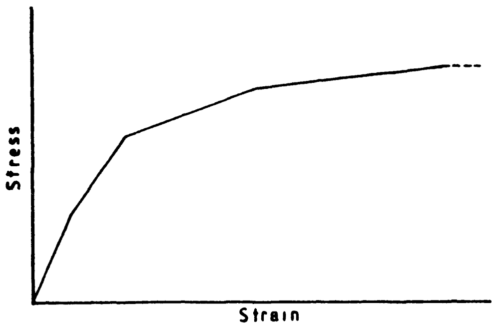

FIG 8. NON LINEAR STRESS / STRAIN CURVE FOR STAINLESS STEEL

THE ADOPTED SYSTEM

20. The anchorage details have been chosen from the configurations tested to give a high degree of mechanical strength and ductility. To prove the adequacy of this system, for particular applications, computer analysis of the configuration is carried out. Areas of concern are as follows:

21. General floor and wall areas Models used for general areas are made up of membrane elements with beam elements representing the anchorage section and springs representing the load/deflection characteristics of the system. All materials are considered non-linear - the materials being stainless steel as are the load deflection curves. (See Fig. 9.)

22. Areas with cast in items The introduction of cast in items tend to cause stress concentrations at or near the cast in item in the liner (See Fig. 10.) The cast in item will be designed to carry the loads applied to it from the liner in addition to operational loads.

23. Internal/external corner edge details Edge details are required to act as "soft corners" ie they will provide no mechanical resistance to in plane loads from the liner (See Fig. 10.) analysis are carried out to ensure the bending stresses are within acceptable limits. The displacements at the edges of the general areas are imposed as boundary conditions as are thermal and hydrostatic loading if applicable.

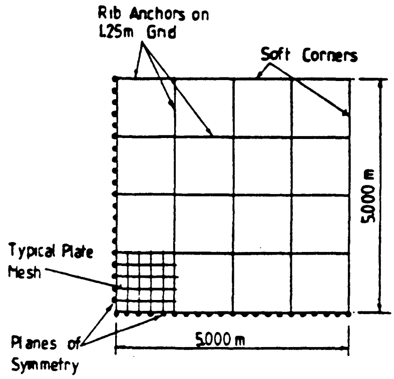

FIG 9. WALL / FLOOR PLATE MODEL

24. Internal 3D corner details The desired behaviour of an internal 3D corner is the same as that of edge details but this case is geometrically stiffer, the same loading conditions are applied here as for the edge details. (See Fig. 10.)

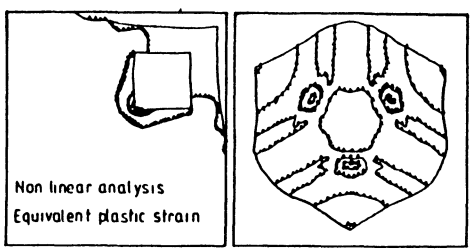

FIG 10. EDGE DETAILS

25. Any other unusual interpenetration detail Any detail that is different from the above is reviewed and analysed as required with as near the correct loading conditions as can be made.

26. General Floor and Wall Areas: Detail to give an appreciation of the methods adopted, a description of the floor areas as an example will be attempted. Similar approaches are used for the other details.

27. A floor area is considered symmetrical about two orthogonal axes to enable only one quarter of the floor area to be modelled mathematically. A

general grid is chosen to represent the cast in anchorages to which the plate is welded. A more detailed grid is overlaid to represent both plate and anchorage stiffness. The anchor springs are connected at each grid point on the liner and grounded to a fixed point, ie the concrete is assumed non deformable. The springs are represented in two directions to give the transverse and longitudinal stiffness of the anchorage.

28. Loading is represented by temperature rises applied to the liner surface either as a variable temperature over the surface or as a uniform temperature depending on loading and conservatism required. All mechanical loads are converted to temperatures, they may represent loading due to static, thermal or dynamic loading and their combinations.

DESIGN LIMIT

29. The design limit set for the liner and anchorage system is taken from ASME III Div 2. This code defines the limit for an anchorage system to be:

30. Normal operating conditions : ¼ of maximum deflection.
Abnormal operating conditions : ½ of maximum deflection. (See Fig. 11. for typical results).

31. The maximum deflection is determined from the tests as described previously.

32. The limits defined for the liner are given in microstrain for each of the above loading conditions for tensile and compressive conditions. The limits vary between 3000 and 5000 microstrain.

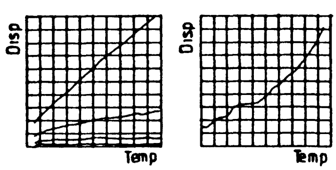

FIG. 11 TYPICAL TEMPERATURE / DISPLACEMENT RESULTS

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14. Design and analyses of the reinforced concrete support corbels for the polar crane in the reactor containment building of Sizewell 'B' power station

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SYNOPSIS. The polar crane within the primary containment building at Sizewell 'B' Nuclear Power Station will be supported by a ring of forty-eight individual reinforced concrete corbel units, separated from each other by 20mm gaps, and anchored, by reinforcement, into the containment building wall. After initial design work undertaken to determine the amount and layout of reinforcement required in each unit, a section of wall and corbel was analysed to assess both stresses in the corbel units and the effect on the containment wall.

INTRODUCTION

1. This paper describes the design and analyses of the reinforced concrete corbels which support the polar crane within the primary containment building of the pressurised water reactor nuclear power station currently under construction at Sizewell in Suffolk.

2. The paper firstly describes the corbel units and the practical considerations and basic principles observed in undertaking their sizing design and determination of their reinforcement requirements. It then describes the analyses carried out to assess both the design of the corbel and its effect on the containment building. The first set of analyses modelled a complete corbel unit in three dimensions to investigate its overall behaviour in response to crane wheel loads. The second set, also three dimensional, modelled one sixth of the length of a corbel unit, with an equivalent short length of containment wall above and below, to assess the stresses in the concrete and main reinforcing bars and to provide the liner designer with strain and displacement information for use in substantiating the design of the local liner and its attachments.

3. The paper finally presents results and conclusions, including those from a full size mock-up of a corbel unit which was commissioned to verify the practicability of erecting the corbel shutter and fitting the reinforcement.

DESCRIPTION OF CORBEL AND PRACTICAL CONSIDERATIONS

4. The corbel is a near continuous reinforced concrete element comprising forty-eight individual units, each approximately 3m long and separated from each other by 20mm gaps. With reference to Figs. 1 and 2 the key features of the corbel and its relationship with the containment building can be summarised as follows:

(a) Each corbel unit is 2m deep and approximately 1.5m wide, and is recessed into the containment wall by 228mm.
(b) The tolerance on the liner radius is $\pm$75mm and that on the crane buffer stop plate radius is $\pm$5mm. Thus, the width of the corbel over the top 400mm depth can vary by up to 160mm and the reinforcement within the corbel had to be designed accordingly.
(c) The tolerance on the liner radius could also result in an increase, from the nominal dimension, in the eccentricity of the crane vertical loads from the face of the containment wall. The design and analyses allow for the maximum eccentricity.
(d) The main reinforcement in the corbel is connected to that in the containment wall by through-liner couplers welded into a 25mm thick coupler plate located between the corbel and the containment wall. To prevent the couplers taking shear loading, for which they are not qualified, they will be wrapped in soft material for a distance of approximately 240mm on either side of the coupler plate.
(e) The top-most reinforcing bars in the corbel take the form of long coupler sleeves anchored at the end remote from the containment by a stud and anchor plate arrangement. The long sleeve detail is intended to minimise on-site work and the anchor plate detail is necessary due to the difficulty in achieving adequate bond length for the top bars.
(f) The corbel is supported from the containment building at the bottom ledge, the top surface of which is lined with a 25mm thick steel plate and set at an angle of 27^{o} to the horizontal. This ensures that the resultant load from the crane is generally at 90^{o} to the ledge and is also intended to prevent the formation of voids in the concrete below the ledge, as this part of the containment is cast.
(g) The top ledge is also lined with a 25mm thick plate, set at an angle of 11.65^{o} to the horizontal.
(h) Whilst the primary load path from the corbel into the containment shell is via the bottom ledge, horizontal shear keys provided on both surfaces of the coupler plate are themselves capable of transferring the corbel vertical loads. They therefore provide redundancy to the bottom ledge.

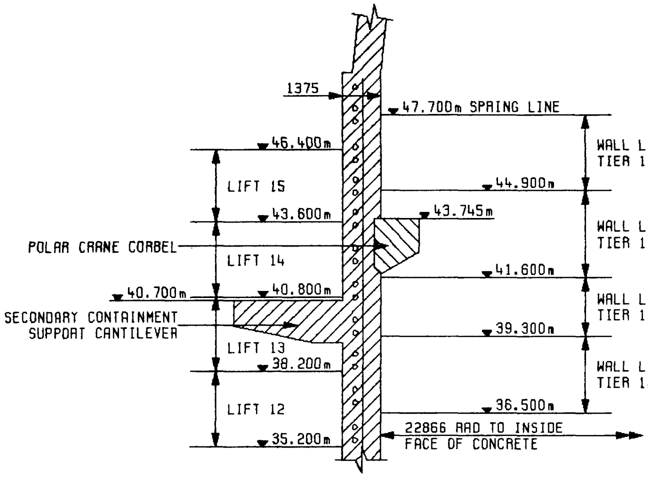

Fig. 1. Location of corbel on containment wall

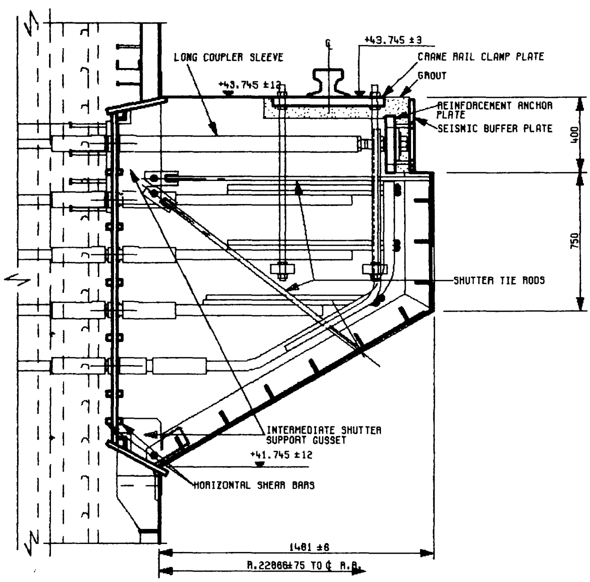

Fig. 2. Key features and dimensions of corbel

(i) Tangential loads are transmitted from the corbel to the coupler plate by vertical shear plates located in the corbel/containment wall recess. The tangential loads are then transmitted into the containment shell by a number of vertical shear bars welded to the containment wall side of the coupler plate.

5. Each corbel unit will be formed using a steel shutter comprising sloping soffit, front and side plates supported from a frame which is connected to the vertical shear plates and intermediate gussets on the coupler plate by an arrangement of bolts and removable ties.

BASIC DESIGN OF CORBEL UNITS

6. The corbel has been designed in accordance with Ref. 1 with further reference to Refs. 2 and 3.

7. The key loads for which the corbel has been designed are the crane wheel loads defined in Table 1.

Table 1. Summary of maximum crane wheel loads

	LOADING CONDITION	VERTICAL Ww (kN)	RADIAL R (kN)	TANGENTIAL T (kN)
1	Out of service	632	-	-
2	Construction	1304	93	65
3	Overload test	1396	99	70
4	High integrity operation	987	70	49
5	SSE	1302	691	477
6	Fault/snag load	1679	0	0

8. Crane wheel vertical loads are transmitted in bearing from the crane rail base plate into the corbel concrete. Radial loads from the crane vertical wheels and guide rollers are transmitted from the crane rail base plate into the concrete by shear friction made effective by preload in the rail base plate holding down bolts. Radial loads from the seismic buffer would be transmitted into the concrete by bearing of the buffer plate and reinforcement anchor plate and straight through into the containment wall. Tangential loads from the crane wheels are transmitted from the crane rail base plate into the concrete by shear friction.

9. In so far as vertical and radial loads are concerned, the corbel is designed as a strut and tie, as explained in Ref. 3. The top reinforcement is assumed to form the tie and the body of the corbel, the strut, which is assumed to transmit its load to the containment building wall via the sloping ledge at the bottom of the corbel.

10. Torsion of the corbel due to both eccentric vertical loading and tangential loading is assumed to be resisted by the keying of the corbel into the containment wall.

11. The corbel is reinforced in accordance with the requirements of Ref. 1. The main reinforcement is in accordance with recommendations of Refs. 2 and 3 and minimum reinforcement requirements are observed for the side faces of each corbel unit on the basis that these are "exposed" faces.

12. Additional reinforcement, to resist bursting and spalling, is provided in areas of local high stress, such as at the bearing surface of the corbel at the bottom ledge and at the rail base plate anchor points.

ANALYTICAL APPROACH

13. Two separate sets of analyses were undertaken.

Preliminary 3-D analyses

14. 3-D analyses of a complete corbel unit were undertaken to identify its behaviour, key characteristics, and the likely worst loading. These analyses considered the corbel only, with appropriate boundary conditions for the containment wall.

Composite 3-D analyses

15. 3-D analyses of a 1.25^{o} circumferential slice of corbel and containment wall were undertaken, incorporating the liner and significant steel components.

Material properties

16. The material properties assumed were the same as those used in the analyses of the containment building. (Ref. 4.)

Loading

17. The loads used in each analysis are described in respective later sections of this paper but a brief description of the basic loadings is given here.

18. Polar crane loads. The polar crane wheel loads were as given in Table 1. The arrangement of the crane wheels is such that it is possible for two wheels to be supported by a single corbel unit.

19. Prestress loads. The prestress loads were the same as those used for the design of the containment building. (Ref. 4.)

20. Dead weight. For those parts of the structure modelled in the analyses, gravity loads were generated automatically by the program. For those parts of the structure not modelled in the analyses, but which need to be considered as applying load to the model the dead weights were calculated and applied as appropriate.

21. Temperature loading. Temperature loading information was provided by the liner designer.

22. Shrinkage. Shrinkage of up to 400 microstrain was applied in the analyses undertaken to estimate strains.

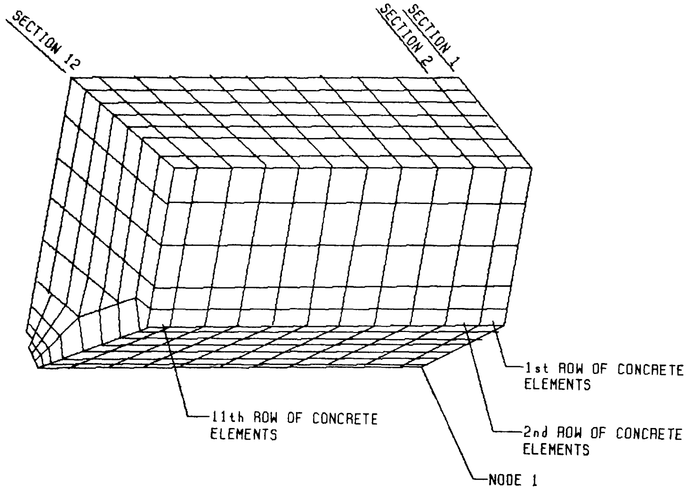

Fig. 3. Finite element grid for preliminary 3-D analysis of corbel unit

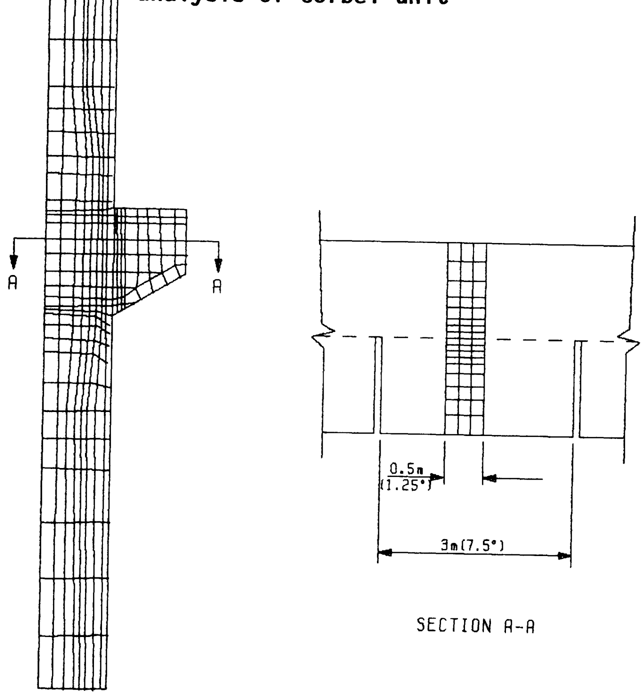

Fig. 4. Vertical and horizontal sections showing finite element grid for 3-D composite analysis of corbel and containment wall

PRELIMINARY 3-D ANALYSES OF CORBEL

23. These analyses were carried out to investigate the mode of behaviour of the corbel, to estimate forces and movements at the through-liner couplers and to identify the worst loading condition for a corbel unit. The analyses also investigated the effects of curvature of the corbel, particularly with respect to bearing at the bottom ledge, and variations in the top ledge boundary conditions. Input information for the 3-D composite analyses was also obtained.

24. A 3-D finite element model of a corbel unit, as shown in Fig. 3, was developed to include:

(a) concrete of the corbel, modelled as brick elements.
(b) reinforcement modelled, as beam elements, the main reinforcement being modelled in such a way as to represent its debonding from the concrete for a length of approximately 240mm on either side of the coupler plate.
(c) provision for gaps to open between the corbel and the wall, should the loading and the response of the structure so dictate, with no friction forces being allowed to be transmitted across closed gaps.

25. External restraint is imposed at the containment wall ends of the reinforcing bars. Additional varying restraints occur due to closures of the gap elements which allow transmission of compressive stresses in the direction perpendicular to the surface but do not allow transmission of tensile stresses.

26. Three runs were carried out using this preliminary model, investigating the effects of various restraint, geometric and loading conditions as detailed in Table 2.

Table 2. Summary of loading and restraints

RUN	CURVED IN PLAN	GAP ELEMENTS			HOOP RESTR.	CASE	LOAD (kN)		
		TOP OF RECESS	BACK OF RECESS	BOTTOM OF RECESS			TANG.	RAD.	VERT.
1	YES	YES	YES	YES	YES	1	+140	-99	-1396
						2	+140	-198	-2792
2	YES	NO	YES	YES	YES	1	+140	-99	-1396
						2	+140	-198	-2792
3	NO	NO	YES	YES	YES	1	+140	-99	-1396
						2	+140	-198	-2792

27. The analyses gave the following conclusions:

(a) On the basis of load transmitted to the bottom bearing ledge, the most severe loading condition is that in which a corbel unit supports two wheels, one at an end and the other 300mm off centre remote from that end. In that configuration, the loaded end of the corbel unit attracts approximately 40% more load than would be obtained by considering the two wheel loads uniformly spread over the corbel unit.

(b) The single wheel load, whilst causing a more extreme variation in load along the bottom ledge, gives a maximum value approximately equal to that obtained by considering two wheel loads uniformly spread over the corbel unit.

(c) Comparison of the results from the various runs showed that satisfactory behaviour of the corbel does not require the maintenance of contact between the corbel concrete and the top ledge, and that curvature of the corbel has an insignificant effect on its overall behaviour.

28. The 40% enhancement factor, noted above, due to the eccentric wheel loading, was used to derive appropriate input loads for the 3-D composite analyses.

3-D COMPOSITE ANALYSES OF CORBEL AND CONTAINMENT WALL

29. These analyses were undertaken to estimate stresses in the corbel and local containment wall and to derive strains in the various steel items in the immediate vicinity. The analyses were also used to assess the range of load in the corbel top reinforcing bars in order to specify the requirements for the cyclic loading tests on the through-liner couplers.

Analytical approach

30. The approach was to analyse a 1.25^{o} slice of the corbel and local containment wall incorporating the coupler plate, liner and attachments. The analytical model comprised three groups of elements in the hoop direction. The basic element was the 8-noded brick type element and nodes were therefore provided on 4 radial vertical planes.

31. Basic details of the analytical grid are shown in Fig. 4.

32. For the construction load cases the model extended up to the top of lift 15, where free movement of the wall was permitted.

33. For both the partially completed and completed structure, a section of the wall was modelled from +35m O.D., where it was restrained vertically, up to approximately the spring line, the corbel being included at the appropriate level.

34. For the complete structure, the dome stiffening effect was simulated by the application of an analytically derived moment at the top of the wall to keep the top surface horizontal.

Loading

35. Loading was applied to the model as follows:

36. Vertical prestress. Vertical prestress load was applied as a uniform pressure to the top of the model.

37. Hoop prestress. Hoop prestress loading was applied as an external pressure to the outer surface of the wall.

38. Primary containment dome load. For cases where the full containment shell was considered, the dome weight was applied as a uniform axial pressure to the top surface of the containment wall, as modelled in the analyses.

39. Secondary containment building loads. The dead weight of the secondary containment dome and the support cantilever was applied as a vertical load together with a radial load representing the moment at the location of the cantilever to shell connection.

Analytical modelling

40. The key elements modelled in the analyses were as follows:

41. Wall. This was modelled as plain concrete, except for the bars passing into it from the corbel.

42. Corbel. This was modelled as a 1.25^{o} slice with appropriate allowance for its internal features.

43. Corbel reinforcement. All the corbel main bars were modelled and the radial bars extended into the containment wall. Debonding of the bars close to the coupler plate was simulated, and the area and stiffness for each row was adjusted to represent the correct total values for a 1.25^{o} slice.

44. Leak chases. These were represented by removing the appropriate rows of elements from the top and bottom of the model.

45. Liner/Coupler Plate. These were modelled, as appropriate, over the complete model.

46. Gussets. These were modelled on the two end sections.

47. Vertical angles. These were modelled on two sections, the plate elements being thickened to give the required total area.

48. Header plates. These were modelled at two levels.

49. Vertical shear bars. These were simulated by repeating the tangential degree of freedom for the wall and coupler plate nodes on the appropriate vertical sections.

50. Horizontal shear bars. These were simulated in a manner similar to the vertical bars. They were not, however, modelled in the analyses performed to provide strain information to the liner designer.

51. Interface gaps between the corbel and the containment wall were modelled as a complete separation medium and in order to investigate the effects of the presence or absence of transverse load transmission

capability at the top ledge, two sets of analyses were carried out, one with the capability and one without, described as "no-gap" and "with-gap" respectively.

52. Prior to carrying out the main 3-D analyses, a horizontal 2-D analysis was undertaken to assess the hoop stiffening effect of the corbel on the containment wall. This showed that to allow for this stiffening effect, a 0.27m width of corbel, adjacent to the coupler plate, should be considered as having hoop continuity.

Method of analyses

53. The analyses were carried out by running a series of unit load cases and then combining appropriately factored results from these to give results for the required loading conditions.

54. Each unit load case was run using a common "E" value of 35000N/sq.mm for "with-gap" and "no-gap" configurations.

55. The load combinations considered were as follows:

(a) A. Crane installed, incomplete structure, no dome:
- A1: Crane overload test (L.C. 3 - normal)
- A2: Fault/Snag load (L.C. 6 - exceptional)

(b) B. Prestressed complete structure, before start up:
- B1: Crane overload test (L.C. 3 - normal)
- B2: Fault/Snag load (L.C. 6 - exceptional)

(c) C. Normal operation (Normal temperature effects)
- C1: High Integrity Operation (L.C. 4 - normal)
- C2: Safe Shutdown Earthquake (L.C. 5 - exceptional)

(d) D. Design Basis Fault (e.g. LOCA or Loss of AC power)
- D1: Crane out of service (L.C. 1 - normal)
- D2: Safe Shutdown Earthquake (L.C. 5 - exceptional)

56. Load combinations A and B relate to construction load cases, C and D to reactor operational cases.

57. Concrete shrinkage and creep have only a secondary effect on the values and distribution of concrete stresses, and therefore the factors applied to unit load cases to obtain concrete stresses were based solely on applied loading considerations. However, these parameters are of primary importance in obtaining correct values of liner strains, and therefore concrete shrinkage effects were taken into account in the analyses used to derive liner strains and the factors applied to the unit load cases included a component related to concrete stiffness.

RESULTS OF THE ANALYSES

58. From a civil engineering standpoint, the analyses results were examined for three main aspects, namely, the forces in the main reinforcing bars, the effect of the corbel on the stresses in the containment wall, and the

stresses in the corbel itself. In a short paper such as this it is impossible to present a full and detailed account of all of these aspects but the main points emerging are discussed below.

Forces in the main reinforcing bars

59. The forces in the main reinforcing bars were examined for two reasons, firstly to verify that, at the location of the through-liner couplers, the bars are not carrying any shear load and, secondly, to determine the range of load to be adopted for the cyclic load testing to be carried out on the couplers.

60. The analytical results did indeed confirm that at the through-liner coupler positions the shear stresses in the bars were very small, thus supporting the intention to wrap the couplers in soft material.

61. The results also indicated the range of load in the main bars, thus enabling decisions to be made in respect of cyclic load testing of the couplers.

Effect of corbel on the containment wall

62. The compressive stresses in the containment wall, indicated by the analyses, are in nearly all cases less than permitted field stress values. Where this is not the case, the predicted stresses satisfy the acceptability criteria for triaxial stress as defined in BS 4975.

63. Some tensile stresses are predicted in the wall concrete near the top ledge of the recess. These tensile stresses result from the Poisson's ratio effect acting on the prestress and dead load compressive stress concentrations.

Stresses in the corbel

64. Figures 5, 6, 7 and 8 show contour plots of maximum and minimum principal stresses in a vertical section through the corbel when subjected to the most severe crane vertical wheel loading (fault/snag condition) following construction of the corbel and containment wall up to the point preceding that at which dome construction would commence.

65. The two pairs of figures, 5 and 6, and 7 and 8, represent the two different conditions assumed for the top ledge contact, namely "with-gap" and "no-gap". As can be seen from the contour plots, changing these conditions has minimal influence on the corbel stresses except for the area close to the top ledge where, in the "with-gap" condition, stresses are induced due to the restraining effect imposed there. Absence of this restraint, in the "no-gap" condition, causes increases in the stresses across the lower ledge and adjacent to the top main reinforcing bars.

66. At a later stage, under the application of prestress, in the "no-gap" condition, prestress load is attracted into the corbel by way of the top ledge.

Fig. 5. Maximum principal stress contours
Fault/snag condition - no dome; with gap

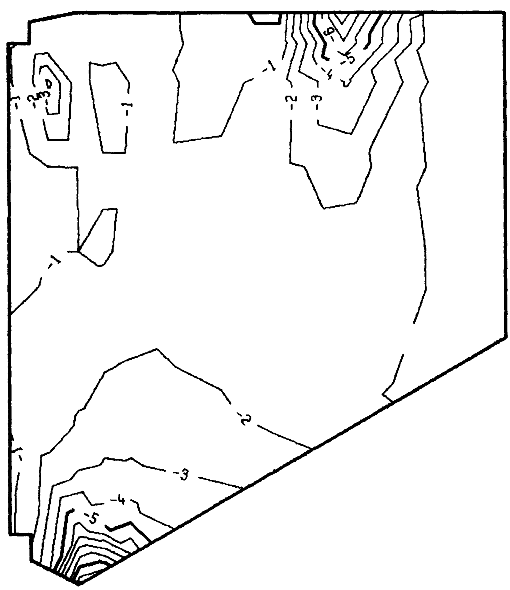

Fig. 6. Minimum principal stress contours
Fault/snag condition - no dome; with gap

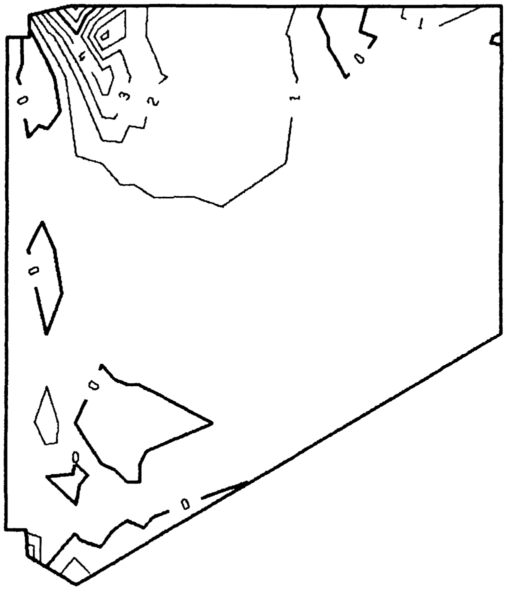

Fig. 7. Maximum principal stress contours
Fault/snag condition - no dome; no gap

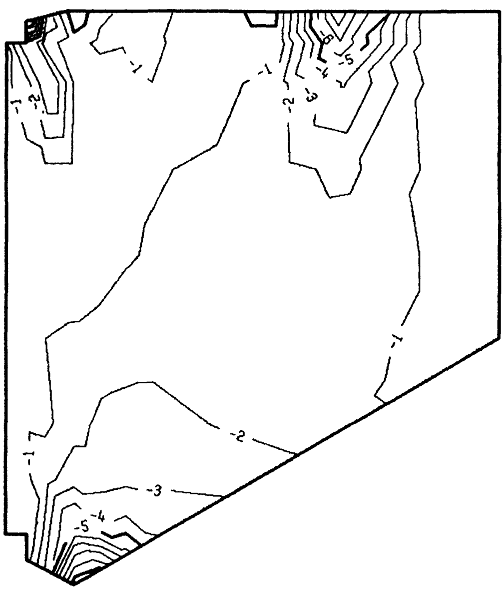

Fig. 8. Minimum principal stress contours
Fault/snag condition - no dome; no gap

67. Generally, the concrete stresses in the corbel are again within permitted limits, with triaxiality considerations being used to justify the acceptance of some high compressive stresses.

MOCK-UP

68. Following completion of the basic design and analyses, a full scale mock-up of a corbel unit was built on site. The objective of this exercise was to verify the practicability of erecting the corbel shutter, fixing the reinforcement and installing the other various embedments. For ease of continuous reference and eventual removal, the mock-up was not concreted.

69. The mock-up indicated the need for a number of minor amendments to reinforcement and coupler positions but, subject to these, demonstrated the overall feasibility of the proposed assembly procedures.

CONCLUSIONS

70. In so far as civil engineering aspects are concerned, the design, analytical and practical work undertaken has demonstrated that the corbel provides a satisfactory and adequate means of supporting the containment building polar crane throughout its working life.

71. The analyses have also indicated that the introduction of the corbel will not prevent the containment shell from fulfilling its required function, although local detail design changes are shown to be necessary to accommodate the effects of the corbel.

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15. The design of building cladding for extreme winds

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SYNOPSIS: This paper describes the development of the design of the building cladding systems for the THORP structures at Sellafield, with particular reference to the evaluation of cladding systems against the effects of extreme winds. Wind tunnel testing was undertaken to confirm areas of high wind loading on the buildings, and this work is described in detail. Load tests on cladding and support systems are discussed, and the way in which the resulting design has been specified, including performance tests, is explained. The impact on the design process of the need to consider the structural integrity of the cladding systems is discussed.

INTRODUCTION

1. The contribution of building cladding to the stiffness of structures is complex and highly variable, and is dependent upon the fixing to the structure. For this reason, building cladding is usually considered as a non-structural element. Nevertheless, in the context of the design of safety related structures for the nuclear fuel reprocessing industry, the building cladding does have a safety function. It is therefore required as part of the structural safety case to evaluate its performance against extreme environmental hazards.

2. This paper describes the development of the design of the building cladding systems for the THORP structures at Sellafield, with particular reference to the evaluation of cladding systems against the effects of extreme winds. Because little practical information was available for the level of loading required, wind tunnel tests were commissioned to confirm wind pressure coefficients as derived from CP3: Chapter V: Part 2. Loading tests on individual cladding panels were also carried out to evaluate their strength and that of the fixings, and the interaction of these with the support system.

FUNCTIONAL CRITERIA

3. Building cladding provides protection to the reprocessing plant from the elements. The cladding does not provide a containment function. Therefore, whilst loss of such protection could hamper reprocessing operations, it would not in itself pose a safety hazard. Design to normal commercial criteria satisfies this aspect of the functional specification.

4. Failure of the cladding system may expose internal plant and structures to conditions for which they have not been designed. In particular, the loss of significant areas of cladding may subject such systems to wind forces.

5. The significance of wind loading on the internal systems is clearly a function of the processes within the building. Considering the difficulties which may be experienced in qualifying items of plant against extreme wind loads, it is clearly preferable to ensure that the cladding does not fail, even under the most extreme wind conditions. The cladding therefore has a safety function in protecting the internal systems from these conditions and is defined as safety related. A structural safety case must be made to demonstrate adequate performance of the cladding system under extreme wind loads.

EXTREME WIND LOADING

6. Wind loading is considered within the structural safety case as an external hazard i.e. an event originating outside the plant. The safety principles adopted by the nuclear industry require safety related plant and structures to be designed to withstand the effects of such hazards acting simultaneously with maximum normal operating loads (ref.1). There is no requirement to consider the simultaneous occurrence of external hazards, unless there is a common mode connection.

7. Extreme weather conditions are based upon the best meteorological data available for the area and take account of appropriate combinations of:

a) extreme wind loading
b) accumulated ice
c) high rainfall
d) heavy snowfall
e) lightning
f) spray and fog

8. What is the extreme wind loading? It is defined as an event with an exceedence level of 10^{-4} p.a., the well known 1 in 10,000 year event. For wind loading this can be derived from CP3: Chapter V: Part 2 (ref. 2) through use of the S_3, statistical factor.

9. The S_3 factor is usually taken as 1.0 for wind loads on completed structures. This leads to a wind speed likely to be exceeded on the average only once in the 50 year lifetime of buildings covered by the code. This is the 50 year wind.

10. More precisely, the definition of the 50 year wind is the wind speed which will be exceeded only once with a probability of 0.2 in any one year. Noting that the return period (T) and the probability (P) are related by TP=1, the probability P that a wind speed greater than the value from the code will occur at least once in a period of N years is given by:

$$P = 1 - \left(1 - \frac{1}{T}\right)^N \qquad (1)$$

11. Thus the probability is 0.63 that the 50 year wind speed (T=50) will be exceeded at least once in a period of 50 years (N=50). The probability that the 10,000 year wind will be exceeded at least once in 50 years is 0.005.

12. Figure 2 of CP3: Chapter V : Part 2 gives the values of S_3 for values of P between 0.001 and 0.63, and values of N up to 200 years. The values given are the mean values of S_3 throughout the United Kingdom. Using an exposure of 50 years and a probability P of 0.005 this figure gives a value of S_3 of approximately 1.42 i.e. the 10,000 year wind speed is 1.42 times the 50 year wind speed. As wind pressure is a function of the square of the wind speed, the 10,000 year wind pressure is approximately 2 times the 50 year wind pressure.

13. CP3: Chapter V: Part 2 cautions that wind speeds with such low probabilities may not be estimated satisfactorily from a record of even 50 years data, as there may be other factors which affect the values of such an extreme. To check the figure derived from CP3, the methods detailed in the Engineering Sciences Data unit (ESDU) wind series (ref.3) were used. These methods are based upon the statistics of wind pressure rather than wind speed, and lead to the conclusion that the CP3 method overestimates the 10,000 year wind speed. This conclusion is supported by later BRE publications on extreme wind speeds (ref. 4-5).

14. For the purposes of the structural safety case the 10,000 year design wind speed has been taken as 1.42 times the CP3 50 year wind speed. The topography factor S_1, and the ground roughness, building size and height above ground factor S_2 are assumed unchanged.

CLADDING DESIGN LOADS

15. The design wind speeds derived from CP3: Chapter V: Part 2 define the general wind environment for the site. In order to confirm the topographical effects of the very large structures on the site wind tunnel tests have been carried out. It is emphasised that this testing was carried out to provide confidence in the design for normal wind loads as well as data for design under extreme wind loads.

16. The wind tunnel tests were carried out at Oxford University Engineering Laboratory during 1984 to obtain, inter alia, the following data.

(i) The pressure distribution on the THORP Building, to check the adequacy of CP3 data, in particular in areas of high suction likely to occur because of funnelling between THORP Head End and the THORP Receipt Building.
(ii)The pressure distribution due to extreme wind loading.

17. The model scale used was 1:250, which enabled the majority of the large adjacent buildings to be accommodated in the wind tunnel, e.g. Pond 5, SIXEP, WVP etc. Smaller buildings, and local terrain features, such as the river were included for completeness. As the terrain around THORP is not uniform, three different simulations were used as shown in Fig. 1.

i) Wind from the sea.
ii) Wind from the land.
iii)as ii) but modified by the adjacent sections of the Sellafield plant.

18. Wind speeds were measured by means of a hot wire anemometer, giving instantaneous readings (of the order of 100 per second) which were processed to obtain mean and turbulence intensity values. Measurements were made to verify the wind simulation at the centre of the turntable without any of the THORP models present; with the surroundings present but not the THORP complex, and at the stack locations with THORP present. In addition measurements were made to determine the effect of THORP on the global wind environment at Pond 5, WVP and future (EP1) sites.

19. The air pressure on the face of a building was measured by connecting tubes set into the face to a pressure transducer. The pressure measured in this way was not constant for a given wind speed and direction, but fluctuated due to turbulence, both in the incident wind, and induced by the shape of the building. The design pressure, according to CP3, is the peak pressure that occurs in a given time interval. However, when the measurement was repeated, the peak observed value changed due to the random nature of the pressure fluctuations. As an alternative, a probabilistic representation of the pressures can be obtained, but this required a large number of pressure measurements, at least 16 values of pressure at each point.

20. On the THORP model a total of 308 points were selected for pressure measurements. During preliminary testing of these, 86 points were subjected to further testing using the multiple pressure method. In addition average pressures on the faces of structures were obtained by use of manifolds. This pneumatic averaging was used to estimate the average pressure on 47 faces of THORP, each face on average representing about $100m^2$ at full scale. The probabilistic method applied to single point readings was used for these 'surface' coefficients.

21. It should be noted that extreme pressure data has generally only been obtained for points where high pressures or suction were shown to exist by the initial tests.

22. Wind pressure coefficients can be expressed in the form

$$Cp=Uc+dc(-\ln(-\ln(1-P))) \qquad (2)$$

where Uc = Mode of pressure coefficient

dc = Dispersion of pressure coefficient

P = Probability of exceedence

and ln = Natural logorithm

23. A similar formula can be drived for the dynamic wind pressure, using coeffients derived from ESDU 82026. By considering the probability of exceedence of the total wind pressure using both data on dynamic wind pressure and on the pressure coefficients (ref. 6), a more accurate and less conservative value of the pressure is obtained than that obtained from CP3.

24. Using the method described above, together with internal pressures calculated in accordance with the CP3 handbook (ref. 7) but using experimental data, values of net pressure were calculated for annual probability levels of 0.02 and 10^{-4}. The former corresponds to the 50 year return period of CP3 and the latter to the return period of 10,000 years used for external hazards.

25. The resulting data for single points were presented for:

i) pressures averaged over 1 second applicable to small areas of cladding
ii) pressures averaged over 4 seconds applicable to large areas of cladding and for the design of purlins and sheeting rails

26. For pneumatically averaged extremes the data was averaged over 4 seconds, and is applicable to elements of the structure less than 50m in length.

27. For the purposes of further discussion the 1 second pressures only are considered for the single point data, and 4 second pressures are considered for the pneumatically averaged data.

RESULTS OF WIND TUNNEL TESTS

50 Year Wind

28. Generally the measured pressures are less than those calculated from CP3 except in the following areas:

(a) at the west corner of the centre building
(b) at the north face of the storage pond cover

29. Also positive wind pressures were measured on the various roofs, whereas in general CP3 would indicate that positive pressures would not occur. However, the building configuration, particularly the layout of the roofs, is outside the scope of CP3. Suction values on the roofs are within CP3 values.

10,000 Year Wind

30. On the walls, the pressures due to the 10,000 year wind exceed the CP3 50 year design pressures at a number of points. With respect to the design of cladding, Table 1 compares the measured and calculated values.

31. For the roof areas, the measured suctions under the 10,000 year wind exceed the CP3 50 year suctions at the south east and south west corners of the Head End complex and on the south west corner of the Storage Pond Cover.

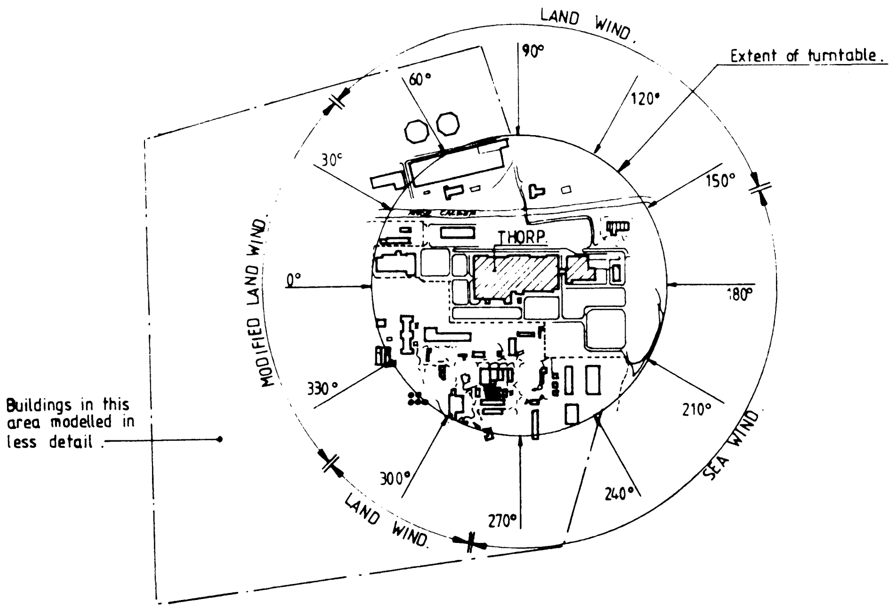

FIGURE 1 - LAYOUT OF MODEL SHOWING WIND DIRECTIONS AND TERRAIN SIMULATIONS USED

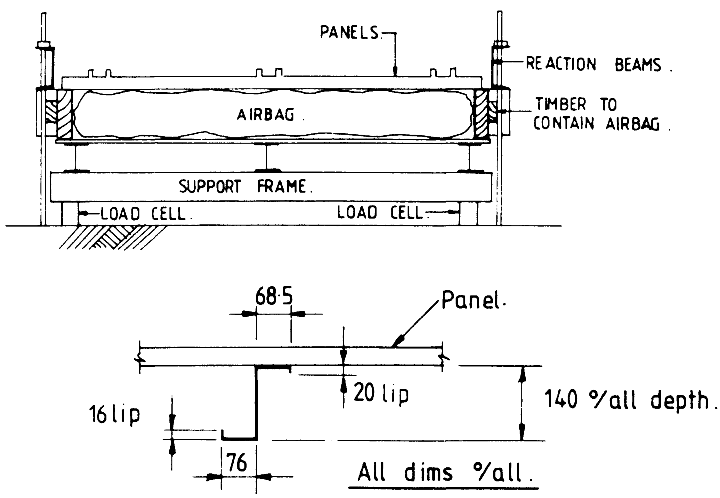

FIGURE 2 - COMPOSITE PANEL DETAILS

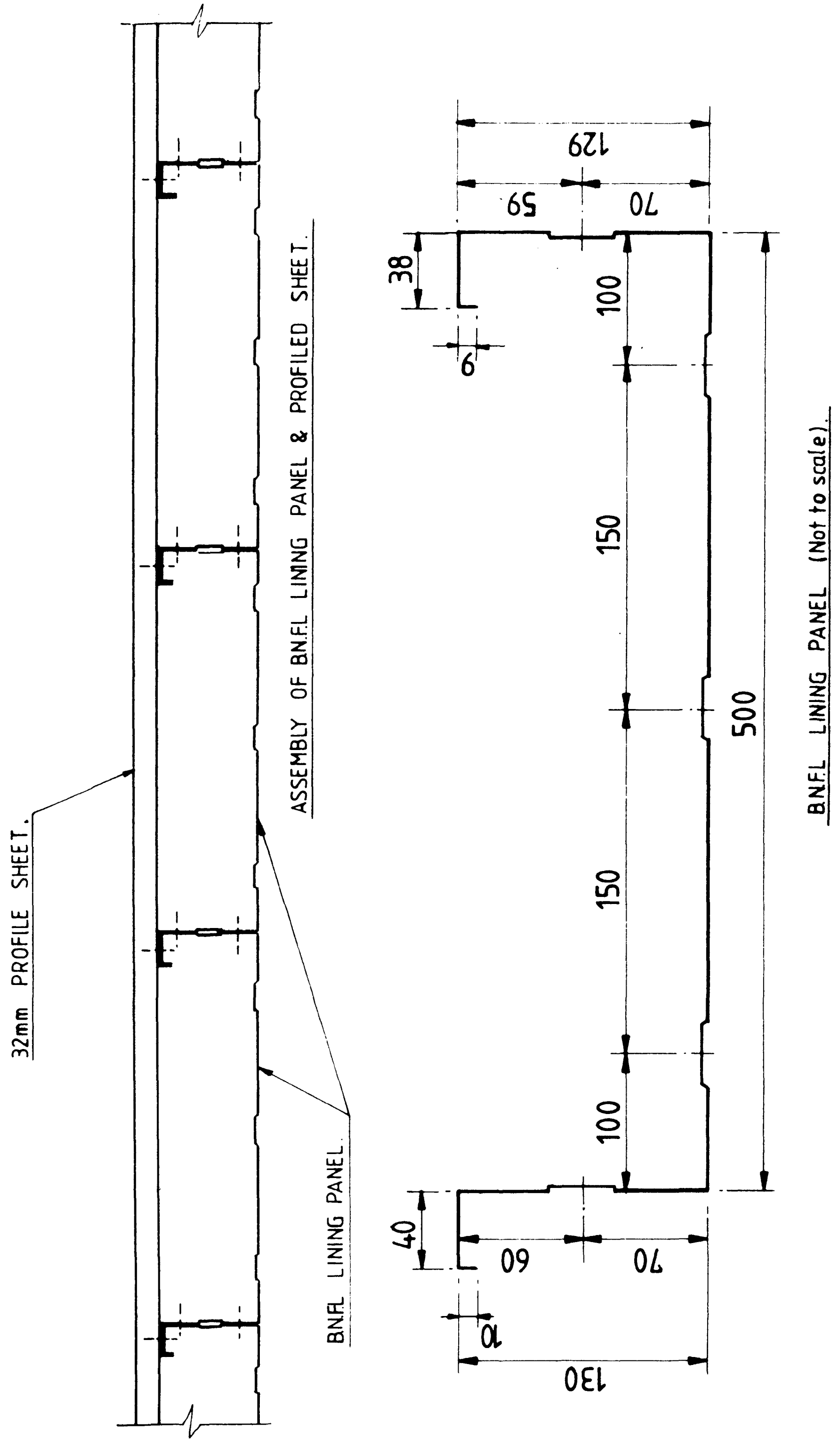

FIGURE 3 LINER PANEL DETAIL & ASSEMBLY

CONCLUSIONS OF THE WIND TUNNEL TESTING

32. The following conclusions were drawn from the wind tunnel tests:

a) Apart from two small areas the pressure distributions derived from CP3 for the 50 year wind will be conservative.
b) Because of conservatisms within CP3, the pressure distributions derived from CP3 for the 50 year wind are generally conservative for the 10,000 year wind pressures, although some increase in local design pressure was required on certain elevations.

33. The results of the wind tunnel testing gives high confidence in the design which was progressed on the basis of CP3: Chapter V: Part 2 wind pressures.

CLADDING TYPES

34. There are two main types of cladding used on the THORP project. These are:

i) Patented curtain walling glazing panels (Management Centre)
ii) Metal panels

35. In general the wind pressures on the Management Centre can be seen from Table 1 to be within the CP3 design values. The remainder of this paper is devoted to the development of the design and testing of the metal cladding systems in which significant suctions occur, particularly on the pond cover.

36. Two types of metal cladding systems were used on THORP. These were:

i) Composite foam filled metal panels (Fig. 2)
ii) Structural liner tray systems (Fig. 3)

The first was selected for ease of maintenance, and was used on the Receipt Building, Head End and Chemical Separation areas. The latter type was used in areas where it was required to minimise contamination traps on the inside face of the building envelope. This system avoids the use of sheeting rails, and, for the THORP Pond Cover, is unique in that the structural trays are manufactured from stainless steel to ensure the cladding is maintenance free over its 40 year design life.

COMPOSITE CONSTRUCTION METAL PANELS

37. Tests were carried out at Salford University to demonstrate that this type of cladding could carry the prescribed loads, the fundamental problems being the very high suction and the very localised restraint to the panels. In view of the conservatisms within the design loading demonstrated by the wind tunnel testing it was considered that proof testing of this system could proceed on the basis of equivalent static loading. Further simplification was achieved in the testing by applying the suction, as a pressure, by means of an airbag. The efficacy of this arrangement was reviewed as the test programme was developed and was judged to provide sufficient accuracy for the purposes of the tests.

38. Initial tests were carried out to establish the significance of the following variables:

a) Sheeting span
b) Sheeting thickness
c) Type of clip
d) Clip seating
e) Number, type and centres of screws
f) Rail type and thickness
g) Tolerances, particularly the tolerance on rail thickness

39. Also these tests established the various forms of failure which can occur as:

i) Panel
The panel may fail overall in flexure or locally at a fastening. Local failure was most likely at the end of the sheet, and therefore the end distance on the tests was carefully selected to match those used in practice. In addition excessive deflection under load was undesirable and was limited to span/120.

ii) Fasteners
   a) The screw head could pull through the clip.
   b) The clip could bend under the applied load.
   c) The bearing pressure could indent the metal surface under working conditions.
   d) The lip of the sheet could tear locally around the clip.
   e) The sheet, under the curvature induced by the load, could pull out of the clip.
   f) Pullout of the fasteners.

The failure in a) had not been observed in the initial tests and was considered unlikely. All the other failure modes had been observed in the initial tests.

40. Following the initial tests, two pilot tests were carried out. One test used a 2mm thick rail to establish the lower limit of pullout strength, and the second test used a 3.2mm thick rail to establish the upper limit of strength, considered to be governed by sheet failure.

41. The pilot tests confirmed the failure modes as pullout of the fasteners from the rail for the 2mm thick rail, and flip out of the sheet for the 3.2mm thick rail. It was noted that delamination of the panel occurred at a lower pressure. As a result of these tests a 35mm panel thickness and a rail thickness of 2.5mm were adopted.

42. Proof testing of the cladding system was conducted on 5 specimens; fixing procedures used were these adopted on site to simulate the effects of workmanship. Maximum loading was $6.0kN/m^2$, which was applied as a uniformly distributed load through an airbag.

43. Pressure was applied in increments of 0.5 kN/m^2 up to $4kN/m^2$, a total of 6 cycles of loading being carried out. Finally the load was increased to 6 kN/m^2 with continuous observation on the end of the sheet to determine the mode of failure.

44. Pull out tests were also carried out on the fixing screws to establish the sensitivity of the results to the positioning of the screws on the top flange of the Zed rail.

45. The tests overall demonstrated the reliability of the system which is difficult to model accurately because of the uncertain force distribution between rails, cladding fixings etc. The tests highlighted several features which would not have been apparent by calculation alone:

a) the sensitivity of fastener pull out to minor changes in thickness of the rails. A 0.5mm difference does have a significant effect on strength and this is approaching the rolling tolerance variation. For commercial reasons it was very important to prove that self tapping fixings would be adequate. The alternative would have been to drill through hot rolled steel sections from the outside and bolt (i.e. teams inside and outside the building). In the end the design of the rails was based on a thickness which ensured that the overall rail was more than strong enough to carry the load.

b) The sensitivity of the cladding failure to detailed aspects of the clip design. Failure was originally initiated by curvature of the sheet around the clip which punctured the outer metal layer prematurely precipitating a failure in the foam. The introduction of a neoprene layer provided resistance to delay this form of failure and significantly increased overall strength.

LINER TRAY SYSTEM

46. The main contractor submitted design calculations for approval to demonstrate that the chosen system, including fixings, would carry the design wind pressures with adequate factors of safety. The design calculations included an evaluation of the liner tray section properties based on BS 5950: Part 4 (ref. 8). This code was chosen as an available, simple to use, limit state code of practice.

47. To give confidence in this new design method, a series of tests was carried out on 3 fully assembled panels. The tests were sponsored by the panel manufacturer and undertaken by British Steel Corporation. BNFL engineers gave technical guidance and witnessed the tests.

48. Each of the 3 panels was 5000mm long x 1650mm and represented an "as-built" section of the building envelope. The tests were carried out using a vacuum box. Two panels were tested under wind suction loads and one panel was tested under pressure. Each of the panels was tested at least 3 times; the first test took up any initial slack in the system. Deflection recordings indicated that the system met the design criteria and also that its behaviour remained elastic at these loads. Each panel was finally tested to failure which was deemed to have occurred on the formation of a 'hinge' close to the midspan of the liner panels. The hinge appeared to be formed by local buckling of the stainless steel tray material. Mid span defections at design loads were approximately half the specified limit and all the panels behaved elastically at 1.4 times the design loads with a comfortable margin to final failure.

49. It should be noted that, in this case, the design loads were based on the 1 in 50 year wind as the cladding was not safety related. However, the cladding performance under the 1 in 10,000 year was evaluated and reported. It was noted that at "failure" during final testing the fixings to the supporting structure remained undamaged and allowed the panel to develop catenary action. None of the panels were loaded further to final collapse due to the limitations of the test rig, but it was possible to calculate an approximate panel collapse load from the load and deflections measured at failure.

SPECIFICATION

50. The test results gave confidence in the new design method and this was recognised in subsequent BNFL cladding specifications. Calculations may be submitted in support of the design of cladding systems as an alternative to full scale tests. The design should demonstrate an adequate factor of safety against fixing failure and it is preferable to ensure failure occurs in the panel before fixing failure.

51. Also included in BNFL specifications is the requirement for material certification. Many cladding designs assume material properties better than those guaranteed by material manufacturers. In effect, advantage is taken of the conservatisms where inherent in material specifications along with further enhancements of material properties due to cold working during manufacture. This is acceptable only when the design includes documentary evidence of the enhanced properties. Test certificates quoting proof and tensile strengths are the usual means to this end.

52. A possible alternative to either calculation or full scale testing is the use of data from previous tests conducted by accredited independant testing organisations. Examples of these are data contained within any agreement certificate or the German Zulassung. While this type of document is very useful when designing a cladding system to resist 50 year winds the effects of extreme winds are often outside their scope.

DISCUSSION

53. It should be clear from the work described in this paper that the impact of defining cladding systems as safety related on the design process has been significant. What would otherwise have been a relatively simple 'catalogue design', required consideration from fundamental principles of the behaviour of systems made of composite materials. The testing described was in the one case essential to develop the fixing details and allowed an economic design to be properly derived and demonstrated; in the second case the testing was required to validate a new design method. In both cases a more innovative approach was required than for conventional design. In addition early recognition of these requirements was necessary to avoid delay in the design programmes.

54. The benefits derived from the work undertaken not only relate to an adequate design of the cladding under extreme winds, but also include a better understanding of the behaviour of these systems under 'normal' loading, leading to improved safety and performance.

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Table 1 Maximum 10,000 Year Wind Pressures for Cladding Design on Walls (kN/m^2)

	CP3 (50 year)	Measured
Receipt Building	Max. +2.6 Min. -2.8	+2.1 -2.6
Pond Cover	Max. +2.6 Min. -2.8	+2.8 -5.9
Management Centre	Max. +1.5 Min. -2.0	+1.6 -1.5
Head End/Chem.Sep.	Max. +2.2 Min. -2.9	+2.6 -2.7
Centre Building	Max. +1.6 Min. -2.4	+2.4 -4.0

Discussion

D. DRYSDALE, BNFL, Sellafield
What account does Mr Mills take of dominant openings in your wind tunnel testing? Was allowance made for wind loadings and pressure on internal walls and structures near dominant openings?

D. E. KEY, CEP Research, Bristol
Surveys of recent hurricane damage in the West Indies, notably in Montserrat and Dominica where gusts exceeded 200 mph, showed the primary cause of cladding failure to be fatigue. Does Mr Mills have any comment?

C. J. HELM, Nuclear Electric plc, Knutsford
The thrust of Paper 13 concerns strain accretion at the corners of lined cells. No reference was made to fatigue considerations in the corner welds, between the sub-panels in the structure, or temperature fluctuations in service with general panel buckling with consequential axial out-of-plane bending on sub-panel corners. Did the design or analysis take account of this?

S. W. JONES, W. S. Atkins Consultants Ltd, Epsom
The use of models not only confirms analysis but also helps to predict construction problems - how were these recorded and used, and how is the scaling up to the real structure turning out in practice?

J. L. COSTAZ, SEPTEN, France
I was a little surprised to hear Mr Paton say that reinforced concrete could replace steelwork when steelwork is not convenient. In general, the reverse is true. I wonder if the difficulties in this area are not the consequence of adding the conservatisms as Dr Key suggested. As far as the safe shutdown earthquake

case is concerned, do you consider the polar crane is loaded with the heaviest equipment?

R. H. COURTIER, W. S. Atkins Consultants Ltd, Epsom
One of Mr Butler's slides indicated that two cycles of testing lower the failure load from 5.8 to 3.5 kN/m^2. The foam used is brittle in character. Did the testing consider cyclic loads, as extreme winds produce high cyclic buffeting?

J. IRVING, Paper 12
The PWR project group have made extensive use of modelling in the planning, the design and the construction of Sizewell 'B'. As described in our Paper, a 1/10th scale model of the primary containment was tested to validate the design for operational test and fault condition loads, and to ascertain the beyond-design basis behaviour. We have also used full size mock-ups to prove construction techniques in difficult areas. The overall design and construction development of equipment and pipework layout was greatly assisted using a plastic scale model of the nuclear island and this continues to be used at site. Finally, 3-D computer modelling has recently been used with great success for the design of the radwaste systems and adapted to model complex areas of embedments, rebar and pre-stressing in the primary shield wall and containment. All these modelling techniques have proved invaluable in helping to identify potential construction problems before they occur.

W. JORDAN, Paper 13
Normal operating loads are static in their nature, i.e. hydrostatic loading does not vary over the design life of the structure, and thermal variations are only a few degrees centigrade, due to the large thermal capacity of the structure.

Out of plane buckling is only considered as a construction tolerance effect, producing enhanced in-plane loads which are added to all design load cases.

A. A. PATON, Paper 14
The difficulties encountered with the steel support system arose from the fact that, whereas the SNUPPS design is based on a safety case which does not require safe shutdown earthquake (SSE) and loss of cooling (LOCA) to be combined, the UK design does require these

two conditions to be considered in combination. The consequent temperature conditions are very onerous and were leading to a steel design which would have been very heavy, expensive and difficult to inspect. Concrete is much less sensitive than steel to temperature loading and the decision was therefore made to adopt a concrete support system rather than a steel one.

Regarding the safe shutdown earthquake condition, this is required to be considered only after the reactor has become operational. By that time, the crane will have been downrated and, while it may well be lifting the heaviest piece of equipment it is required to lift at that time, the vertical wheel load is less than any of those considered for construction, overload test and the fault/snag condition. The radial and tangential loads during this condition are, however, at their maximum values, and due account was taken of these.

Paper 14 briefly mentions two features which played

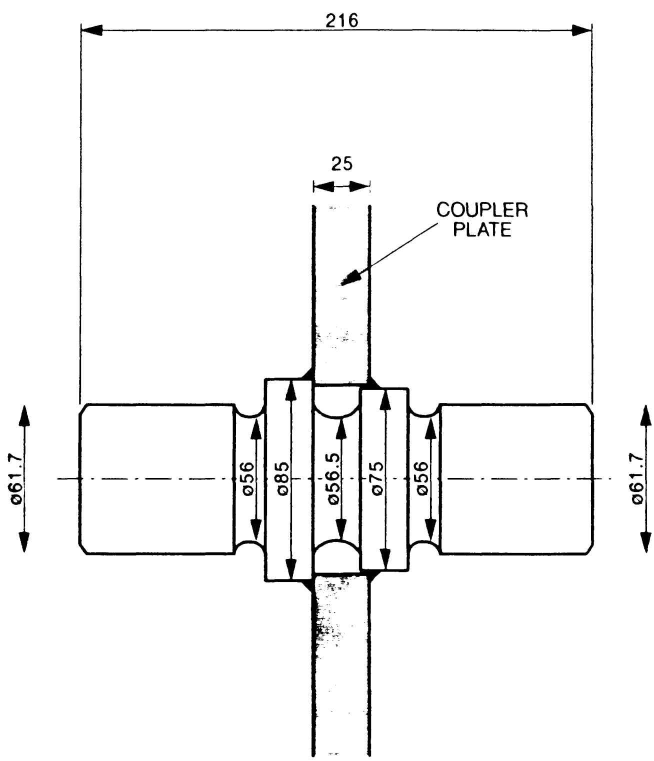

Fig. 1. Through-liner coupler.

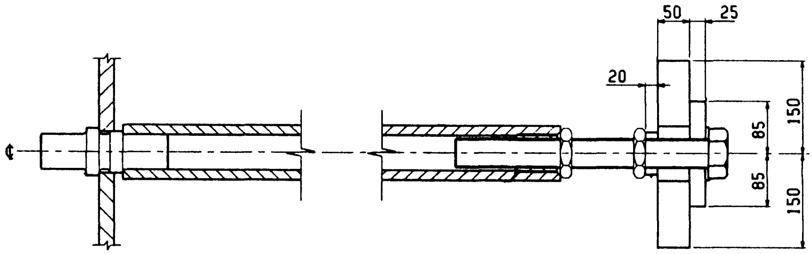

Fig. 2. Top bar long coupler and anchorage.

key parts in the success of the corbel design and I would like to take the opportunity here to expand on the information given. The first of these two features is the through-liner coupler, which is shown in Fig. 1. This is probably the thing that has contributed most to the feasibility of the corbel design. The through-liner coupler is essentially a double-ended stud, threaded at both ends with two flanges near the middle. The smaller of the two flanges is passed through a hole in the coupler plate and then the flanges are seal-welded to the coupler plate as shown. Reinforcing bars, with female threaded coupler sleeves already attached, can be connected to the projecting studs, thereby giving continuity of reinforcement through the containment liner without violating the requirement of maintaining a gas-tight membrane. These devices had been used to allow 50 mm diameter bars to pass through the reactor building floor liner, and all the relevant qualification tests had already been done for this purpose. Thus, while for the corbel it would have been possible to use 40 mm diameter bars for the main reinforcement, the decision was made to use 50 mm bars and so avoid the need for the further qualification tests that would otherwise have been necessary.

The through-liner couplers are not qualified to carry shear loads and for this reason they and the reinforcement attached to them are wrapped in soft material for a distance of 240 mm on either side of the coupler plate to reduce shear loading to a negligible level. The action of this wrapping was reflected in the analytical results. The couplers did, however, need to undergo a series of cyclic load tests and the information for these was derived from the analyses.

The second feature I would like to mention briefly is the top bar long coupler detail, which is shown in Fig. 2. This detail was chosen for two main reasons; firstly because there is insufficient length across the top of the corbel and insufficient room down the front face to accommodate a 50 mm bar with adequate bond length, and secondly because it provides the flexibility necessary to accommodate the variation in bar length resulting from the large tolerance on the radius of the containment liner. The long coupler is anchored at its end remote from the coupler plate by an anchor plate, nut and washer arrangement, as Fig. 2 shows.

J. H. MILLS, Paper 15
In response to Professor Key's point, the observations made during testing suggest that the performance of cladding is dependent upon careful detailing, and would support the conclusions of the surveys mentioned by Professor Key. As finally detailed, the cladding system withstood the 10 000 year wind loads with a considerable margin, and failure due to fatigue was judged to be unlikely.

Dominant openings were not modelled in the wind tunnel testing. Wind loads due to the influence of dominant opening were considered by reference to the wind code and its explanatory handbook. The effects on internal partitions were also considered, again with reference to the above documents. It is part of the usual design process to consider these loads where appropriate.

The slides of test results related to the liner tray system, which does not include a compressible foam insulation. However, fatigue-induced cladding was considered unlikely.

16. The development of quality assurance and quality management in civil engineering

N. D. HASTE, Laing Civil Engineering

SYNOPSIS. The paper describes the development of Quality Assurance and Quality Management at Sizewell from the lead provided in the Client's documentation. The hard lessons learnt from dealing with major suppliers and sub-contractors are explained. The paper concludes by describing the ways in which total Quality Management is now addressed within Laings Civil Engineering following the experiences at Sizewell and elsewhere.

INTRODUCTION

1. The civil engineering Works at Sizewell 'B' station is subject to quality assurance requirements commensurate with its integral importance as part of a major nuclear installation. Quality assurance is in accordance with BS5882 and this entailed a respectful and objective approach to the whole ethic of quality management and quality assurance.

2. The tender process for the main civil engineering works commenced in January 1985 and at that time quality assurance was in its infancy in the civil engineering industry although quality assurance was well advanced in the mechanical and off-shore oil industries. The only other project at this time where quality assurance was a requirement to similar standards as those to be achieved at Sizewell 'B' was the THORP project at Sellafield.

3. The works at Sizewell 'B' commenced on site in September

1987 and at the time of writing this paper the works are 60% complete against a 78 month programme. I am pleased to report that the whole project is on course to achieve commissioning by the date originally set, that is early 1994. The main civil engineering works is due for completion on February 28th 1994 but the bulk of the work will be completed by the Spring of 1992.

4. The main aspects of the development of quality assurance and quality management have been:

- the setting up of quality systems

- the education of staff and the workforce in a rigid quality assurance environment

- the management of off-site works carried out by suppliers and sub-contractors

- the management, monitoring, audit and improvement of quality systems.

DESCRIPTION OF THE WORKS

5. The works comprise all of the major buildings associated with power generation and the treatment of low level nuclear waste.

6. Sizewell 'B' is the first PWR to be built in the UK. The reactor building is the dominant structure within the station and is prominent when viewed from a distance because of its hemispherical dome. The reactor building, auxiliary building, control building, and fuel building comprise what is commonly referred to as the Nuclear Island; and forms half of the power block. The other half of the power block consists of the Turbine Hall, Mechanical Annexe and Electrical Annexe which combine to form the conventional power generation buildings. Low level waste is treated in the radwaste building and there

are other peripheral buildings housing control equipment and chemical plant. There is a maze of cable tunnels which extend around the perimeter of the power block below ground.

7. The civil engineering works led the mechanical and electrical works by two years. The main interfaces between civil and mechanical and electrical works occurs at the provision of steel plate embedments located to fine tolerances on the face of concrete surfaces both horizontal and vertical. These embedments are used to attach supports for machinery, pipework, cable trays and other equipment. There are numerous other cast in items all of which have to be set to exacting standards of accuracy. All embedded items also had to be manufactured to stringent specifications.

The major civil engineering quantities are:

Excavation	-	600,000 cu m
Structural Concrete	-	450,000 cu m
Formwork	-	645,000 sq m
Reinforcement	-	64,000 tonnes
Structural Steelwork	-	9,500 tonnes
Embedments	-	50,000 No

QUALITY ASSURANCE REQUIREMENTS

8. The Client, CEGB Nuclear Electric, sensibly laid down the levels of quality assurance to be attained in relation to the safety requirements of the operational plant. Six levels of quality assurance were specified as follows:

Q - Highest grade of control and verification activities

N - Non safety classified items, but covering a wide variety of plant and services which impact safety classified equipment.

N/S - Safety related importance to the plant

NO - Operational importance which would result in reduction in station design output if a failure occurred.

NE - Non safety, non operationally significant items which require significant design input by the contractor or manufacturer.

N/ - Non safety, non operationally significant items produced in mass quantity in accordance with manufacturers specification by routine manufacturing operations.

9. The objective of this grading ensured that quality assurance requirements were applied to suit the importance of the relevant part of the works in the nuclear safety context.

10. There was a requirement at the outset for the contractor to demonstrate that he had a coherent quality plan with systems and procedures to meet the parameters laid down by the Client. Such systems had to provide for internal audit, the agreement and delegation of quality requirements to sub-contractors and suppliers and the maintaining of documentation control for the purposes of retaining lifetime records. These records had to include for full traceability on materials and processes.

SETTING UP OF QUALITY SYSTEMS

11. Overall responsibility for quality assurance is taken by the Laing Project Director. The Quality Manager reports to the Project Manager on day to day matters but has direct access to the Project Director.

12. The Quality Manager is responsible for documentation, quality control inspection, audit and review. His

responsibilities also include assessment of off-site suppliers and sub-contractors prior to award of contract and during the execution of the works.

13. Quality Management had to be established from basic principles and this was commenced by preparing a Quality Assurance Programme and manual which addressed the areas defined in BS5882.

14. From this base the following documents were produced as live documents subject to change when necessary.

Quality plans for individual sections of work incorporating method statements.

Standard procedures which would be used within method statements.

Laboratory procedures for the testing of materials.

Site procedures for the formalisation of practices in the execution of the works.

15. Responsibility for the preparation of procedure was given to each operating department and was organised such that wherever possible existing practices and interface management were formalised. This was done to create the acceptance of quality assurance and indeed a commitment to it.

16. The Quality Assurance programme provided for the regular review of the adequacy and effectiveness of systems and procedures.

17. The organisation of responsibilities of individuals within the quality management sphere was clearly defined and it was decided to set up an independent inspection department for on site works as well as off-site works. In effect this entailed providing what would be accepted as Resident Engineers and Inspectors in a normal consulting engineer contractor relationship. Whilst this set-up worked in terms

of controlling the "paper-chase" it did nothing to develop an all round commitment to quality management. The pressures to achieve production particularly in the early stages of construction meant that supervisors and engineers responsible for production were receiving the "big stick" treatment from the quality control inspectors. This was a valuable lesson learnt for the future and later it will be explained how a total commitment to quality management was achieved. In short the simple answer is to make sure that the responsibility for quality control and verification that the required standards have been met is placed firmly with those who are carrying out the work.

THE EDUCATION OF STAFF AND THE WORKFORCE IN A RIGID QUALITY ASSURANCE ENVIRONMENT.

18. The American Society of Civil Engineers has given a lot of attention in recent years to developing a manual of professional practice as a guide to achieving quality in the constructed project.

19. The guide opens with the following statement which is worthy of note when considering education of people in quality management.

> "Quality is never an accident. It is always the result of high intention, sincere effort, intelligent direction and skilful execution. It represents the wise choice of many alternatives."

20. It was decided at the outset to carry out comprehensive induction of all staff into the quality assurance requirements of the project and the role that each individual was expected to play, together with his or her responsibilities. With a staff that peaked at over 500 the induction task was very onerous but nonetheless vitally necessary. In most cases it was necessary to convince members of staff that all that was being sought was a formalisation of what they already did by

writing it down and then making sure that they provided hard documentary evidence to verify what they had done.

21. This very simple approach worked in most cases but there was a mixed reception from the older died in the wool foreman who saw this as yet another requirement to prevent them getting on with the job. I am pleased to say that the value of quality management is now appreciated by everyone. This is evidenced by the pride and commitment to producing a good job but I should stress that it has not been achieved by the operation of a quality assurance programme in the formal sense.

22. Considerable training was necessary at all levels within the organisation to ensure compliance with the basic requirement that all those involved with quality related work should have the appropriate qualifications to carry out their duties. The training extended for instance to ensuring that those operatives who would normally be considered competent to vibrate concrete, were qualified to do so by putting them on a course and issuing a certificate when they satisfied the standard required.

23. Training records are kept and updated as appropriate. A programme of regular review was incorporated within the Quality Programme, to ensure that training requirements are being fulfilled for those working on Quality related works.

24. All training is geared to obtaining the commitment of everyone to quality improvement to developing and maintaining the skill levels needed to comply with Q.A. requirements.

25. Traceability of operations as well as materials are such that individuals can be identified against completed work. This further demonstrates the need to allocate responsibility to individuals carrying out the work.

THE MANAGEMENT OF OFF-SITE WORKS, CARRIED OUT BY SUPPLIERS AND SUB-CONTRACTORS.

26. The Quality Department was responsible for carrying out

assessment of Suppliers and Sub-Contractors before award of contracts and for Q.C. Inspectors after award.

27. Regrettably this sphere of activity proved to be the most difficult and has invoked considerable criticism of the way in which some companies run their business.

28. With each major supplier and sub-contractor a detailed assessment was carried out of their Quality Systems and their capacity as well as capability to carry out the work.

29. In the early stages of placing, supplying orders and sub-contracts, there was no detailed examination of the way companies managed their businesses outside the scope of Q.A., but it was soon realised that good Quality Management Systems do not necessarily reflect good overall Management. This was a hard lesson to learn and one that all Civil Engineering Contractors should be aware of.

30. Most of the management deficiencies occurred in well established and in many cases, very well known Mechanical Engineering related Companies.

31. In considering the statement in the ASCE guide referred to earlier, Quality Management stopped at the formal documentation and procedure stage in the cases of some companies. There was a distinct lack of good man-management practices with disregard for the natural need to communicate between manager and worker. This resulted in lack of motivation, reduced effort, more than tolerable re-work, excessive labour costs and delays to the programme. The very maxim that should be served by a Quality Assurance programme was undermined and meant that considerable man-power had to be invested by the main contractor to save his programme and at a large cost.

32. It became necessary to place senior people at the premises of some suppliers to do their work for them. It should be stressed that in one or two cases, major national and international firms with so-called proven track records were involved.

33. On the positive side there were some companies notably younger ones with a staff of committed people who produced excellent work and with a realistic eye on their future reputation. It was refreshing to see the approach when compared with more established firms.

34. In order to provide the reporting necessary to monitor performance of suppliers and sub-contractors off-site, an off-site supplier manager was appointed after the early salutory lesson, to supplement a number of Q.C. Inspectors. Regular Audits and reviews of Quality Systems were carried out.

THE MANAGEMENT MONITORING AUDIT AND IMPROVEMENT OF QUALITY SYSTEMS.

35. The Quality Assurance programme provides for regular reviews and monitoring of internal systems. In addition the Quality Systems are subject to Audit by both Nuclear Electric and the Nuclear Installations Inspectorate. Non-conformances and corrective action notices are dealt with speedily.

36. Many lessons have been learnt to date, the main one being that a Quality Assurance Programme with its associated systems and procedures does not in itself provide the recipe for good Quality Management. It does form part of Total Quality Management, a much used piece of jargon these days.

37. To produce Total Quality Management, Laing through early lessons well learnt, decided to create a process of continual updating in Quality Awareness, the immersion of all staff in a regime of complete openness with respect to deficiencies, mistakes, wastage etc, thereby encouraging contributions to improvement of quality in the widest sense. This has been achieved through regular Quality Seminars, inviting ideas and suggestions from everyone. Also, mild propoganda campaigns have been mounted to create in everyone's mind an identity with quality and pride in the job. It is absolutely essential

to maintain this momentum with regular if not continual re-appraisal in order to reap the real benefits of Quality Assurance and Quality Management. This message has been spread company wide and is not project specific.

38. One fundamental change that would be made on future projects is the placing of responsibility for quality control inspection with those carrying out the work and not retaining an independent Q.C. inspection function, because the way this has been organised at Sizewell has fragmented the responsibility which in turn has detracted from achieving the objective of total involvement by everyone.

39. The experiences with off-site suppliers and sub-contractors has led to improvements in the Management of those companies by Laing at Sizewell but I doubt whether those companies will take the positive action necessary to improve their own performances on a lasting basis.

40. It is a fallacy to believe that Quality Assurance saves money. In itself it is expensive to administer and whilst it may be shown by those selling Q.A. that money invested in Q.A., is effective in reducing the cost of re-work by "getting it right first time", this maxim can only be achieved by investing time and effort to create the right atmosphere to attain responsiveness from all levels of staff and operatives. Quality Management in its very broadest sense will achieve cost savings and even then it requires a dogged commitment to continual review and re-appraisal together with the will to make changes where necessary.

41. The longer experience of Quality Assurance in the Mechanical and Off-shore sectors of Industry may lead Civil Engineering Companies to use the organisation of Q.A. in those sectors as a model. Other than for basic principles this would be wrong. In general within Mechanical and off-shore related Industries there is a high degree of non destructive testing and the application of procedures which can remain standard and unchanged from project to project, particularly

for manufactured items. The intelligence and skill levels of operatives are generally higher than in Civil Engineering Operations. This means that we must work much harder at increasing skill level and qualifications for the work to be done by operatives. This will only be achieved by the best man management and communciations which must be backed by a total commitment from the Employer to Quality Management which is standard within Laing.

17. The development of quality management techniques in civil engineering design and site supervision

A. J. COWAN, BSc, MEng, MICE, LicIQA, Williamson (QA) Ltd

SYNOPSIS. Quality assurance techniques have developed considerably over the past few years, and in particular the application of Quality Management within a service industry such as civil engineering has evolved into a well developed management control system.

James Williamson and Partners were the first members of the Association of Consulting Engineers successfully to achieve third party accreditation with BSI for civil and structural design. The initial system was generated for work on Torness Power Station in the early 1980s, and the firm has, in the last ten years, developed a flexible quality system to effectively control and co-ordinate complex projects.

This paper discusses the use of quality assurance techniques in a civil engineering design office, looking at a few areas where the requirements of BS 5750 and BS 5882 are difficult to interpret. Some observations are also made on the adoption of QA for site supervision.

DEVELOPMENT IN MANAGEMENT TECHNIQUES

1. Managing large numbers of people on complex projects can be a problem. It is not a new one however, and records show that in 3,000 to 4,000 BC the King of Babylon had problems managing his slaves. This resulted in a number of rules being written down, 282 of them in fact. One principle being that "when a builder builds a house, if that building falls down and kills the owner then the builder shall be put to death". This is a fairly severe form of management that obviously had its benefit at that time.

2. In 1513 Nicolo Machiavelli announced two approaches to management. Firstly, the love approach: Make use of regard and respect and by treating people properly you receive a good job. However his second approach became far more popular and that was "Rule by Fear". There are many instances even in today's society where that approach still would seem to apply.

3. When the industrial revolution came along there was a spirit of innovation that lead to inventions. These inventions resulted in factories and they in turn required

a need for management of people, equipment and materials. The management that developed was limited predominately to incentives by piece rates or bonus systems, with a work force left to make their own decisions about duration and methods. This philosophy also still has some hangover in today's construction industry where bonuses are paid to labourers, and to some tradesmen, to meet targets.

4. In the 1880s Frederick Winslow Taylor laid down some rules that related specifically to the employee and to the employer. He noted that the employee had to accept that the production process would be determined scientifically by management and that it was not the employee's place in life to influence the production process. He was there to work. The employer should determine the duration and method for each task and should set up a suitable organization to take all responsibility from the worker, except that for the performance of the job. He should develop a science for each operation to replace opinion and rule of thumb methodology.

5. Elton Mayo noted in 1924 that the efficiency of production is not only dependent upon the physical environment or the adopted processes of work. One can more logically attribute an increase in efficiency to a betterment of morale than to any other factor. In other words, a happy work force was a good work force.

6. In 1937 Ludwig Von Bertalanffy, who was a biologist, realised that a group of people working on a project was very similar to an organism. Organisms tend to strive for a steady state of equilibrium. Any change to the state of the organism affects the equilibrium and any change to the environment has the same effect.

7. All of these theories highlight particular aspects of of management with which we are all familiar, and can recognise within the construction industry. However, the techniques of Quality Assurance were developed primarily for the production industry where the same item is produced over and over again and translation of these principles to a service industry can be confusing. This has lead to examination of the principles of a Quality Management System as laid down in British Standards and to the adoption of trial and error techniques to establish those principles within the industry.

8. One of the principles in Quality Management is "Get It Right First Time". Unfortunately, the one thing that you cannot get right first time is the Quality Assurance System. It has to be developed gradually to take account of the ways in which the organization operates. Many companies initially cannot see how to apply quality assurance to the way they work. Many of these problems are self induced and very often the problems of implementation are due to the fact that management does not supply the same management rules across the whole company. They start off from the wrong basis, applying techniques because the

client requires them for a particular project, rather than because the company wishes to improve the quality of management.

9. Project based management works to a limited extent but the benefits of quality assurance are really only seen when applied in total across the company and a philosophy of better management is introduced. In the early 80s when quality assurance was first introduced by James Williamson & Partners to their methods of working, it was difficult to appreciate how to apply the principles to design activities. Site activities being performed by the contractor were prescribed and monitored, and a programme of site inspection and testing was established for each work element. Processes and procedures could be written down and followed for those activities, however the conceptual elements of design were not addressed. Much of this was due to misunderstandings, and a great deal of bad publicity about quality assurance which was around at that time. This gave rise to preconceived ideas of what Quality Management was all about. In fact these ideas were quite wrong and have taken a long time to overcome. Many people today in the construction industry still do not understand the aims of quality management and the benefits that can be achieved.

MISCONCEPTIONS

What is Quality?

10. Quality is defined in BS 4778 : Part 1, as being "The totality of features and characteristics of a product or service that bear on its ability to satisfy stated or implied needs". What does this mean? It is not the usual idea that quality reflects luxury or a higher specification for a product e.g in the motor industry, a Mini compared with a Jaguar - both are quality products. They both meet the needs of their specifications. The applicability of this definition for construction has been, and still is, a subject of much debate within the industry.

11. What is an implied need and how does a designer review and implement stated needs? Certainly for technical work stated needs from the client are quite often not clearly defined.

What is Quality Assurance?

12. Quality Assurance is defined in BS 4778 : Part 1 as being "All those planned and systematic actions necessary to provide adequate confidence that a product or service will satisfy given requirements for quality". This can be simplified as "meeting the specified requirements for quality". i.e. Complying with the brief, specification, conditions of contract etc., and being able to give confidence that those requirements have been met.

13. Perhaps this can be summarised as:

(a) having a written set of documents that state what you do and how you will achieve the specified requirements,
(b) doing what you say you will do,
(c) keeping records of what you do,
(d) being able to prove that you did in fact do what you said you would do.

14. In Part 0 of BS 5750, the guidance given is that an organization should seek to accomplish the following three objectives with regard to quality. Firstly, the organization should achieve and sustain the quality of the product or service produced, so as to meet continually the purchaser's stated and implied needs. Secondly, the organization should provide confidence to its own management that the intended quality is being achieved and sustained. Thirdly, the organization should provide confidence to the purchaser that the intended quality is being, or shall be achieved in the delivered product or service provided, when contractually required. This provision of confidence may involve agreed demonstration requirements.

<u>What Quality Assurance Is Not</u>

15. Quality assurance is not quality control or inspection. The emphasis in the philosophy of quality assurance is in using the correct information first time, so that whatever is produced is correct. Whereas quality control and inspection tend to check and test items after they have been constructed or designed so that if there is a mistake the work has to be re-done.

16. Quality assurance is not a super-checking activity. The level of checking should be such that requirements are met, and no more. It does not automatically mean that everything has to be examined by another engineer. Quality assurance is not a technical check. The process is a management tool for administrative and management purposes. Technical matters are dealt with by technical staff at the appropriate level.

17. Quality assurance is often blamed for generating paper. QA is not a paper generator. The paper generated should only be sufficient to demonstrate the ability of the company. Massive paper generation is in fact often due to a lack of planning at the start of work, or a lack of specification in the traceablity that is required throughout a project. On power station construction there is often a very high degree of traceability required. Records of testing, sampling, construction and use of materials all must be retained for later reference. This is not a direct result of quality assurance. It is a result of the client requiring information to be retained for later use. The only records that are required through quality assurance are those to demonstrate that the organization has carried out it's work in accordance with

the rules. These should be concise and should not generate any paper that is not necessary for the essential management of the company.

18. Quality assurance is often seen as a major cost area. Initially, I would agree that is the case. Organizations have to spend time and money looking at how they operate to set up the management system. But in the long term, the activities involved in quality assurance management are no different to those which engineers should be carrying out in day-to-day project management. Quality assurance is implemented by everybody and does not require a large team of specialist engineers headed by a QA manager in a little empire of their own.

19. Finally, quality assurance is not the cure for all ills. An organization with a quality assurance system will still make mistakes. The design work will still be subject to technical checking. The level of accuracy produced by engineers will not change.

20. What will change, is the ability to use time and cost effectively. The advantages of proper planning and co-ordination take the principal role in activities, so that information is correctly used and distributed. This will result in fewer mistakes due to misinformation.

THE DESIGN PROCESS

General

21. Utilizing a quality management system in the design office requires particular attention to be paid to certain areas of the British Standard (BS 5750). Whilst all clauses have to be satisfied, there are some which require closer attention and these are discussed below.

Management Responsibility

22. Whilst the requirements of clause 4.1 apply in all cases. Particular recognition must be made of clause 4.1.2.2. "Verification, resources and personnel".

23. It states here that verification must take place of all activities that are quality related. This includes design work, report writing, document receipt, issue and modification, and receipt and storage of goods. Definition of the activities that are quality related is important. The activities are verified by audits. The difficult application here is in deciding which activities are quality related. In order to achieve a satisfactory result in this area the organization must have set down clear aims for the quality system. The levels of quality that are required in different aspects of work must be clearly defined. This allows verification of quality related activities to be identified, and for staff to understand the levels that are required.

Contract Review

24. Here the British Standard defines the need for the supplier (the consulting engineer or contractor or sub-contractor) and the purchaser (the briefing authority, client) to ensure the following:

25. Firstly, the purchaser's requirements have been adequately defined and that all relevant information is available to carry out the works. Secondly, adequate resources are available to carry out the work. Thirdly, any changes to the contractual requirements are identified and reviewed. Fourthly, the information passed to other parties is complete, current and adequate to carry out the works.

26. This is all fundamental to any construction project. The brief has to be properly interpreted. Agreement of the interpretation should be sought and all parties must understand what is happening. The essence here is planning with everyone in the project taking time to prescribe actions and generate discipline through documentation. It is a good discipline in which to become familiar. All of us think, no doubt, that we do this. But how many of us document it thoroughly, such that we can demonstrate that we have planned our work?

27. An important area to be considered is how to handle changes as the job develops. The implications of those alterations or changes must be looked at, not only in the light of what is happening at that particular time, but in the overall planning of the project. Engineers need to sit down and review the requirements regularly to ensure that the Client's specification is continually being met.

DESIGN CONTROL

28. Many will argue that quality assurance of Design Control is difficult to achieve since each time design work is carried out the conditions are different. The ground conditions and the geology are different. However, the management process to control information to be used in any design can be prescribed, and a standard process can be established. These can be effective without hampering the innovative aspect of design work.

29. The designer is required to draw up plans and programmes that identify the design activities. The work shall be planned such as it is carried out by a suitably qualified group of persons with adequate resources. All interfaces between the groups shall be identified and the flow of information documented and regularly reviewed. In other words, time shall be taken to plan the project. This applies whether the job is a small piece of remedial work, or a design of a new power station.

30. The benefits of writing this information down are fairly obvious. In many instances the designer does not know the most economic solution from day one and several sketch designs may have to be carried out. It is important

that the degree and depth of study required is understood by those on the job, such that graduates for example, don't undertake complicated finite element analysis, spending a lot of time and money, when perhaps only an outline design was required. If instructions are not written down people can misinterpret requirements.

31. Design input data which will be used in a design must be reviewed prior to being adopted in the design. Areas where inadequacies appear should be identified and resolved. This is all too often ignored and people go into design without considering what information they require. Typically, this can often occur (or not occur, as the case may be), at site investigations. The site investigation may be designed and planned with very little reference to the designer's requirements. In situ tests are carried out, samples are taken and tested in laboratories, boreholes are taken across the site to meet a site investigation programme, providing a comprehensive set of results. But how much of the information is actually used in the design process. Perhaps the information that the designer actually requires has not been collected or the appropriate tests have not been taken.

32. Where was the planning and the interface between the design team and the site investigation team? What controls were in place to ensure that the Client's money was effectively used to provide information to design the works? It all hinges on effective communication.

33. At the design output stage a QA system should ensure the procedures are developed to cover the production activities of calculations, drawings and reports. To ensure that requirements are met, acceptance criteria need to be identified such that any design produced conforms to requirements. The characteristics of the design that are crucial to safe and proper functioning of the end product are identified.

34. The design work should be verified to ensure output meets input requirements and the verification process should comprise one of the four activities:-

(a) holding and recording design reviews
(b) undertaking qualification tests and demonstrations
(c) carrying out alternative calculations or
(d) comparing the new design with a similar proven design if available

35. It is probable that design verification can most commonly be carried out by holding design reviews although alternative calculations in comparision with similar proven designs, such as full scale pile tests, could be used.

36. What happens with design changes? Often, this is the area where control disappears. The first thing an Engineer wants to do when notified of a change is pick up the rubber or the razor blade, scratch out the old detail and change it. 'Let's get is sorted and out the door, back to the

Client, as quickly as possible". However, this is where it is most important that control is applied. The British Standard quite rightly requires the design changes are initiated, reviewed, approved, implemented and recorded. BS 5882 requires that design changes are reviewed by the body who did the initial design. It is only common sense that people should not change drawings or reports without checking thoroughly that the change is in fact valid. These elements are fundamental to design work whether it is in a Consultant's office, or part of a Client organisation, or part of a Contractor's organisation. All the above features should be considered on any project no matter it's size and yet, how often are they forgotton about. A system/procedure for identification, documentation review and approval of changes is basic to good business practice.

THE BENEFITS

37. Operating a Quality Assurance system is not easy. It is a new concept to the construction industry and one which requires work and commitment for its success. But there are benefits to be gained. Personal experience of operating the system shows that the benefits are there to be achieved.

38. Firstly, control of work is improved. Particularly when there is pressure of time or budget and management can be assured that work is still carried out in accordance with designated procedures.

39. The control of information is improved, so that incoming details can be shown to have been passed to those who should have received them. The control of information retrieval is also improved and much less time is spent searching for information.

40. Mistakes are reduced because staff become better informed as a result of the more controlled information system. Where mistakes have occurred, they can be tracked after the event and the proper actions taken.

41. Overall efficiency improves because all of these areas of inefficiency and mistakes are reduced and therefore, in-house costs and overheads are reduced as the QA system takes effect. Typically, engineers see quality assurance as a great hindrance when it is introduced. But a recent survey of staff within James Williamson & Partners has shown that the majority of technical people in the organisation are now in favour of quality assurance systems.

42. Staff recognise the benefits, although they do still think there is room for improvements. This is not surprising, as the technical staff have been trained to question things and to look for ways of doing tasks more effectively. Comment/criticism of the system from technical staff is valuable feed back in developing the system. The QA system provides documentary evidence to permit self examination by management. In itself that is

valuable feedback. The essence of a quality assurance system is for it to provide opportunity for a critical look at the management organisation, to improve it and to make it more cost effective. There is now an encouraging response to quality assurance in the industry and as systems are adopted by consultants and contracting organisations there will be a better understanding of what the Client requires. Fewer mistakes will occur in the progress of the works and civil engineering clients will be assured that Engineers manage the project and money sensibly and economically.

SITE SUPERVISION

43. Very often the areas that cause concern in the construction industry are the interfaces where the project develops from one stage to the next. From site investigation to design. From design to construction. The implementation of quality assurance in site supervision can ensure that design requirements are properly interpreted in the construction phase, and any last minute decisions do not wrongly influence the construction process. However, all too often when you look at the systems adopted on site you find that the supervising engineer relies heavily on the contractor's Quality Assurance System. This is quite wrong. Just as the designer has a quality system for the management of his information through the design process, it is up to the supervising engineer to develop a management system for site supervision. It should not impinge on the contractor's work. The system should document and demonstrate how he is carrying out his duties on the site. The engineer should have procedures on all his activities specifically relating to the project. These should demonstrate how he supervises tasks. The difference between supervision and survelliance should be clear and the system should be able to demonstrate which inspectors have supervised which areas of work.

44. Again, planning of activities is important and the system should be used to demonstrate that the engineer is in control of the supervision process such that information on aspects of design are being properly communicated to all parties on site. Plans and documents require to be drawn up to show how this will be done. These can be prepared in association with the contractor's documents so that his supervision techniques are built around the construction process. Neither the contractor nor the supervising engineer should have to change the way they operate to suit the requirements of a quality system. The system should reflect the way they carry out their work.

45. The theory is perhaps easy, it is far more difficult to put the above into practice, particularly when time is of the essence and everyone is trying to get the technical aspects correct. A fault perhaps in the construction

industry is that we are too busy engineering and do not spend sufficient time managing.

46. If Quality Management Systems are to be successfully adopted throughout the industry, we have to recognise that management is more than a paper pushing exercise. Successful management depends on efficient administration combined with the perception that time taken to plan is time well spent.

47. The main stumbling block is still in our attitude to management techniques and perhaps too little time and effort is taken to train engineers to manage.

48. The increase in the use of quality assurance throughout the industry has certainly induced an awareness of the need to improve and perhaps once we all stop talking about it and try to implement these techniques we will not only be good engineers but good managers.

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18. The development of quality management techniques for independent pre-commissioning design appraisal, inspection and test

M. J. GELDERD, BSc, MIQA, MISIN Quality Management Ltd

SYNOPSIS. The Independent Inspection Agency for the Primary Containment Structure for Sizewell 'B' Nuclear Power Station was appointed in 1987. To manage and control its activities the Agency was required to establish and implement quality management systems that conformed to the requirements of BS 5882 : 1980.

INTRODUCTION

1. The design of Sizewell 'B' Nuclear Power Station is based upon the design known as SNUPPS - (Standardised Nuclear Unit Power Plant System) which was developed by Bechtel Power Corporation using the ASME Boiler and Pressure Vessel Code Section III.

2. The safety case for Sizewell 'B' presented by the Central Electricity Generating Board (now Nuclear Electric plc) in its Pre-Construction Safety Report (ref. 1) to HM Nuclear Installations Inspectorate (NII) included a commitment to subject selected items to independent pre-commissioning design appraisal, inspection and test.

3. The items nominated for independent appraisal included components of the primary circuit system - for which failure is deemed (by design) to be incredible and the primary containment structure.

4. The extent of these independent services is subject to specification and contract between the individual Independent Inspection Agencies (IIAs) and Nuclear Electric.

5. The Agencies employed by Nuclear Electric are the UK equivalents of Authorized Nuclear Inspection Agencies required by the ASME Code Section III, Sub-Section NCA (ref.2) for Certificate of Ownership in the USA and Canada.

6. The requirements for employment of the Agencies are defined in a document, prepared by Nuclear Electric, which is an adaptation of the ASME Code Section III (ref. 3). This adaptation was

developed for reasons that the ASME Code relates to institutions and practices which are specific to the USA and Canada and is, therefore, restrictive to users outside that area.

7. The method of employment of the Independent Inspection Agencies is unlike the situation in the United States where each organisation performing ASME Code activities is required to hold a valid agreement with an Authorized Inspection Agency (AIA) as one of the requirements for their Certificate of Authorization.

IIA FOR PRIMARY CONTAINMENT

8. The Civil Engineering Branch of the Central Electricity Generating Board, Generation and Construction Division located at Barnwood, Gloucester, were appointed in 1987 to act as the IIA for the Primary Containment Structure. This group of engineers now operate within Projects Division of Nuclear Electric plc.

9. The IIA are subject to evaluation, assessment and audit by the Engineering Inspection Authorities Board (EIAB) working under the auspices of the Institution of Mechanical Engineers who have adopted a similar role in the UK to that of ASME in the USA and Canada. The IIA's organisation and capabilities have met with the approval of the EIAB, and Nuclear Electric have been granted Owner's Certification. Quality assurance is an essential element of the requirements for such certification and the specification for independent services requires the IIA to establish, document, implement and maintain a quality assurance programme which complies with the relevant requirements of BS 5882 : 1980 (ref. 4).

10. The organisation structure of the IIA is shown in Fig. 1.

11. The personnel of the IIA are required to be totally independent of Nuclear Electric's design and construction team - the PWR Project Group (PPG) - and of support provided to the PPG at all phases of the pre-commissioning design and construction programme.

12. As a result of the appointment of the IIA for the primary containment to an in-house organisation, steps were actively taken (ref. 5) to ensure that the group engaged in the IIA role were not only independent but could also be seen to be independent from other groups providing support to PPG. The measures taken have been reviewed and accepted by Nuclear Electric's Health and Safety Department, the NII and the EIAB.

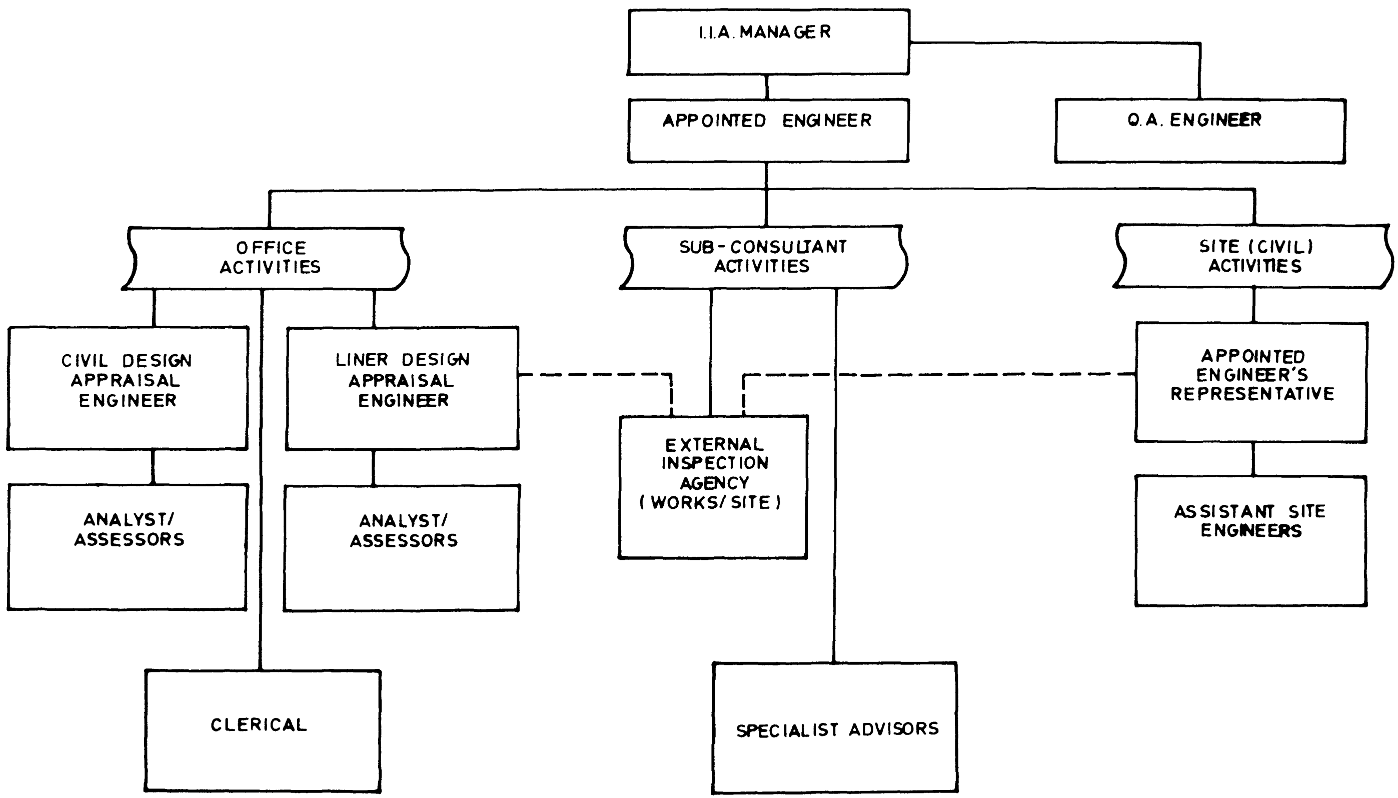

FIGURE 1: ORGANISATION CHART FOR THE INDEPENDENT INSPECTION AGENCY

SPECIFICATION FOR WORK TO BE UNDERTAKEN BY THE IIA

13. The Specification for work to be undertaken by the IIA for the Primary Containment Structure includes requirements for the following activities to be performed :

- To review and endorse the design of the Primary Containment Structure.
- To undertake, as considered necessary, independent design analysis in support of their endorsement.
- To review and comment on contractor submissions.
- To designate hold points on quality plans and to inspect against them.
- To witness in-process fabrication, erection inspection and testing.
- To endorse data reports and construction reports used as a means to certify that items comply with the intent of the ASME Code.
- To review and endorse completion testing procedures and to witness and endorse the testing.
- To endorse provisions established for In-Service Inspection.

14. In performance of their defined tasks, the IIA were also required to develop procedures to demonstrate that their inspectors were sufficiently trained and covered the same scope of work as the Authorized Nuclear Inspectors in the USA.

15. The requirements for qualifications and duties for Authorized Nuclear Inspection Agencies and personnel who undertake independent inspection activities required by the ASME Code are defined in ANSI/ASME document N626-1985 (ref. 6).

16. The IIA were required to develop adaptations of this document for the main reason that the ANSI/ASME document relates to institutions and practices which are specific to the USA and Canada and its application is, therefore, restrictive to users outside its country of origin. Furthermore, there was also a need to embrace the Sizewell 'B' Contract Specifications rather than make references to the ASME Code.

17. Two adaptation documents were developed by the IIA to cover the full scope of inspection activities to be performed by the IIA and their sub-consultants - the External Inspection Agency (EIA).

18. Adaptation of ANSI/ASME N626.2. Defines the requirements for the qualifications and duties of concrete inspectors employed by the IIA.

19. Adaptation of ANSI/ASME N626.0/N626.2. Defines the requirements for the qualifications and duties of "mechanical" inspectors employed by the EIA. This second document is a hybrid of Parts 0 and 2 of the ANSI/ASME document. Part 0 defines the requirements for Authorized Nuclear Inspectors and essentially covers mechanical and welding activities and Part 2 defines the requirements for Authorized Nuclear Inspectors (Concrete). So, this second adaptation was prepared extracting those duties relating to the fabrication of metallic components, which are included in the concrete inspectors' duties in the ANSI/ASME document, and combining them with the mechanical inspectors' duties defined in Part 0.

20. Thus, unlike the ASME Authorized Nuclear Inspectors (Concrete), the IIA concrete inspectors are not responsible for monitoring welding and associated activities as these are undertaken by the IIA's sub-consultants the EIA.

QUALITY ASSURANCE PROGRAMME

21. The IIA have established, implement and maintain a quality assurance programme which complies with the relevant requirements of BS 5882 : 1980 (ref. 4).

22. The scope of of BS 5882 does not specifically include the services provided by independent third party inspection bodies, but the process of its application has been broadly similar to that adopted by other service organisations implementing quality management systems.

23. In the development of its quality assurance programme, the IIA had to pay particular attention to :

(a) Measures to preserve its independence. The IIA's "independence" as an in-house organisation is required to be demonstrable.This has been achieved by applying appropriate procedural controls to ensure that all its staff are totally independent of project support work to Nuclear Electric's design and construction team during all stages of the IIA's work. The adopted polices and procedures have been extended to include all the IIA's advisers,from whom written assurances of their independence have been obtained.

(b) Training. Training was considered to be of fundamental importance to the acheivement of success and effectiveness of the work undertaken by the IIA. In this respect a training procedure was developed which defines the minimum requirements for experience, train-

ing and qualifications of all its staff. Provisions were made to include indoctrination in the implementation of the IIA's Quality Assurance Programme and procedures and familiarisation with key documents such as the ASME Code and its UK adaptations (refs. 2, 3 & 7).
Training needs are reviewed annually to confirm that staff continue to meet project requirements, and the records maintained serve to demonstrate achievement of the specified requirements either by demonstrable experience or by attendance at suitable training courses.

(c) Design reviews. Review of the design of the primary containment structure is one of the prime activities of the IIA. Thus in order to manage and control this work a series of procedures and work instructions were developed to define how the work was to be undertaken.
Controls were provided for the review of Design Reports, Specifications, Design Drawings, Contractor Submissions, Non-Conformance Reports, and any other documents required by the IIA to endorse the design, as well as for the control of activities undertaken by the IIA in support of their design reviews, eg., preparation of calculations, reports, verification and validation of computer codes and independent design analyses.
Further controls were also provided to establish responsibilities for the IIA endorsement of design documentation and to provide tracking mechanisms to monitor the timely certification of the design and clearance of any caveats.

(d) Inspection and surveillance activities. This is another of the prime activities of the IIA which is controlled by a series of procedures and work instructions. Of particular relevance was the need to provide controls, and the necessary training, to ensure that the duties of Authorized Nuclear Inspectors, as discussed previously, were effectively executed.
In addition to general surveillance, the IIA specify those activities for which they require prior notification for attendance including, as considered necessary, the stipulation of Hold Points which are indicated on the Contractors' Quality Plans. The level

of inspection applied by the IIA is influenced by a number of factors which are, in part dictated by the requirements of the IIA Specification, and in part by the nature and complexity of the activities concerned. The IIA have the ability, if considered necessary, to issue Non-Acceptance Reports when, they have observed any item or activity which does not meet the Specification. It is mandatory that any Non-Acceptance Reports so issued are satisfactorily resolved prior to obtaining IIA endorsement.

(e) Contractor QA Programme Monitoring. The IIA monitor contractor and sub-contractor QA Programmes in a number of different ways. IIA procedures provide arrangements for the review and comment on contractors' QA Programmes, day-to-day monitoring by their inspectors and the conduct of audits.

(f) Records. The IIA maintain records to demonstrate compliance with its QA Programme and to substantiate endorsement of the design and associated hardware.
These records include :
- reviews of the design and design changes
- reviews of contractor submissions
- IIA procurement records
- inspection and surveillance reports
- IIA non-acceptance reports
- audit reports
- training and qualification records

COMMUNICATIONS

24. All formal communications from the IIA are addressed to Nuclear Electric's design and construction team - the PWR Project Group (PPG) - for action.

25. For reasons of expediency contractors' technical submissions for off-site work are made concurrently with those to the PPG. The IIA response, however, is made to the PPG.

26. Monthly reports are provided to the PPG detailing the results of all IIA on-site and off-site inspections including any non-acceptance reports which have been issued to the PPG within 48 hours of their occurence.

27. Meetings are held between the IIA and HM Nuclear Installations Inspectorate twice per annum to discuss general progress and the results of IIA activities.

COMPARISONS BETWEEN US AUTHORIZED NUCLEAR INSPECTION AGENCIES AND THE IIA FOR SIZEWELL 'B' PRIMARY CONTAINMENT STRUCTURE.

28. There are a number of differences between the US and the UK approaches to Third Party Independent Inspection in nuclear power plant construction and some of these differences are now considered.

29. Terms of Reference. The objectives of US Authorized Inspection Agencies (AIAs) and the IIA are broadly similar :

- To monitor as a third party, the participating organisations' implementation of their QA Programmes and compliance with the Specification.

30. The responsibilities and duties of AIAs are specified in the ASME Code Article NCA-5000 which in turn references ANSI/ASME document N626-1985. The responsibilities and duties of the IIA, however, are the subject of a procurement specification prepared by PPG and which is based on ASME requirements. The need for a common standard is more obvious in the US where there are multiple users of this service.

31. Method of Engagement. AIAs under the ASME regime are engaged by each organisation performing Code activities. The IIAs for Sizewell 'B' are employed by the Owner's Representative - PPG - who have contracted two agencies to cover the Primary Circuit and the Primary Containment Structure. Items purchased from the US, however, are covered under the ASME system.

32. The manner of employment on Sizewell 'B' has reduced the potential number of inspection agencies and has also affected communication routes. In both the US and at Sizewell the Agencies formally correspond with their employer, in US it is the organisation performing Code activities - the Contractor, whereas on Sizewell 'B' it is the Owner's Representative - PPG.

33. Agency Qualifications. AIAs are designated by, or are acceptable to, the appropriate legal authority of one of the cities or states of the US or province of Canada that has adopted the Code.

34. AIAs must either be licensed to write boiler and machinery insurance in the US or Canada, or be the Enforcement Authority for a state, city or province of Canada.

35. The IIAs for Sizewell were assessed by the Owner's Representative and were selected on the basis of their ability to meet the prescribed requirements.

34. Personnel Qualifications. The experience and qualifications required by Authorized Nuclear Inspectors (ANIs) is defined in ANSI/ASME document N626-1985. The qualification of ANIs is administ-

ered by the ASME Board against these requirements.

35. Experience and qualification requirements for IIA Primary Containment inspectors are based on the ANSI/ASME document and the arrangements are a matter of internal administration.

36. Duties. The ANI's duties are defined in ASME Section III article 5000 and ANSI/ASME N626-1985. The IIA undertake equivalent tasks to those described in the ASME documents.

37. The allocation of work to IIA staff differs from that in AIAs such that :

- IIA concrete inspectors are not responsible for monitoring the fabrication of metallic components which are covered by the EIA mechanical inspector.
- Design assessments are performed by IIA Head Office staff and not the inspectors.
- Formal Contractor QA Programme monitoring is performed by the IIA's full time QA Engineer.

38. In the US AIAs are required to participate in the ASME surveys of all organizations for which they provide authorized nuclear inspection, and they review and accept any modifications to the Certificate Holder's QA Programme. The IIA, on the other hand, review Contractors' QA Programmes and provide comment, as necessary, but the approval of QA Programmes is given by the Owner's Representative.

39. Quality Assurance. AIAs are required to establish, document, implement and maintain an internal programme to provide assurance that the requirements of ANSI/ASME N626 are fulfilled.

40. The IIAs are required to establish, document, implement and maintain a Quality Assurance Programme which complies with the relevant requirements of BS 5882 and which is subject to audit by the Owner's Representative (and also by Nuclear Electric's Health and Safety Department, the Engineering Inspection Authorities Board and HM Nuclear Installations Inspectorate, should they so require).

41. Internal Audit. AIAs employ ANI Supervisors to supervise and audit the activities of ANIs within their employ and their duties are defined in ANSI/ASME N626. The IIA employ an " Appointed Engineer " and an " Appointed Engineer's Representative " (at site) to perform supervisory tasks, and a QA Engineer to audit all aspects of the IIA's QA Programme including the implementation of the inspectors' duties.

CONCLUSION

42. The structured approach of control by employing quality management techniques as adopted by the IIA for Primary Containment, provides valuable added assurance in the achievement of the desired quality, and a sound basis for monitoring performance in operation through review and endorsement by the IIA of in-service inspection proposals using the knowledge and experience gained during design and construction phases.

ACKNOWLEDGEMENTS
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Discussion

R. CROWDER, Taywood Engineering Ltd, Southall
Mr Haste has rightly drawn attention in his presentation to the importance of good management, planning, communication and operation motivation in achieving high-quality construction. Of course many reputable contractors have been putting these into practise for many years, long before the advert of QA, and many buildings and structures of undoubted high quality have resulted. Are we to conclude, therefore, that much of the vast bulk of paperwork that is associated with QA contributes little in the way of additional quality. If this is not the case what aspect of the Sizewell construction has had quality enhanced by the QA procedures in force?

N. THORNTON, W. S. Atkins Northern, Whitehaven
W. S. Atkins Northern, like Williamson, has recently been accredited by BSI. There was a noticeable change of attitude among the staff during the preparation for registration. Initially, most people's perception of Quality Assurance was 'why do we need it?', whereas by the time we reached registration there had been a complete turn around and everybody was totally committed to a regime they could identify as useful and worthwhile. What are Mr Cowan's thoughts and experience of this? What management structure has been established to develop the QA system beyond registration?

J. D. TUPPER, John D. Tupper & Associates, Coventry
Clerks of works have also performed quality control for many years. How do the contractors feel about clerks gaining registration?

L. A. STRINGER, L. G. Mouchel & Partners Ltd, West Byfleet

I welcome Mr Haste's comments regarding quality assurance and agree that QA is not the panacea to the various ills that may befall a project. Good management and properly motivated staff are also essential to the completion of projects to time and budget. Having said this, QA is very much here to stay and we must all be committed to it.

There is another aspect of QA about which there are some misconceptions: the cost. The QA establishment have been rather coy about this, and have suggested that QA pays for itself - it does not cost anything. This maybe true in the long term perhaps, unless there has been a sea change in attitudes and the approach to civil engineering, but in the short term the profession must recognize that QA costs money. If you are producing a better product with better documentation then you must expect to pay more for it. These costs must be budgeted for and client organizations should recognize this.

N. D. HASTE, Paper 16

The paperwork referred to by Mr Crowder does not contribute to quality at Sizewell, other than providing a record of verification that conformity to the standards required in the various aspects of the works has been achieved. The quality procedures have provided the discipline and formalization of practices used by a motivated workforce committed to achieving quality. Clerks of Works or Quality Control Inspectors are an integral part of the quality management process.

In response to Mr Stringer, QA is not cheap but the real savings are produced when the team is committed to enhancing quality by 'getting it right first time', minimizing wastage and being efficient in the way operations are carried out. The documentation costs little compared with the consequences of a poorly motivated team.

A. J. COWAN, Paper 17

I agree with Mr Thornton's view that just prior to certification the enthusiasm becomes infectious and very positive steps are made in operating the quality system. There is, however, a tendency for things to relax substantially after accreditation and for old habits to return. It is very difficult to maintain the momentum without continually improving operating procedures. There is a danger in this, of course, that the cost of quality increases and becomes an obsession. A company must recognize when it has reached its desired level of quality management and learn to maintain it. This can best be done through auditing and system reviews.

It is worth stating that BS 5750 requires procedures to be established for maintaining the system once it is in place. This requires management decision to be taken, recognizing the results of audits and reviewing the strengths and weaknesses of the system. System maintenance relies greatly on senior management using the system to its maximum potential. All too often this does not happen.

In reply to Mr Tupper, quality assurance techniques work particularly well at site level. It must be remembered, however, that the system should be used to demonstrate compliance with the company's own management system and not, as frequently occurs, as a claims-generating machine.

To Mr Stringer, I agree that cost is a very important factor, and very often does not get considered properly, since those promoting QA may feel that it turns people off. The cost of setting up a QA system is difficult to estimate since it depends very much on the degree of control and complexity of existing management functions. Employing a knowledgeable consultant can reduce the cost by saving effort in 'reinventing the wheel', but it must be recognized that development will cost probably in the region of one man-year.

19. Dynamic response of added pipe restraints in an existing steel powerhouse

J. H. K. TANG, E. C. RAINFORD and C. M. ALEXANDER,
Ontario Hydro

SYNOPSIS. An operating nuclear power plant is required to be retrofitted with additional piping restraints to provide, in the event of a rupture of the main-steam line, environmental protection to control room and some selected equipment located in the conventional structure. Given the significant magnitude of the loads, severe restriction on space, and real concerns with practicality of the modifications, the structural systems were optimized by non-linear dynamic analysis with full consideration of the dynamic characteristics of the pressure transients and the elasto-plastic behaviour of the material.

INTRODUCTION

1. Pickering Nuclear Generating Station A, a four-unit (4 x 540 MWe) power plant, has been in-service since 1971 (Fig. 1). Each unit is an independent power source, but uses certain common station services. The principal group of buildings consists of four reactor buildings, the powerhouse running the full length of the station (1,000 ft or 305m), and the reactor auxiliary bay which is attached to the south side of the powerhouse. The reactor auxiliary bay is a conventional two-storey steel frame building fitted around the northern halves of the reactor buildings. In addition to some reactor auxiliary systems, this building houses, at the mid point of the station between Units 2 and 3 on the second floor, the four-unit station control centre, including the control room, the control equipment room, and the shift and work assignment offices. Main steam pipes and relief valves run above the reactor auxiliary bay roof on a system of supporting steel work.

2. In response to changes in the regulatory requirements, this existing nuclear station has to be retrofitted to provide environmental protection in the reactor auxiliary bay to ensure that a rupture of the main-steam line does not pose an unacceptable risk to essential safety functions of the plant (Ref. 1).

3. Specifically, added piping restraints and barriers will be required to provide the following:

(a) Protection of the Control Room, Control Equipment Room and their ventilation equipment from pipe impact, sustained loads and jet impingement loads, at designated locations

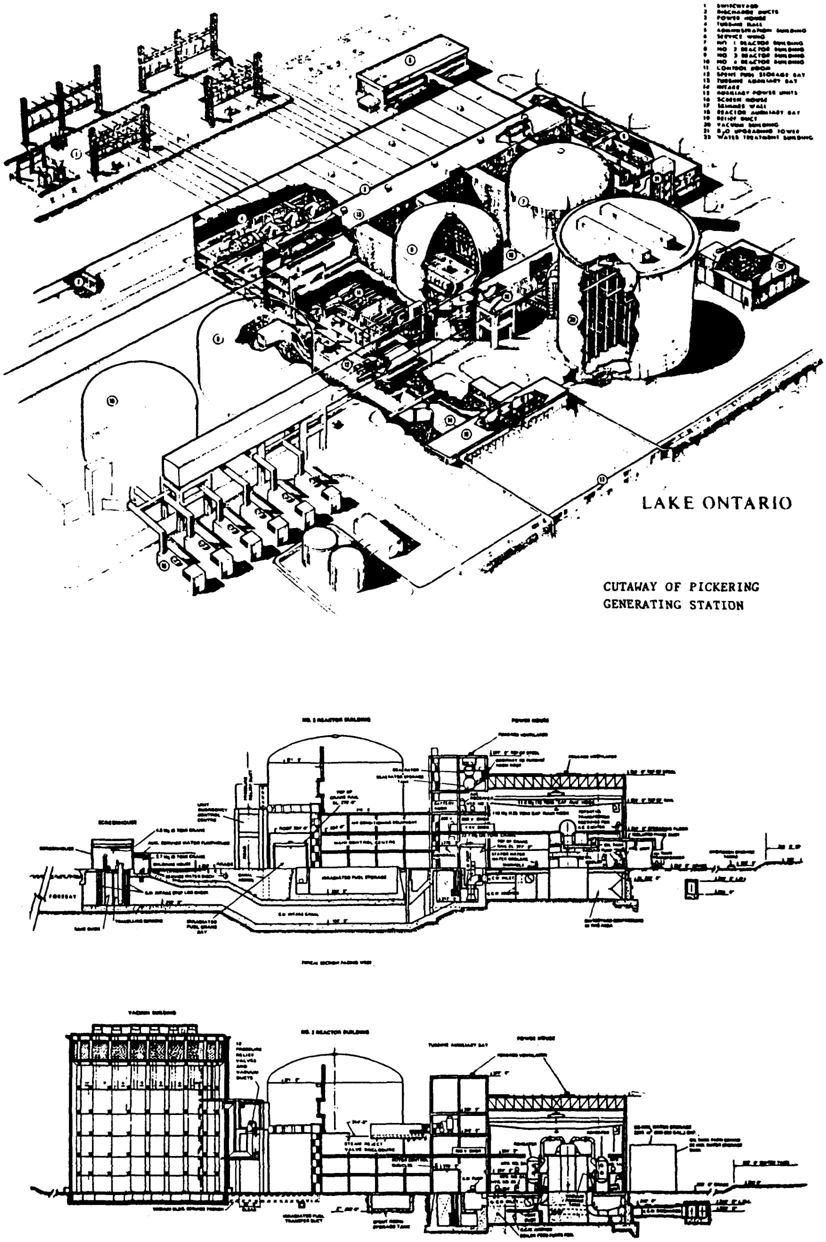


Fig. 1. Pickering Nuclear Generating Station A.

in Unit 2 and 3.

(b) Protection of the Unit 1 Class II cable enclosure from pipe impact, sustained loads and jet impingement loads at designated locations in Unit 1.

These locations are summarized in Fig. 2.

4. Due to the fact that the referenced areas are occupied by existing major equipment and are designated for other defined spatial requirements, constructibility of additional heavy pipe-whip restraints is a primary concern. As a result, every effort is being made to minimize the member sizes and at the same time achieve the desired objectives.

5. To the best of our knowledge, installation of additional pipe-whip restraints in a conventional building of an existing nuclear station has not been implemented elsewhere. The purpose of this study is to identify a reliable means of realistically assessing the performance of these restraint systems which must be appended to a structure designed previously for only normal loads which are significantly less severe than the action of a time-varying load of the magnitude encountered in the rupture of the main-steam lines.

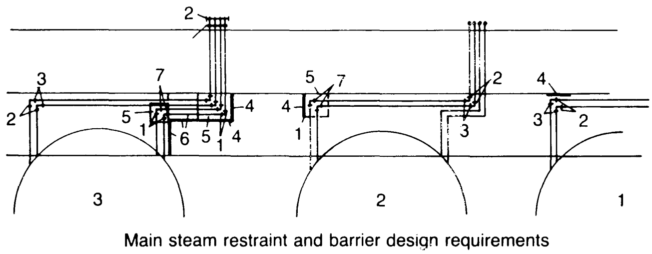
6. This paper presents a general description of the restraint structural system as well as the individual restraint structures. Subsequent sections deal with a description of the methodology, a presentation of the analysis data, the results, and discussions.

DESCRIPTION OF STRUCTURE

7. Piping restraints and barriers are required for main-steam piping routed within the reference areas in the reactor auxiliary bay. Within the specific regions, in the areas immediately surrounding the postulated break locations of the main-steam lines, the flooring has been heavily reinforced with a system of wide-flange steel sections covered with heavy plating. This system of beams is taken out to the building columns where it is integrated into the global structural frames of the powerhouse.

8. On top of the above system of beams at the identified locations are built steel structures closely surrounding the steam lines. These structures are the primary restraint system for the steam lines and are designed to arrest the motion of a single failed 28 inch (71 cm) diameter pipe, but not to interfere with its normal operating movements.

9. Typically two frames in orthogonal directions act to restrain postulated pipe breaks at either side of the pipe elbow. The two frames form the principal parts of the piping restraint structure at each break location. From the piping analysis of the main steam lines, several critical locations where pipe break would have major consequences for the environmental protection of safety systems were identified and investigated in the overall study. Fig. 3 shows the plan view and piping restraint system at one of the critical location A, and Fig. 4 shows the view at another critical break location B, both of which are identified in Fig. 2.



1. Restraint to arrest motion of a single failed MS pipe from guillotine and fishmouth ruptures.
2. Restraint to arrest motion of a single failed MS pipe from guillotine rupture only.
3. Restraint to guide motion of a single failed MS pipe from guillotine rupture only.
4. Jet impingement barrier to shield structures from direct steam blast from guillotine ruptures.
5. Jet impingement barrier to shield structures from direct steam blast from fishmouth ruptures.
6. Steam barrier to contain steam releast and direct it toward an atmospheric vent.
7. Restraint to arrest motion of a single failed MS pipe from fishmouth rupture and to guide motion of a single failed MS pipe from guillotine rupture.

Fig. 2. Main Steam Piping Restraints and Barriers.

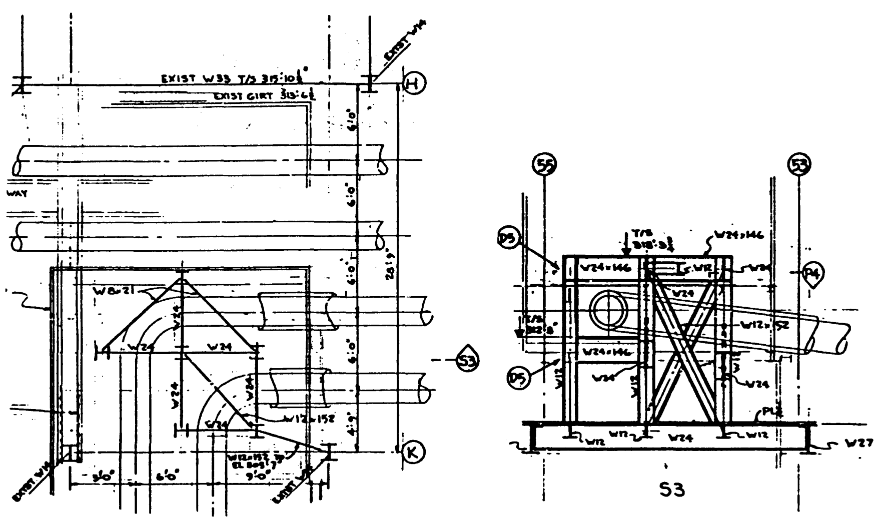


Fig. 3. Piping Restraint System at Location A.

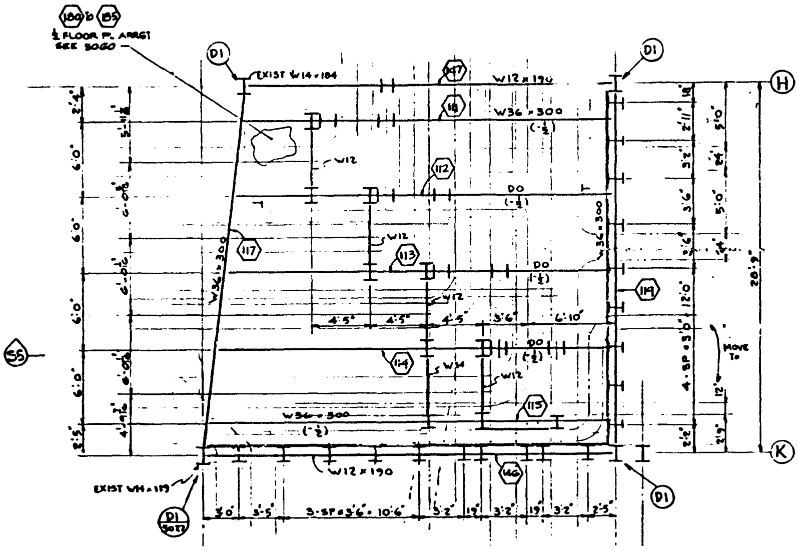


Fig. 4. Piping Restraint System at Location B.

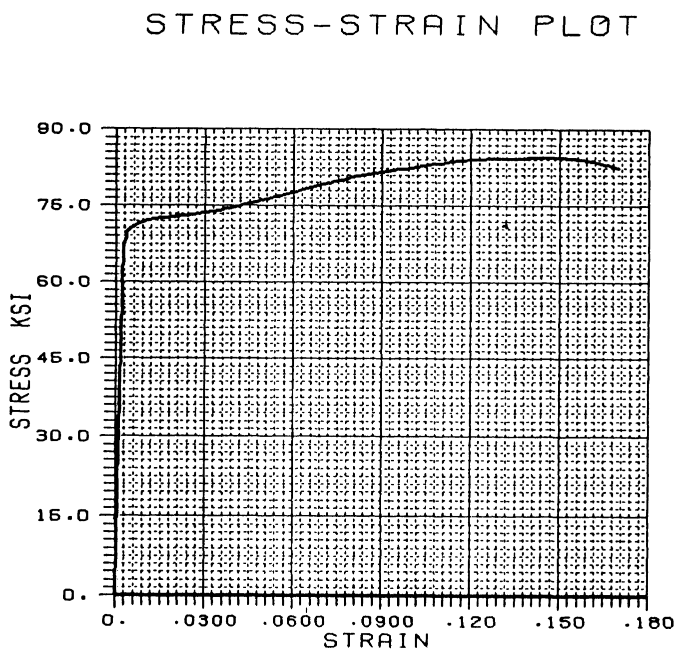


Fig. 5. Stress-Strain Curve of Steel.

Table 1. Main Steam Restraint Design Loads

Type of Rupture	Jet Thrust Load (kips)		Net Thrust Load (kips)	
	initial	long term	initial	long term
Guillotine	445	380	165 172	82(E-W) 65(N-S)
Fishmouth	790	685		

Table 2. Static Analysis Results for Frame S5 due to Vertical (Fishmouth) Load.

Member	Size	Max. Shear (kips)	Max. Stress (ksi)
1	W12X190	1023	38.9
2	W12X190	70	10.9
3	W12X190	70	4.2
4	W12X190	1023	36.0
5	W12X190	70	10.4
6	W12X190	70	3.7
7	W12X190+	505	15.5
8	W12X190+	526	15.5
9	W12X190+	2	1.5
10	W12X190+	2	1.7
20	W36X300	399	64.7*
21	W36X300	104	48.1
22	W36X300	632	58.8*
55	W12X152	0	6.6
56	W12X152	0	6.0
57	W12X152	0	10.5
58	W12X152	0	9.5

DESIGN REQUIREMENTS

10. The main function of a piping restraint is to control pipe motion due to a circumferential (guillotine) or longitudinal (fishmouth) break. The restraints should allow sufficient thermal movements but keep to a minimum the travel before impact, in the event of a break.

11. Guillotine breaks produce horizontal loads on barriers and restraints due to jet thrust from both the Boiler Side and Steam Balance Header Side. Only the net thrust force equal to the net difference in the blowdown forces between upstream and downstream jets will be exerted to the global structure.

12. As the restraints have to make use of existing steelwork for anchoring purposes, the structural floors and steam jet deflection techniques shall be utilized for thrust balancing to minimize the load transfer to the existing bracing systems of the global structure.

13. Since the pipe-whip restraints which do not serve as normal pipe supports need to be designed for one-time use only, plastic deformation is allowed (Ref. 2).

ANALYSIS DATA

14. Members of the primary restraint structures are fabricated from steel conforming to ASTM A572, Grade 60. The stress-strain curve used was supplied by the steel manufacturer (Fig. 5).

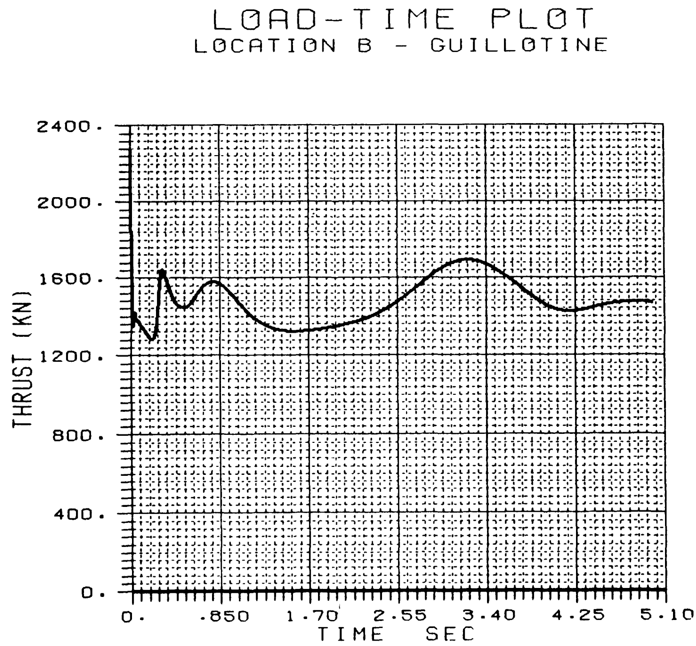
15. As mentioned above, two kinds of loadings, guillotine and fishmouth, were considered. The thrust forces from a single guillotine opening at the end of an elbow act in the horizontal plane, while the thrust forces from a single fishmouth opening at an elbow side act vertically up or down. The dynamic loadings for location B are shown in Fig. 6. The fishmouth loads are in general approximately twice the magnitude of the corresponding guillotine loads. However, the characteristics of both loads are similar: each has a high initial blowdown jet of very short duration reducing significantly before it is built up to a peak value which subsequently declines gradually in an almost monotonic fashion to a steady value in about 3 seconds.

16. Table 1 shows the peak thrust loads to be used with a dynamic load factor (usually equal to 2.0) to determine pipe restraint reaction loads if a pseudo-static approach is employed for design.

METHODS OF ANALYSIS

16. The structures were modelled and analysed by the finite element technique using the MSC/NASTRAN, a large scale, general purpose, digital computer program (Ref. 3). Typical models for restraining structure at location A and an individual frame, S5, at location B are shown in Fig. 7.

17. At the initiation of a rupture, the postulated broken pipe would impact at its restraint. Although the peak impact load is not insignificant, its effect on the restraint was not considered important since the duration was estimated to be extremely short. As a result, only the thrust loads defined as a function of pressure transient interaction were taken into account in the dynamic analysis.



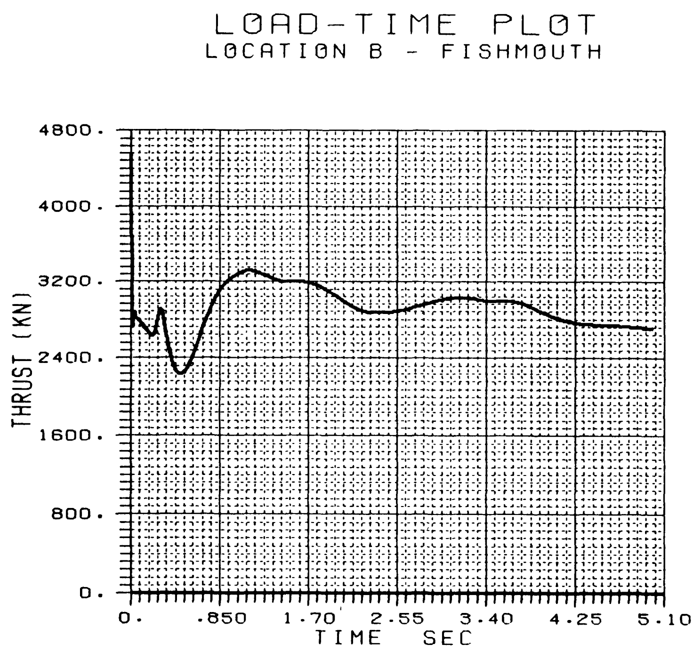


Fig. 6. Thrust Forces of a Ruptured Pipe at Location B.

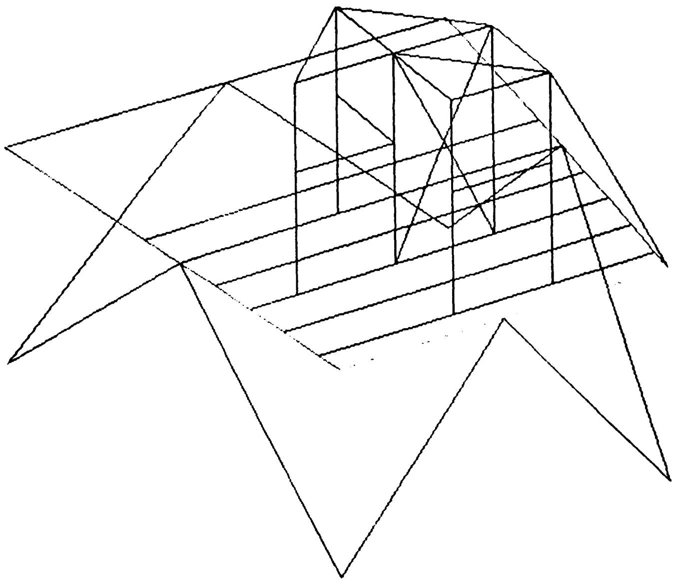


Fig. 7(a). Analytical Model of Restraint at Location A.

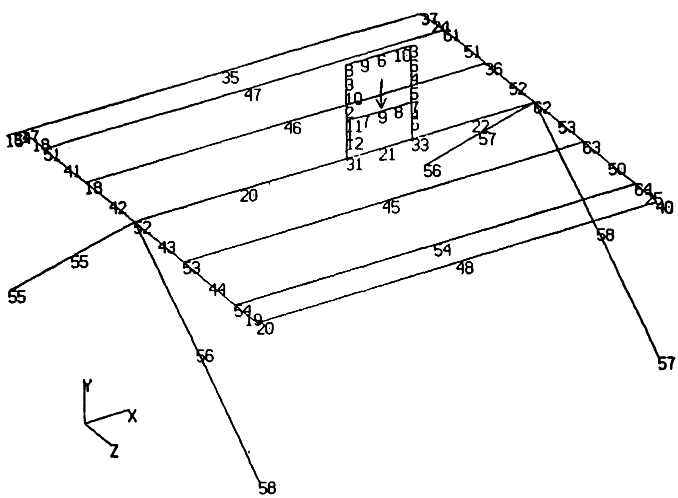


Fig. 7(b). Analytical Model of Frame S5 at Location B.

DESIGN

18. The analyses were carried out for two kinds of loading, guillotine and fishmouth. A static analysis was performed followed by a linear dynamic, normal modes analysis and, where warranted by very high stress output from the normal modes run, a non-linear dynamic analysis.

19. The loadings used in the static analyses were the starting peak values of the blowdown load. These runs were performed for reference only. The results could be used to determine the amplification factors if appropriate for other applications.

20. Each restraint was analysed dynamically by the normal modes method. The crucial output from these runs were the stresses and the shears, since these were the values used for the design of the members. If the stresses exceeded the limit of proportionality for any member, a non-linear dynamic run was performed using the given stress-strain relationship of the material.

21. It should be noted that for both the linear and non-linear dynamic runs, the solution for each time step (0.0005 sec.) of integration was obtained. However, the output was printed only at requested intervals, every hundredth time step.

SUMMARY OF RESULTS

22. For the purpose of this paper, the results of only one of the restraints, Frame S5, are selected for discussion. Table 2 gives the results for static analysis using the initial (equal to 1030 kips or 4565 kN at time 0) blowdown thrust force as the input load.

23. Table 3 summarizes the results for the linear dynamic normal mode analysis. The displacements, reactions, moments, shears and stresses from the linear dynamic run vary in a cyclic manner with a frequency of 10.4 Hz. which is close to the fundamental frequency of the structural system. The complete curves for the various quantities mentioned above, of course, are made up of the contributions from several modes of the response of the structural system to the load. Fig. 8 shows the shear and stress time histories for member 20. It can be found that the maximum stress computed is 110.3 ksi (760 MPa) which is greater than the limit of proportionality of 58.5 ksi (403 MPa).

24. Table 4 gives the results for the non-linear dynamic analysis. The computer results indicate that plastic strain first appeared at about 0.05 sec. after initial impact and remained unchanged for the rest of the run, indicating that the highest strain was attained at this stage. However, this was not typical, as maximum strain for different members occurred over a wide range of times from 0.05 to 2.0 seconds.

25. The maximum non-linear strain encountered in the whole study was 1.71% corresponding to a total strain of 1.90%. This is just short of the strain at which strain-hardening commences, and therefore is still significantly below the ultimate strain of 17%. The corresponding stress was 73.8 ksi (508 MPa).

26. The maximum displacement at node 9 (point at which the pipe-whip load was applied) computed by the static analysis is 1.24". The corresponding value from the linear dynamic analysis is 1.93". It might suggest that the amplification factor would

Table 3. Linear Dynamic Analysis Results.

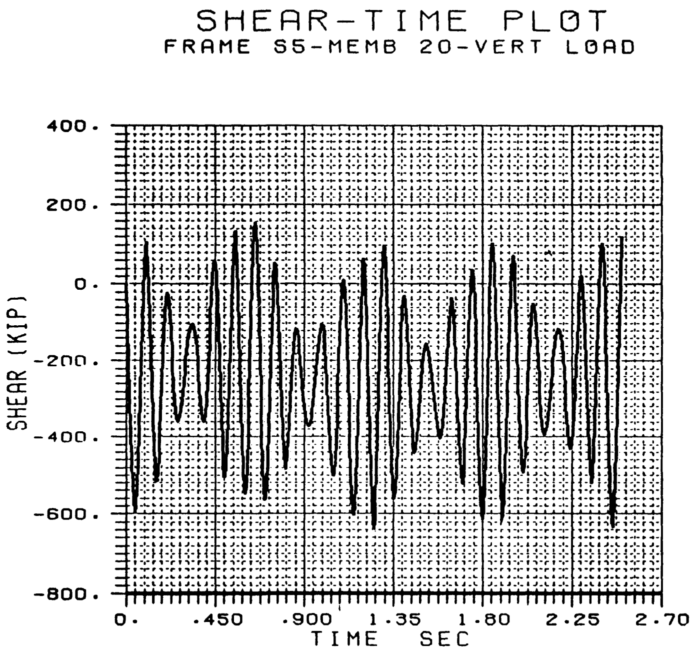
Member	Size	Max. Shear (kips)	Max. Stress (ksi)
1	W12X190	1594	79.1*
2	W12X190	212	24.9
3	W12X190	291	15.9
4	W12X190	1497	74.0*
5	W12X190	307	22.9
6	W12X190	288	21.1
7	W12X190+	770	33.5
8	W12X190+	750	35.3
9	W12X190+	212	15.5
10	W12X190+	169	16.2
20	W36X300	641	110.3*
21	W36X300	306	76.9*
22	W36X300	967	90.5*
55	W12X152	0	13.3
56	W12X152	0	12.1
57	W12X152	0	18.7
58	W12X152	0	16.9

* Values exceeded the limit of proportionality.
\+ Reinforced with 2 plates 13.38" x 0.5".

Table 4. Non-linear Dynamic Analysis Results for Frame S5 due to Vertical (Fishmouth) Load.

Member	Size	Max. Shear (kips)	Max. Stress (ksi)
1	W12X190	-	60.5*
4	W12X190	-	59.2*
7	W12X190+	-	58.5*
8	W12X190+	-	58.5*
20	W36X300	-	73.4*
21	W36X300	-	64.0*
22	W36X300	-	66.1*

\- Values not available from output.
* Values exceeded the limit of Proportionality.



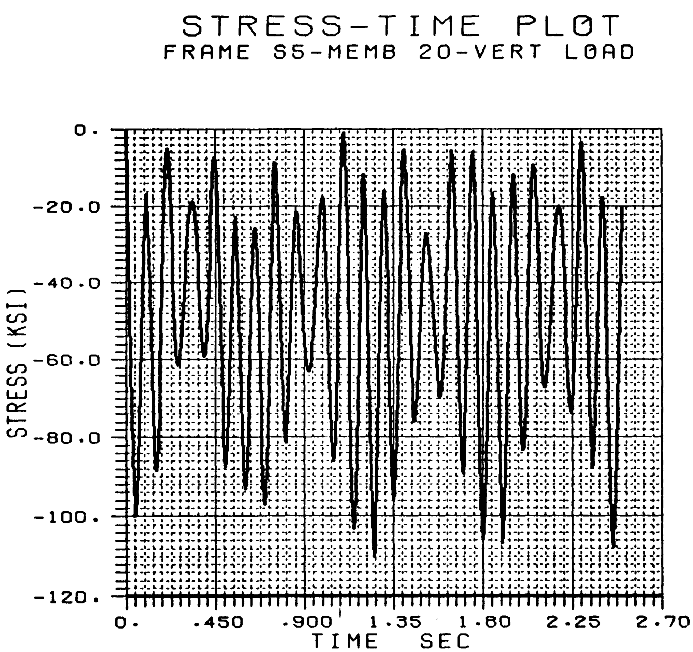


Fig. 8. Time History Results of a Typical Member.

be about 1.55 instead of 2.0. Comparison of Table 2 and 3 confirms that this factor is reasonable for the main members.

27. The real maximum displacement at node 9 was given by the non-linear dynamic analysis. It is 2.78" which is higher than the theoretical linear run since the structure must experience yielding under the severe thrust load.

DESIGN EVALUATION PROCEDURES

28. The design process involved an interactive procedures between the analyst and the designer who carried out the final design evaluation. The values that were of concern in the evaluation were the absolute maxima. In order to identify these maxima among the hundreds of results that were generated, the output from NASTRAN was copied onto a personal computer and sorted by LOTUS-123.

29. Tables of the maxima for shears and stresses were prepared for each critical frame that was analysed. In preparing these tables, the stresses from the linear dynamic run that did not exceed the limit of proportionality were included along with the corresponding shears. Where the limit of proportionality was exceeded, the non-linear stresses were used along with the shears from the linear run. This is in contrast to using all stresses from the non-linear run, where appropriate, and the shears from the linear run.

30. The enveloped maximum shear and stress for each member of Frame S5 used for illustration purposes appear in Table 5.

31. These Tables of maxima values were used to evaluate the design. Should a structure fail to meet the acceptance criteria, it was modified, re-analysed and the process repeated until a satisfactory solution was achieved.

DISCUSSIONS

32. A quasi-static analysis is obviously inadequate to apply in a case of this kind with loads of this nature and magnitude. The major difficulty is in determining appropriate amplification factor. The tendency with a static approach in a critical job of this kind is to go conservative. This is a luxury we could not afford in a retrofit situation.

33. A dynamic analysis is essential to performing any realistic assessment of the structural response to these loads. Further, non-linear behaviour must be considered to obtain valid and acceptable results. Invariably, the results from the non-linear analysis show a re-distribution of stresses, as compared with the results from the normal mode (linear) analysis. The re-distribution was such that the stresses from the highly stressed areas were reduced, sometimes significantly, depending on the level to which they had risen, while in other areas, the stresses remained roughly the same or were increased. In some cases, stresses that were previously below the limit of proportionality increased into the non-linear range.

34. The method used, within the context of the technology that is available to-day, is a valuable and reliable tool to apply in problems of this nature.

Table 5. Maximum Shear and Stress for Design.

Member	Size	Max. Shear (kips)	Max. Stress (ksi)
1	W12X190	1594	60.5**
2	W12X190	212	24.9
3	W12X190	291	22.0
4	W12X190	1497	59.2**
5	W12X190	307	22.9
6	W12X190	288	21.1
7	W12X190+	770	58.5**
8	W12X190+	750	58.5**
9	W12X190+	212	15.5
10	W12X190+	169	16.2
20	W36X300	641	73.4**
21	W36X300	306	64.0**
22	W36X300	967	66.1**
55	W12X152	0	6.0
56	W12X152	0	5.4
57	W12X152	0	6.3
58	W12X152	0	5.7

** Values obtained from non-linear dynamic analysis.

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20. Pipebridge refurbishment: cost effective seismic qualification by dynamic isolation

M. MANDZIJ, BSc, and D. S. KNOWLAND, BSc, Design Group Partnership

SYNOPSIS

The aim of this paper is to present an appreciation of the way a refurbishment problem was tackled in the first instance, subsequently the technical merits of the seismic qualification and finally the system employed to achieve the cost benefits and set targets. The set aim was to provide a cost effective solution to the problem of qualifying an existing pipebridge to an acceptable level of seismic loading. The technical solution to the problem involved the use of a dynamic isolation system, in particular the use of elastomeric bearings to limit the likely levels of dynamic interaction of a multi component system. The practical solution to the problem of refurbishment - the cleaning, strengthening and addition of the isolation system was achieved without the neccesity of providing substantial temporary works to expedite the required solution. A quality system was in use throughout to reinforce the effectiveness of the end result.

1.0 INTRODUCTION

As part of the increasing public awareness of environmental issues and the responsibilities of the Nuclear "watchdog" - HMHSE Nuclear Installation Inspectorate - the British Nuclear Fuels Sellafield site has come under additional scrutiny in recent times.

Although, Sellafield has been operating since the late 1940's, safety criteria have progressed with the available technology following an evolutionary path as our knowledge of static and dynamic mechanisms has increased over the past 40 years.

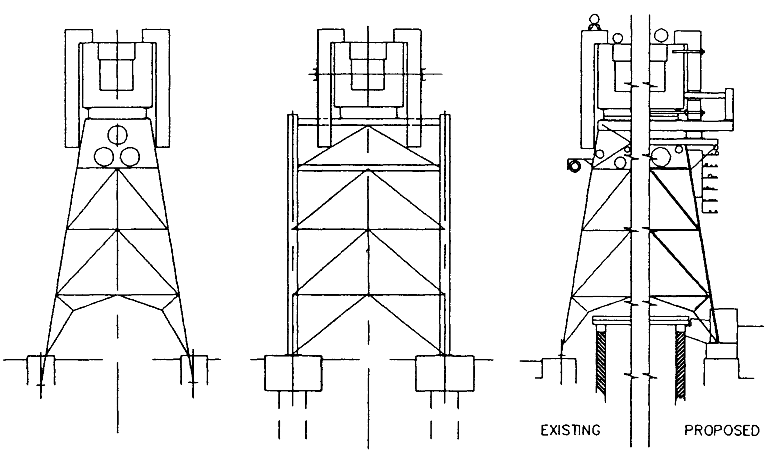
To ascertain compliance to current safety requirements buildings and plant which have been operating since the start of reprocessing at Sellafield require seismic assessment surveys. Many buildings and plant were not designed to seismic requirements but will nevertheless attain some level of seismic qualification as basic structural stability will have been assured and particularly as code requirements will have been satisfied for wind loading.

Inevitably not all will attain compliance with current requirements, in which case retrospective qualification will have to be carried out. It is this process which will be addressed in this case study.

2.0 BACKGROUND TO CASE STUDY

Pipebridges on the Sellafield Site which were commissioned in the early 1950's, were not subject to seismic qualification. The result of a number of studies carried out over recent years concluded that refurbishment of the pipebridge network was required in order to satisfy current levels of seismic loading.

A number of schemes were examined by various parties. These primarily involved the strengthening of existing structural steel supports and optionally the addition of new support frames. Extensive foundation works were also deemed necessary for such strengthening schemes with a possible requirement for piling.



EXISTING TRESTLE (SCHEME A) PROPOSED TRESTLE (SCHEME B) D G P PROPOSAL (SCHEME C)

DGP proposed only moderate strengthening to existing support trestles and the inclusion of an isolation system between the trestles and the pipeduct. This proposal required only limited remedial work to the foundations. To achieve this low level of site modifications a design concept was used which varied considerably from the standard attitudes used in earlier attempts at a solution.

Before reviewing the technical merits of the refurbishment, the approximate cost comparisons for the structural modifications are presented below;

Scheme A - Strengthening of existing trestles and foundations for minimum seismic qualification (0.05g)	£6.0M
Scheme B - Strengthening and inclusion of new trestles and foundations for current seismic qualification	>£10M
Scheme C - Strengthening and seismic isolation for current seismic qualification ***** THE DGP APPROACH *****	**<£4.0M**

3.0 INITIAL TECHNICAL ASSESSMENT

The accepted way to accommodate increased loading is to strengthen the structure, add new members or replace existing members with sections of greater strength. However, that load must eventually be resisted at the foundation. The other more complex alternative is to reduce wherever possible the loading within the structural system.

The first phase of the refurbishment project was a pipebridge almost 200m. long and the majority of the loading comes from the pipeduct @ 14te/m. The pipebridge can be idealised to a stiff supporting structure with large loads transfering directly to the foundation, the loads coming from the lightly reinforced concrete duct 8-10m off the ground. Hence, the analogy of the mass on the end of a cantilever as the first step in a manual assessment is fully justified. It is likely that enhancement of the input loading will occur for the system described when inspecting the broad banded response spectra for the site. What the enhancement will be, depends on the natural frequencies of the constituent parts of the structural system and how they relate to each other in terms of mass and stiffness.

It should be mentioned that two options are notionally available in dynamic analysis for minimal enhancement, either a flexible structure or a very stiff structure. To attain a very stiff structure would be difficult and would result in extensive strengthening works being necessary to resist the much higher loading. This would be impractical both from a construction and finance point of view. The more practical solution is to make the structure flexible to resist the dynamic loading.

In view of our expertise in Soil Structure Interaction (SSI) it is relevant to mention that when considering the loading transfer to the ground with **maximum dynamic enhancement**, a more substantial foundation may be required. The increased foundation loading over a greater area makes SSI more likely. Hence, a reduction in the dynamic loading would also eliminate the need for SSI analysis with the attendant savings in cost.

3.1 SCHEMES A & B

A hand assessment of the coupled system ie, fully connected, considers the supporting steel frame as a single degree of freedom(SDoF) system and the prime contributor to the dynamic system. From inspection of the site response spectra an enhancement of the seismic motions by factors of 3-4 can be expected when the fundamental frequency is in the range 3-10 Hz and critical damping is around 5%. Further dynamic amplification can also be expected above the trestle head.

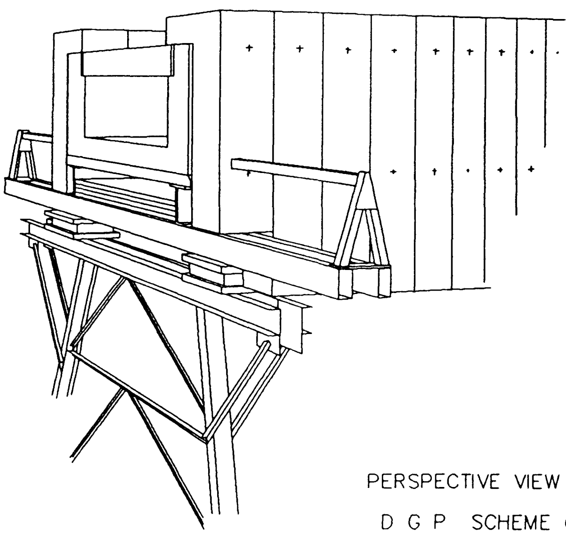
The above assessment was confirmed by modal analysis carried out on the supporting frame alone, which showed that the fundamental frequency contains almost the total mass and hence can be considered as a SDoF system. The peak acceleration plateau of the input response spectra falls in the range 3-10 Hz and the frame can expect full enhancement at the 4Hz frequency calculated.

When the full system is analysed the maximum frequency can be predicted well but the levels of acceleration require further interpretation and interpolation using empirical methods. For instance, the rigid body acceleration can be taken directly from the input response spectra but enhancement depends on the levels of dynamic interaction of the components which comprise the full system.

A time history solution of the coupled system gives response spectra directly at all points of interest with peak enhancement at the fundamental frequency of the system as predicted in the modal analysis. The acceleration increase up the structural system with an enhancement of 4-5 at the system fundamental frequency, depending on the critical damping levels.

3.2 SCHEME C

The scheme accepted was that proposed by DGP. In brief terms this was to introduce a purpose designed elastomer bearing into the coupled system which then effectively transforms it into a decoupled system with an isolation filter.

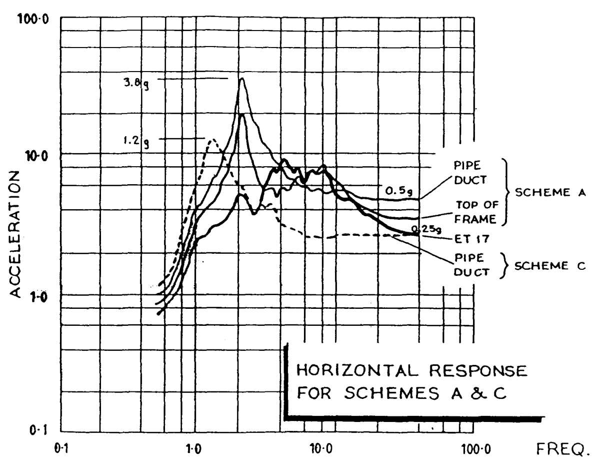


PERSPECTIVE VIEW OF
D G P SCHEME C

A hand assessment of the system is only feasible for the components themselves, which are almost identical to the fully connected system except for the elastomer bearings. A low frequency filter had to be selected to give good dynamic separation between all components in the system. The final system frequency was calculated to be similar and slightly lower than the fundamental frequency of the loaded bearings.

A modal analysis was initially performed on all the components and then on the full system itself. The results for the full system gave the expected results for the fundamental modes and the maximum acceleration. Immediate benefits were apparent in as much as the maximum acceleration was only marginally enhanced to the top of the structure where previously more than a threefold increase in acceleration would have occurred.

When the time history solution was carried out, the benefit of the isolation bearings was obvious. The maximum horizontal acceleration had changed only slightly and the enhancement to the peak acceleration was only a third of the value for the coupled system proposed initially. The enhancement at the fundamental frequency was less than twice the possible peak input acceleration.



To summarise; the acceleration above the bearings had been reduced from what could have been 4g using the Scheme A & B approach, to 1.2g with the isolation system(Scheme C). The benefits of this reduction are self-evident to all, especially the designer.

3.3 TECHNICAL SUMMARY

The elastomer bearings perform a number of functions on a global basis:

a. Reduce the system fundamental frequency
b. Introduce a low frequency filter
c. Increase damping and energy absorption

Generally a flexible system with a low frequency is beneficial, as forces and moments are reduced, but the displacements should be monitored carefully as they increase with increased system flexibility.

The filter effects shows enhancement only at the fundamental mode and attenuation thereafter up to the zero period or dynamically inactive range.

Finally the isolation bearings serve to dissipate the dynamic loading due to the energy absorption associated with the high local damping.

4.0 ANALYSIS IN DETAIL

A. Initially several frames were selected for investigation of strengthening requirements. The parametric studies involved changing section sizes of the members and introducing new trestle head details to accommodate the isolation bearings.

B. The dynamic characteristics of all components were studied by hand calculation or analysis:

a) Trestles/Towers
b) Active Pipe Duct
c) Isolation Bearings
d) Head Beam Connection

It was recognised that the trestles could be idealised to a SDoF systems. Hence a simple beam (or stick) model was used for the 3D dynamic analysis.

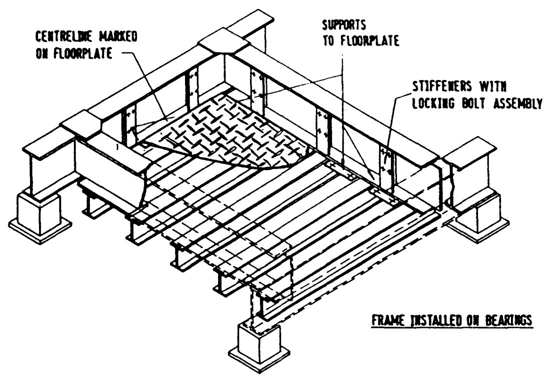
C. 3D modal analyses of the representative section of the bridge were carried out. The bearing stiffness was varied and acceptable deflections and forces were deduced.

D. The final 3D time history analysis was carried out to provide values for the design. The reason the time history technique was adopted was due to the necessity to include discrete damping and to be able to generate response spectra. It is worthwhile to note that reduction in displacements, forces and moment were achieved not only because of the additional sophistication of using the time history method but also because of the reduction of conservatisms over the modal combination method.

Due to the large number of pipebriges under review, progressing of the analysis and design was carried out in line with the preferred construction phasing.

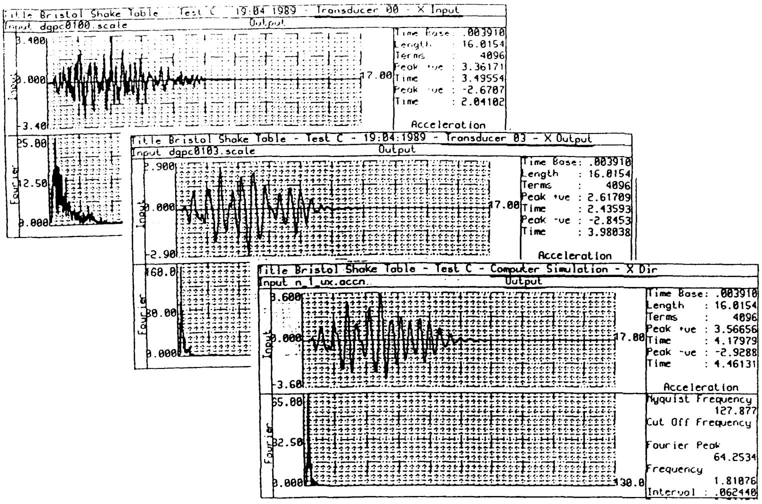
5.0 VALIDATION AND TEST

The use of elastomeric bearings for the refurbishment of the pipebridges was not a common solution as has been described. As a consequence a great deal of attention was paid to the dynamic performance of the bearings. Although the bearing designer-manufacturer had provided appropriate and adequate material data, it was clear additional validation was required to satisfy all parties as to the performance of the bearings under dynamic loading.



Shake table testing at the foremost dynamic laboratory in the UK, at Bristol University, was agreed and a test rig was designed and built by ourselves to achieve full capacity of the test system. The test rig was loaded to 11t on four quarter-sized bearings and accelerations in excess of 0.1g were achieved during dynamic testing. All six axes of the shake table were active during the full testing but initially only the representative earthquake for the Sellafield site was the basis for the comparison between analysis and test.

Thus, the scaled version of the isolation system was subjected to the maximum acceleration achievable and proved that the elastomer bearings were capable of outperforming the original manufacturer's material data, particularly with regards to the damping levels. On the most conservative estimate the validation tests showed a 40% saving on the analytical base data solution.



The most convincing result of the testing performed was the durability of the bearings themselves. Well in excess of a hundred earthquakes were experienced by the bearings, including additional resonance tests, with no evidence of distress.

6.0 SUMMARY OF PROJECT DESIGN & ANALYSIS METHODOLOGY

The expertise and past experiences at DGP enabled us at an early stage to propose the use of elastomer/isolation bearings in the design. A similar design philosophy had been pioneered and satisfactorily adopted on a previous project for BNFL. The system used on the earlier project was developed and modified to suit this project. The design team allowed a structural scheme to be developed which included an economic installation method of the isolation systems without resorting to expensive temporary works and jacking arrangements. The refurbishment of the pipebridges showed itself to be an excellent piece of engineering and a vindication of DGP's policy of integrating the design and analysis teams.

It is worthwhile to mention that for all stages, from initiation to final design, the design and analysis was carried out using our in-house QA system which has been certified and approved by the British Standard Institute. As a function of this system the design office, client and independant assessor were kept fully informed and their comments taken into account from an early stage through to formal submissions.

The efficiency of this method of approach is underlined by the fact that all the project milestones were achieved within agreed targets.

7.0 CONCLUSION

The refurbishment of the pipebridge with the isolation system is an ongoing project with most stages having been completed. The principle of using elastomer bearings for seismic isolation has been accepted by the NII and a Notice Of No Objection has been obtained for all phases of the construction.

By far the most notable of the results of this exercise and the reason for this paper ie, **Cost effective seismic qualification**, is that DGP put forward a proposal that will cost the client £4M for the first phase of the work whereas the alternative schemes were 2 - 3 times more expensive, and would have created substantially more disruption to the production process on the site (lost revenues of £1M/day can be a realistic figure for a shutdown of the reprocessing cycle).

21. Design and construction of immersed tube offshore cooling water tunnels at Sizewell 'B' power station

J. T. VAUGHAN, and D. T. WILLIAMS, Nuclear Design Associates

SYNOPSIS. Steam used to drive the turbine generators at Sizewell 'B' Nuclear Power Station is cooled by passing through condensers where heat is transferred to cool sea water pumped through the Cooling Water System. The paper describes the initial investigations, design, and construction of the offshore tunnels used to draw cool water from 800m offshore and discharge the warmed water 200m offshore.

COOLING WATER SYSTEM

1. The Main Cooling Water System, Fig 1, is a conventional 'once through' direct cooled design. The civil works consist of the intake tunnels, the C.W. pumphouse, intake culverts from the pumphouse, culverts through the turbine blocks and cooling condensers in the turbine house, outlet culverts to a surge chamber, and outfall tunnels leading to the offshore outfall structure.
2. The system has a total flow of approx 52 cumecs to meet all station demands. The design velocity through the tunnels was optimised at 2.5m/s, ensuring self cleansing to prevent sedimentation, and giving a good balance between construction costs and running costs. Large, slow flow, low head loss tunnels result in high construction costs and low running costs, whilst small high velocity, high head loss tunnels give lower construction costs and higher running costs.
3. After investigations by the CEGB the location of the intake and outfall structures was chosen taking into account the recirculation effects, and capital costs. The intake was located at the 9.0m contour approx 800m offshore and the outfall was located at the 5.0m contour approx 200m offshore, the resulting vertical and horizontal separation restricting increase in intake temperature, due to recirculation, to 1°C.

SITE GEOLOGY

4. The geological succession at Sizewell comprises a pre-Permian floor of the London-Brabant Massif overlain by Gault Clays, Chalk, Lower London Tertiaries, London Clay, Crag deposits, and glacial drift and recent deposits.
5. The Chalk beds 90m below ground level, are between 300m

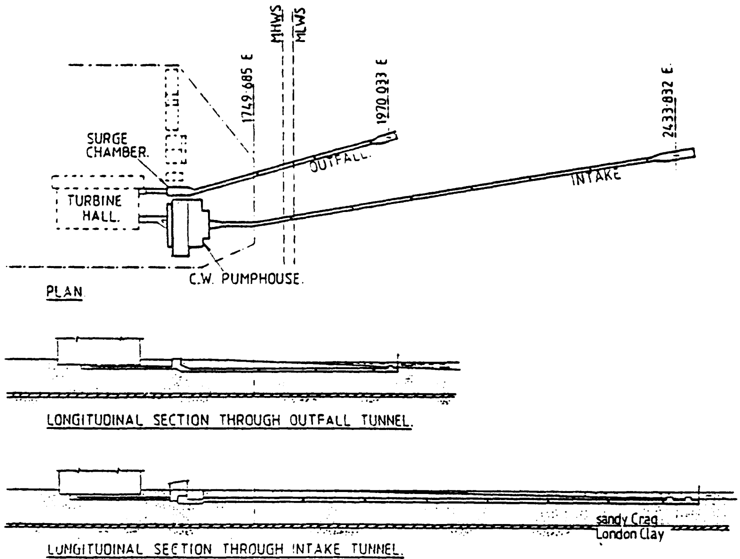


Fig. 1. LAYOUT OF COOLING WATER SYSTEM.

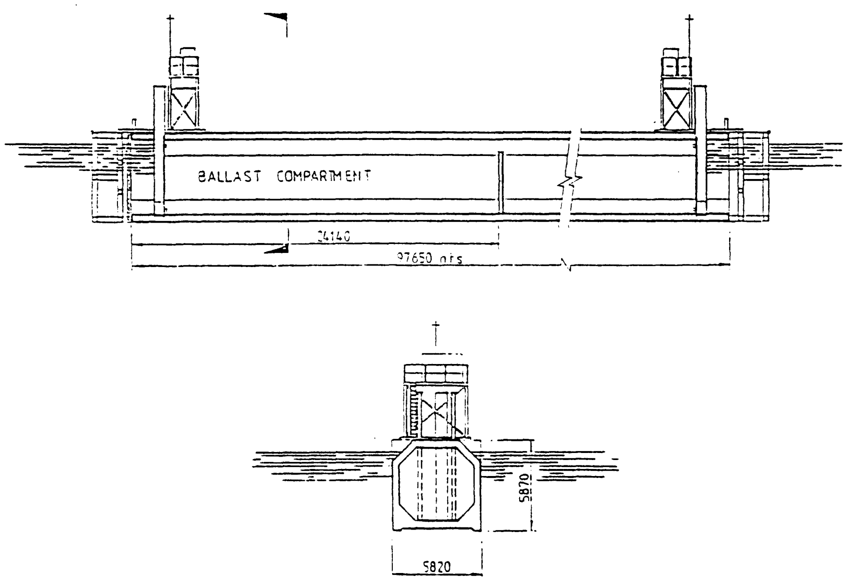


Fig 2. STANDARD PRE-CAST UNIT

and 350m thick. Up to 21m of Lower London Tertiaries overlay the chalk. Above this lies approx 14m of London clay, the top level falling from -43m O.D. in the North West, to -49m O.D. in the South East of the site. The upper sandy Crag deposit, on which there is some superficial recent soft sandy deposits, is thus 49-55m thick over the Sizewell site. North of the site there is a depression in the Crag resulting in a marshy area up to 10m deep consisting largely of peat with soft clay and silt lenses. This area is protected from the sea by the Bent Hills a sandy ridge running along the rear of the beach.

6. When considering the various options for tunnel construction, the materials of particular interest are the London Clays and Crag deposits, these being the most accessible strata suitable for conventional tunnelling techniques. The Crag deposit is essentially a dense to very dense fine to medium sandy deposit, with high permeability. The London Clay at Sizewell is typical of the London Clay beds and consists of a uniform deposit of clay and silt sized material having a low permeability.

INITIAL STUDIES

7. Studies were undertaken at the outset to establish feasible construction schemes for the CW Tunnels, and to compare the relative merits and costs of each proposal. A preferred scheme was identified and further work was carried out to develop the scheme to the stage where enquiry documentation could be prepared. This proposal, the Engineer's preferred scheme, became the Reference Design and set out the parameters and requirements against which any contractor proposed alternatives could be assessed.

8. Eleven tunnelling schemes were considered, eight dewaterable and, after discussing the frequency of tunnel access with the client, three additional non-dewaterable schemes.

1) Driven tunnels in sand using compressed air-Single †
2) Driven tunnels in sand using compressed air-Twin
3) Driven tunnel in sand using Bentonite Slurry Shield-Single †
4) Driven tunnel in sand using Bentonite Slurry Shield-Twin †
5) Driven tunnel in clay-Single †
6) Driven tunnel in clay-Twin †
7) Dewaterable Submerged tube laid in trench-Twin
8) Proprietary precast pipes (Bonna) laid in trench
9) Short precast pipes placed from causeway in sheet piled trench
10) Short precast pipes placed from marine craft
11) Non-dewaterable submerged tube laid in trench.

†These five schemes were rejected after an initial review in 1982.

9. The single driven tunnel in sand uses compressed air with pressures in excess of 40 psi (3.0 bar). Under present Health

and Safety Legislation the limitation on pressures and decompression times make this scheme impractical.

10. At the time of the investigation, Bentonite Slurry Shield tunnelling was unproven in the United Kingdom, especially offshore, and the risks were considered too great to recommend the technique as the preferred option.

11. Consideration was given to the proposal to sink shafts, some 55m below ground, and drive tunnels in the London Clay. The risk of break out into water bearing sands above and below the relatively thin London Clay strata was considered unacceptable and a substantial programme of offshore geotechnical exploration would be required to prove the clay layer along the route of tunnels. Therefore the risks were considered too great to recommend the scheme as the preferred option.

The remaining schemes were considered in more detail.

12. The twin bore driven tunnel scheme employed conventional compressed air tunnelling techniques, the only novel construction being the offshore tunnel end shafts. These could be driven from a cofferdam or pushed up from the ends of the tunnel, the technique used for the adjacent Sizewell 'A' tunnel shafts. The offshore intakes require special capped heads which would be cast onshore and transported to the final location and lowered into place.

13. The dewaterable submerged tube double bore scheme was developed to allow the economic production of standard 3.75m diameter precast tubes, 2.4m long, which could be post tensioned together to form 60m long units on the sea shore. These units would be fitted with end bulkheads and other ancillary items to allow the units to be towed into position and lowered onto preplaced pad footings. Adjacent units would be placed and backfilled to half height to provide stability. The initial rubber seal joint would be completed by placing internal insitu reinforced concrete. Special intake and outfall units would be constructed for the ends of the tunnels. An alternative scheme evolved from this proposal. A twin bore section was developed to be self buoyant during tow out but incorporated a central chamber which could be filled with water ballast during lowering, and finally filled with mass concrete to form permanent ballast. These twin units would be too heavy to handle onshore and so the commonly used concept of building the units in a dry dock which is flooded when all units are complete would be adopted for unit handling.

14. Proprietary precast concrete pipes have been used extensively in the French Nuclear Power Station programme for the construction of cooling water tunnels. Sociéty Des Tuyaux Bonna agreed to estimate costs for the design, supply and installation of their system at Sizewell. The proposal utilized 3.7m internal diameter precast reinforced concrete pipes with a sandwiched steel cylinder, produced in France and shipped to site for final assembly and placing. The 2m long sections would be joined together to form 18m long tubes which

were to be transported to jack-up barges and lowered into a levelled, predredged trench. The initial joint between units was a rubber sealed spigot and socket pulled together by hydraulic jacks and made good with internal insitu concrete. Adjacent pipes were to be laid and backfilled to half height to ensure stability.

15. The remaining three schemes were non-dewaterable allowing lighter units, acceptance of minor seepage at joints, and less onerous loadings during operation.

16. Precast pipe sections could be manufactured onshore complete with spigot and socket joint and ring seal, transported to the foreshore and picked up by a gantry crane running on a causeway built progressively from land. The pipes would then be placed in a trench between sheet piles on a levelled gravel bed, all work carried out by plant operating overhead on the trestled causeway, a technique used in the United States.

17. Similar precast sections could also be placed offshore in a dredged channel employing marine craft to place the pipes on a prepared gravel bed.

18. The non-dewaterable submerged tube scheme involved the construction of 5.2m diameter precast concrete sections 2.0m long post-tensioned together to form 100m long units. These units would be constructed in a casting basin and, after fitting out with steel bulkheads and ancillary equipment, would be towed to site where they would be lowered on to preplaced concrete pads laid to line and level. Backfill would be placed as soon as possible after the unit was placed, and vibrated under the unit to provide support. The unit would then be flooded, the bulkheads removed, joint completed and backfill placed.

19. The schemes were then subjected to an engineering, risk, programming, and economic assessment to determine the preferred scheme.

20. Engineering Innovation

The driven tunnel scheme involved least innovation employing conventional mining and tunnelling techniques. The proprietary pipe scheme had been successfully completed in France on several Nuclear Power Stations, and placing of easily manageable segments appeared to be well proven on other projects. The submerged tube technique is well proven on many road and rail crossings and some outfall tunnels, but the proposed tunnel would be the first in the North Sea, and would thus be subject to more extreme environmental conditions.

21. Risks

The risks associated with the driven twin tunnel scheme, especially two tunnels driven side by side, present all the risks associated with tunnelling in compressed air. The risk of blowout is present, especially whilst using the relatively high pressures required. The shaft ends would present a hazardous operation, as would the placing of the special intake heads. Risks for the remaining schemes centred on the marine activities which increase with the unpredictability of the weather, and its effect on the local sea state, length of time offshore, number of marine activities, and complexity of operations. Desk studies indicated that, during the summer, weather conditions would allow marine work to be undertaken. The proprietary pipe scheme involved placing over 100 No 18m long pipes on a prepared bed, the precast pipe schemes use 960 No 2m long pipes, and the submerged tube scheme employed only 10 No 100m long units, posing least risk of weather disruption. However, the larger units involved a complex handling operation requiring specialist plant and expertise.

22. Programme
Construction times for the various schemes were assessed to verify that each scheme could be constructed within the overall station construction programme. Timescales were within the range of 24 months to 54 months and therefore satisfied the station programme requirements.

23. Economics
Costs for the schemes were prepared on a comparative basis, the conventional compressed air driven tunnel being taken as the reference. The scheme prices ranged from 125% for the proprietary pipe scheme to 81% for a 100m long large diameter submerged tube scheme taking advantage of existing casting yard facilities off site. It was decided that the submerged tube scheme should be adopted as the Engineer's preferred scheme.

ENGINEER'S REFERENCE DESIGN

24. The submerged tube scheme adopted as the Engineer's preferred option was further developed so that Enquiry Documents could be issued to selected tenderers.

25. Investigations were undertaken to confirm design parameters, to define hydraulic profiles, to confirm system performance and accurately assess risks. These included soil and hydrographic surveys, trial trench excavation, unit stability model tests, together with intake head and recirculation models, pumphouse forebay model tests, and environmental wind and wave analysis. As a result of these investigations the Reference Design evolved from the initial submerged tube proposal.

26. Early in the development of the scheme, fundamental decisions were taken to ensure that the Engineer's scheme represented a practical proposal but would allow the benefit of high levels of contractor expertise in the construction methodology.

27. Firstly, it was decided to design buoyant units capable of being towed safely over long distances without additional buoyancy or stability aids, as there was insufficient area available to form a casting basin at site.

28. Secondly, the unit configuration was chosen to allow conventional reinforced concrete construction by selecting a rectangular section with large corner splays. This gives good hydraulic performance whilst allowing conventional construction techniques, and provides a stable unit both during tow and when sitting on the sea bed.

29. Thirdly, a gravel bed was chosen as the means of supporting the units. This allows rapid flooding of the unit ensuring minimum risk of unit disturbance once in place, and obviates the need to have backfill materials available and plant operating during the unit placing phase.

30. These parameters dictated the form of the concrete unit to be towed to site and lowered into place. Unit handling was the responsibility of the contractor but, in order to be satisfied that the scheme was practical, considerable investigation and development took place on methods of dredging, towing,handling, ballasting,lowering and backfilling the units. Included in the development was the type of joint compression equipment, external bulkheads, internal bulkheads to form ballasting compartments, and collision guards. The resulting complete scheme was then reviewed, and priced to establish overall budget targets.

31. Temporary works associated with the construction of the onshore tunnels were also developed, taking into account the interface between insitu onshore tunnels constructed within the whole site dewatered diaphragm wall cofferdam, and precast offshore tunnels. Large retaining walls were designed to hold the beach and maintain access along the foreshore during the dredging operation, and to form flooded cofferdams to allow precast units to be floated in across the foreshore to seal against the ends of previously constructed insitu onshore tunnels. Incorporated into the scheme were all necessary construction phases to prevent inundation of the main site during the controlled breaching of the diaphragm wall which surrounded the site. Large areas of wall in excess of 20m deep were required and as driving sheet piles to such depths in the very dense sandy Crag deposits was deemed to present considerable difficulty, diaphragm walling, already successfully constructed round the site, was adopted in the Engineer's Scheme.

PRELIMINARY INVESTIGATIONS

Site Investigation and Hydrographic Survey

32. In June 1982 a joint venture comprising Soil Mechanics and Ham Dredging, known as Sizewell Offshore Joint Venture (SOJV), were instructed to carry out offshore site investigations by the C.E.G.B.

33. The investigation consisted of conventional marine boreholes and cone penetrometer tests together with insitu testing, sampling and laboratory testing, a trial dredge combined with periodic hydrographic surveys, and the deployment of oceanographic and meterological instrumentation for a period of five months to record sea state and meteorological data.

34. Results indicated that the thickness of the superficial deposit varied up to a maximum of 4.0m, confirming that the submerged tubes were founded well into the dense Crag Deposits. The Standard Penetration Test results varied from 30 to 150 blows per 300mm indicating dense to very dense material, borne out by Cone Penetrometer Tests where the majority of results were in excess of 120 kgf/cm^2, averaging nearly 400 kg/cm^2 at 5m into the Crag.
35. Consulting Engineer's Lewis & Duvivier were commissioned to report on the stability of the sea bed and reviewed several sources of data from 1824 onward, concluding that the sea bed was generally stable although local storms caused some movement of loose sand. In 1982 the sea bed was surveyed to produce accurate profiles from which tunnel levels and hence overburden depths could be established for design, and minimum cover ensured for tunnel protection.
36. Data on tide levels, currents, waves and wind was all automatically recorded and stored for detailed analysis.
37. Analysis of wind and wave data indicated that the prevailing westerly winds have a flattening effect on wave heights, and wind from most easterly quarters induced wave action. However, during the summer period, records showed that there were significant periods when wave heights were below 0.7m, the anticipated maximum acceptable wave height tolerable during unit placing operations, and that in general, wave heights exceeded 0.7m for not more than 20% of the time in June, July and August.
38. As a result of this analysis, together with comparisons of published and recorded data available for periods of up to 10 years previously, it was concluded that there were no inherent environmental problems which would preclude the use of submerged tubes at Sizewell.
39. Observations taken after completion of the trial trench indicated that after initial siltation and slump, the sides of the trench remained stable over at least the following 6 month period when soundings were taken. From the initial slope of 1 in 4 the northern slope flattened to a stable 1 in 5 and the southern slope flattened to 1 in 7 over the same period. The invert suffered some siltation, the trend being an infill of approximately 200mm per month. The stable side slopes were adopted as the dredge profile for trench excavation.

Extreme Wave Analysis

40. The only element of the tunnels exposed to wave action on completion of the construction works are the intake head structures and the outfall. It was decided that the most severe wave likely to occur once in fifty years would be used to assess the strength required and thus investigation took place to derive the design wave characteristics. Conventional wave prediction methods, using 1 in 50 year wind data, were used to establish offshore deep water wave parameters and simplified shoaling and refraction studies, appropriate to the local coastline, defined inshore wave design parameters.

Hydraulic and Thermal Studies

41. The system design, involving calculation of hydraulic head losses and pumping costs, culvert sizing, syphon and seal weir level, location of intake and outfall, pumphouse layout, and surge analysis was carried out initially by a CEGB-NNC Joint Team at CEGB's Generation Development and Construction Division headquarters at Barnwood, later reformed as the Project Management Team (PMT) at Knutsford, now known as the PWR Project Group (PPG).

42. A programme of design studies and physical model testing was initiated to establish the system layout and effective hydraulic profiles at critical areas in the system.

43. The Central Electricity Research Laboratories at Leatherhead (CERL), now the National Power Technology and Environment Centre (N.P.Tech.), carried out hydraulic model tests on the proposed intake heads to investigate the flow patterns and draw-down of any warm water drifting over the sea surface in the vicinity of the intake heads. Various intake head configurations were tested before finalising on the present outline which limits recirculation (ie the increase in intake water temperature above the cool water ambient temperature) to about 1°C, gives acceptable hydraulic flow performance, and reduces the quantity of fish drawn into the system.

44. Onsite measurements of the existing warm water plume generated by the Sizewell 'A' Power station were taken by both NP Tech and the Hydraulics Research Station (now HRL) to plot accurate profiles. These were used to verify mathematical modelling techniques for the prediction of future developments at Sizewell. After completing these mathematical model studies the design team fixed the locations of both intake and outfall such that performance parameters dictated by thermal efficiency, and construction costs, were satisfied. For Sizewell 'B' the maximum sea temperature should not exceed 26°C and recirculation should not exceed 1°C.

Specialist Marine Consultants

45. Marine operations are an integral part of the submerged tube technique and influence unit layout and design. In the early stages, before the active involvement of the contractor, an outline sequence of operations was developed. This formed the basis of the Engineer's Reference Design and was included in the Enquiry Documentation as a conceptual construction sequence.

46. It was envisaged that the units would be constructed in a casting basin which would be flooded to allow the units to be towed to site. Once at site the units would be attached to cranes, ballasted, and lowered onto the prepared bed. After accurately positioning and joining to the previously placed tunnel segment the unit would be flooded and backfilled.

47. It was essential to establish the wave environment during the tow and to ensure unit strength and stability during this

phase of the work. Noble Denton Consultancy Services were employed to give predictions of the sea states for a notional tow along the east coast. It was decided that, although the period that the unit would be at sea would not be greater than about 7 days, the minimum conditions the unit must be capable of riding out should be the worst likely to occur once in ten years. Thus, the 1 in 10 year wave was used to calculate the wave bending moment. In addition, after issuing enquiries to several hydraulic laboratories, Noble Denton commissioned Taywood Engingeering Laboratories (TEL) to carry out model stability tests in their wave tank. The largest models that would permit waves of the required height to be generated in the TEL tank were constructed and weighted to float at the specified draught and disposed to give a natural period as close as possible to the calculated scaled prototype period. Tests were considered advisable as the damping effects of corner submergence and wave overtopping significantly affect unit responses to wave action. Analysis of the results from the model tests confirmed stability curves for the concrete unit. These were then used to assess the stability of the unit for various combinations of events, ensuring that the units satisfied the recommended stability criteria.

48. The unusual nature of the tunnel units, when considering them as floating vessels, meant that unique stability criteria needed to be agreed with certification and insurance bodies early in the design process.

49. The conclusion of all the testing was that the proposed units would be stable during the tow to site even when subject to 1 in 10 year wave conditions, a surcharge due to additional equipment and leakage of up to 150mm of water into the unit.

Analysis & Design

50. Preliminary structural analyses of the units were carried out to assess the reinforcement quantities. These were of necessity, tentative in nature as the stresses induced during manufacture, launch, towing, handling, and lowering were based on a conjectural methodology. In the permanent condition, less onerous transverse loadings were anticipated, but longitudinal loadings would be induced by distortions due to tolerances in the stone bed. However, from these calculations the concrete outline and reinforcement arrangement was evolved for tender.

ENQUIRY AND TENDER ASSESSMENT

51. The technical specification for the Cooling Water Contract comprising the CW Pumphouse, Surge Chamber and associated works, together with the tunnels linking with the intake and outfall structures, was considered under three headings, conventional onshore civil construction, offshore clauses relevant to the Engineer's scheme, and a performance specification for the design of modifications to the Engineer's scheme or alternative schemes.

52. The whole of the Cooling Water Contract is subject to a marine environment and therefore the material specification clauses for the offshore works referred to the equivalent onshore clauses, amended and expanded as necessary to cover the construction of the precast tunnel units.

53. Additional clauses covered dredging, stone foundation bed, stone scour protection, precast concrete revetment systems, offshore piling, offsite tunnel unit casting facility, unit floatation, transportation and handling, offshore plant, tunnel joints, and steel bulkheads. Testing of various aspects of the work, including concrete watertightness, bulkhead leakage and final leakage, was also addressed. The philosophy adopted when preparing these clauses was to ensure adequate control of the works, acceptable standards of performance, and maximum flexibility of methodology.

54. The performance clauses set out parameters adopted in the preparation of the Engineer's scheme to enable alternatives to be engineered to the required standards. These included layout coordinates, design wave height and wind speeds, and hydraulic performance data including total flow, nominal flow velocity, maximum head losses, intake and outfall velocities, water temperatures, hydraulic surge and tide levels.

55. The project Quantity Surveyors Langdon and Every, now Davis, Langdon and Everest (DLE) prepared Bills of Quantities based generally on the Civil Engineering Method of Measurement, 2nd Edition, 1985 (CESMM2). Essentially the tunnel manufacture was measured conventionally as precast concrete elements complete with steelwork inserts. Separate items were included for bulkheads, transportation of the units from the place of manufacture to site, and for lowering the units into position.

56. The final Enquiry drawings package for the offshore tunnels consisted of dredging profiles, unit concrete general arrangements, longitudinal sections, joint general arrangement details including typical section reinforcement arrangement, steel end bulkhead general arrangement, end navigation warning markers and a conceptual placing sequence. Temporary beach works layout and a sequence drawing outlining the method of joining onshore tunnels built within the main site diaphragm wall and the offshore tunnels were included to ensure the safety of the site during construction.

57. Assessment of the submissions for the offshore tunnels was carried out by independent Consulting Engineers, Sir William Halcrow and Partners, appointed by the PPG. The assessment compared all the submissions and studied the more competitive schemes in detail, which included alternative driven tunnel schemes, as well as submerged tunnel schemes.

58. After careful assessment a submerged tube tunnel scheme was accepted, the successful tenderer being Kier Construction Ltd. Accordingly a letter of acceptance was issued in September 1988 and the Inaugural Meeting took place late in September 1988.

DEVELOPMENT AND FINAL DESIGN

59. At the Inaugural Meeting it was agreed that a series of design liaison and coordination meetings should take place to clarify design responsibilities and discuss and agree design parameters and jointly develop the final scheme.

60. The contractor's tender scheme followed the Engineer's rationale in most respects. Precast concrete standard units 100m long, Fig 2, and non-standard intake and outfall units, manufactured off site are towed to Sizewell, attached to cranes on marine craft, ballasted by flooding ballast compartments, lowered to a prepared bed, drawn together to form a watertight joint, adjusted for position and flooded prior to fitting precast concrete covers to all openings and backfilling to the final sea bed profile.

61. However, the Contractor proposed two significant changes to the Reference Design, revised temporary beach works and unit joint configuration, and two substantial variations on the conceptual construction method, revised unit handling at the manufacturing site and the use of floating barges to lower the units to the sea bed.

62. The temporary beach works proposal substituted steel sheet piles for the concrete diaphragm walls. It was agreed that, although pile driving in the very dense sand to the depths required would require careful investigation and proving, the proposal was acceptable in principle and design and construction should be the responsibility of Contractor. In view of the risk to the programme of any failure, and the potential of site inundation, calculations were vetted and approved by the Engineer prior to construction.

63. The Contractor proposed a compression joint between units instead of the spigot and socket joint detailed in the Enquiry. The rubber compression seal was developed for Kier by the Dutch manufacturer Rubberfabriek BV of Ridderkerk via their agent in the UK, Burleigh Marine International, to parameters specified by the Engineer. This change when fully developed to include the compression seal and fixings, and separate top and side backfill protection covers represented a simplification of unit production and a saving on unit weight, resulting in improved flotation and stability characteristics.

64. Other proposals having significant design consequences were the unit handling method at the casting yard, and the location of the yard, some 200 miles north of Sizewell, at the mouth of the river Tees in an existing disused dock area.

65. The unit strength was assessed for transverse and longitudinal load cases defined by external loadings and temporary and permanent support conditions.

66. The contractor defined temporary loadings, including the tow attachment locations, unit lowering attachment points, temporary winch locations and ancillary equipment positions, allowing the final analysis, design and detailing of the units to commence.

67. The design of the unit cross section was dictated by the maximum external water pressure imposed on the unit when

empty. Once on the sea bed the unit is flooded, equalising pressure prior to backfill. Longitudinally, several conditions were checked, including temporary support during handling, wave forces during tow, temporary support during ballasting and lowering, and final stresses caused by distortion due to tolerances in the gravel bed profile.

68. Local strength was checked at attachment points for the steel grillage onto which towing bollards, temporary guidance, and a generator support tower were all fixed. Separate attachment points located on small plinths along the length of the unit to allow installation of the survey towers and additional bollards required for temporary fenders and unit mooring line attachment were also checked.

69. Design and development of the end bulkhead gates progressed concurrently with concrete unit design. These items were designed to withstand full hydrostatic loadings imposed during unit lowering to the sea bed. The layout incorporated the unit bilge pumps housed in tubular wells connected by drain tubes to the unit, ballast and unit flood valves, air release valves, and man access shaft. All these items, required in the specification documentation, were incorporated in the bulkhead gates so as to minimise possible leakage and additional marine operations involved in placing closures. The layout of the gate and equipment was developed by the contractor to suit the alternative joint detail and ballasting method. The contractor also designed and detailed the temporary, removable internal bulkheads which formed the inner end of the ballasting compartments.

70. A similar design and detailing exercise was carried out to prepare working drawings for the intake and outfall closures, the end bulkheads being common to all units.

71. Discussion took place on the construction and handling methods preferred for the manufacture of the intake head units, after which design and detailing commenced.

CONSTRUCTION

72. Whilst initial design discussions took place the Contractor initiated negotiations with subcontractors and preliminary work commenced on site, including sheet pile driving tests prior to commencement of onsite offshore tunnel temporary works.

73. The offsite casting yard was chosen and the unit handling subcontractor was appointed. This allowed the unit handling methods to be finalised and design loadings to be established. Work started at the selected location at Smith's Dock on Teeside in late October 1988.

74. The special intake and outfall units were constructed in the dry dock and, when completed in March 1990, the dock was flooded. The units, which floated within 20mm of the predicted level, were then moored close by for a period of up to 3 months until required at site, during which time leakage was so slight that the installed pumps were not utilized.

75. The Contractor, having had negotiations with several marine contractors, sub-contracted the placing of the stone bed, the towing from the Tees to site, and the placing in position of all units to SmitTak, an experienced Dutch marine contractor.

76. The trench in which the units were to be placed was dredged in one single operation starting in midApril 1990 and completed at the end of May 1990. During this time some 500,000m^3 of dense sand was dredged from the intake and outfall channels and removed to an approved dumping area on a natural sand bank known as Sizewell Bank, 2km offshore. The majority of the dredged material was subsequently reclaimed and used as backfill over the tunnel units, the beach and offshore sea bed being reinstated to its original pre-construction profile.

77. After dredging, the stone for the bed was dumped over the bottom of the trench up to 200mm above the required level. A specially developed mobile screeding machine displaced all stone above the required level to the side of the trench leaving a stone bed to a tolerance of ±50mm as required in the specification.

78. The first of the units was towed from Teeside on Sunday 24 June 1990 and arrived at Sizewell some 30 hours later after a stable, trouble free journey. On arrival, the weather forecast and environmental conditions were acceptable and therefore the unit was made ready for lowering into place. Crane hooks were attached, survey towers installed, and when all was ready, the unit was ballasted by simultaneously flooding compartments at each end of the unit imposing a load of approximately 125 tonnes on each crane hook. This was deemed the optimum load to ensure stability during unit lowering but minimum resistance to final movement during unit positioning. Once ballasted the unit was lowered onto the previously prepared stone bed.

79. The first outfall unit on site was manoeuvred into a specially constructed cofferdam to install the unit as far as possible up the beach. Subsequent units were fitted out with guidance beams and hydraulic joint compression rams prior to lowering to the seabed. Each unit was checked for position by sighting onto survey towers and adjusted using the specially laid mooring lines, snatch blocks, anchors, and winches on board the SmitTak barges. Adjustments to within 75mm of the true position were regularly achieved. When accurately positioned, and with the joint adequately compressed the unit was fully flooded, followed by slot and joint cover placing.

80. The operation at Sizewell, from arrival at site to final flooding on the gravel bed, took about 48 hrs, but if weather conditions deteriorated, it was possible for the unit to be held at various stages, either offshore attached to the tow tug, or inshore held by the mooring lines and crane.

81. The final unit was placed in August 1990, after which piling of end anchorages and marker platforms were completed

and backfill placed. The whole operation from towing of the first unit in June 1990 to flooding the last unit in August 1990 was accomplished, without any significant problem, well within the allotted programme time.

ACKNOWLEDGEMENTS

82. N.D.A. as civil engineering consultant to the Nuclear Electric Sizewell 'B' PWR Project Group wish to acknowledge the contribution of Kier Construction Ltd in the development of the final details, NNC in the initial investigations, and the assistance of Sir Robert McAlpine & Sons and Taylor Woodrow Construction Ltd, the N.D.A. parent companies, in the practical aspects of the initial studies, and later specialist marine input.

83. N.D.A. also wish to thank the PPG for the opportunity to develop this challenging civil engineering project and the authors wish to thank the PPG for their permission to publish the paper.

Discussion

R. R. KUNAR, BEQE Ltd, Warrington
As the bearings only lowered the frequency from 2 to 1.5 Hz, then it would seem that we are taking advantage of the damping as the dominant factor in reducing the seismic loads. Could the same solution be obtained by adding some damping, rather than a vibration isolation system?

Vibration isolation reduces acceleration but increases displacements. Bearing this and the length of the bridge in mind, were seismic travelling waves considered in the safety assessment?

J. W. SAUNDERSON, Merz & McLellan, Newcastle-upon-Tyne
I would like further information about the joint between the precast tunnel units. To what extent will the joint allow differential settlement between units after placement and in service?

J. P. NEWELL, W. S. Atkins Engineering Sciences, Bristol
My question is in two parts, relating to the durability and non-linear properties of elastomeric bearings.

What type and chemical composition of elastomer was used, and what durability will the bearings provide in relation to ozone attack and exposure to the local environmental conditions?

The mechanical stiffness and damping properties of elastomers are known to show strong non-linearity with regard to dynamic strain and frequency. To what extent, therefore, can the scale-model tests reported in Paper 20 provide confident information on full-scale performance? In particular, the reported scale-model tests differed markedly from the manufacturer's dynamic test data.

B. BRUCE, ABB Impell, Warrington
On the assumed stress-strain curve for steel, was the

effect of the high rate of strain taken into account in order to enhance the yield value? Why was it necessary to apply a strain limit of 2% when there is ample test evidence that steel can reach strains of up to 20% and more before failure?

B. JORDAN, BNFL Engineering, Warrington
The prospect of aligning 3000 tonne precast concrete units is facinating. What alignments were achieved under water, and were divers used under water during linking and alignment operations?

R. R. KUNAR, BEQE Ltd, Warrington
Were the pipe restraints added for impact loads as well as jet impingement? Was a cost/benefit analysis performed to determine whether it was better to add restraints or provide protective barriers for the control systems?

D. DRYSDALE, BNFL, Sellafield
What dispersal testing was carried out at the outfall sections to validate design of the dispersal level? Was any pressure testing of the completed structures carried out?

G. VACIAGO, Rendel Geotechnics, London
Two of the three papers presented in this session have shown the benefits and savings that can be derived from taking advantage of ductility and non-linear behaviour in structural design under extreme loading conditions, such as seismic conditions. It is well known that stiffness attracts load. High loads are also transmitted to secondary structures and plant by stiff primary structures. The need to remain in the elastic regime under extreme load conditions should be reviewed, and the panel and the floor are invited to comment on the possible extension of the principles shown in the Papers to other elements of the civil works and nuclear installations.

R. HILL, AEA Engineering, Warrington
Did any of the Authors use boronated concrete as a sacrificial layer to help in decommissioning operations?

M. MANDZIJ, Paper 20
The response spectra shown compares the original (unmodified) steelwork (scheme A) and the isolated

system (scheme C) when the lateral frequency reduction was from 2 to 1.2 Hz. However, it must be noted that scheme A is not a viable design option since the seismic capacity of the existing system was below 0.05 g design base earthquake.

Consequently, to resist a 0.25 g DBE the steelwork requires strengthening which increases the system stiffness, and the frequency increases to almost 4 Hz. Hence the true comparison of system frequencies (for valid design options) yields a reduction from 4 to 1.2 Hz. Therefore, damping becomes a second-order effect, compared to frequency reduction and filtering of the motions. It is indicative of the damping contribution that model analysis with no damping contribution from the bearings) showed up to a 20% increase in horizontal dynamic response.

As Mr Kunar suggests, travelling waves were considered but did not pose a significant problem since the pipebridge had been constructed with a number of expansion joints, which provide articulation every 20 m along the length.

In reply to Mr Newell, the bearings were manufactured by Andre (a division of BTR Silvertown Ltd) from natural rubber compounds; the exact composition was determined by a specialist designer (Materials and Engineering Research Laboratories) to meet a specified dynamic stiffness. The durability of these compounds has been proven in practice under varied exposure conditions. Additionally, control bearings will be used to monitor condition and performance, and the design encompasses the facility to replace any bearing without major temporary works.

We would agree that this material exhibits 'strong' non-linearity for strain values up to approximately 10%, but starts to improve above this strain value. In the operational design range of 20 to 30% strain, linearity shows a dramatic improvement. The dynamic properties of the bearings were fully tested in the strain range of use and performed as expected. It is not entirely correct to describe the shake table testing as scale model testing, as the bearings were full height (but reduced plan area) units working in the relevant strain range with a representative dynamic load.

It was not reported in the Paper that the shake table tests differed markedly from the manufacturer's dynamic test data. The Paper refers to the 'original manufacturer's material data' which is the target design specification. Actual dynamic tests on the bearing material show the damping characteristics to exceed the target design by a significant margin. This gives rise to the dramatic improvement in dynamic performance shown by the shake table tests.

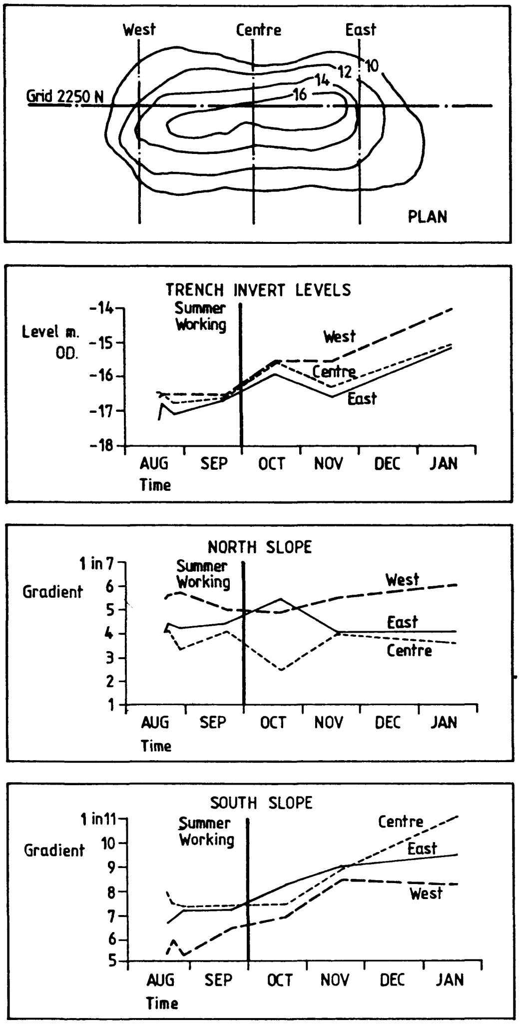


Fig. 1

D. T. WILLIAMS, Paper 21

Nuclear Design Associates (NDA), in fulfilling the role of consultant to Nuclear Electric , are responsible for the design of all civil engineering works associated with Sizewell 'B' Power Station. These responsibilities include the design of a record-breaking diaphragm wall dewatering scheme, a roll on - roll off berth for the delivery by sea of large loads to the site, the highly sophisticated analysis and design of all buildings within the

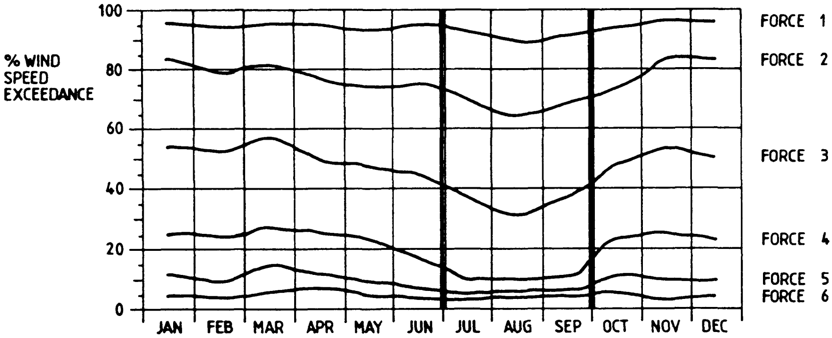


Fig. 2

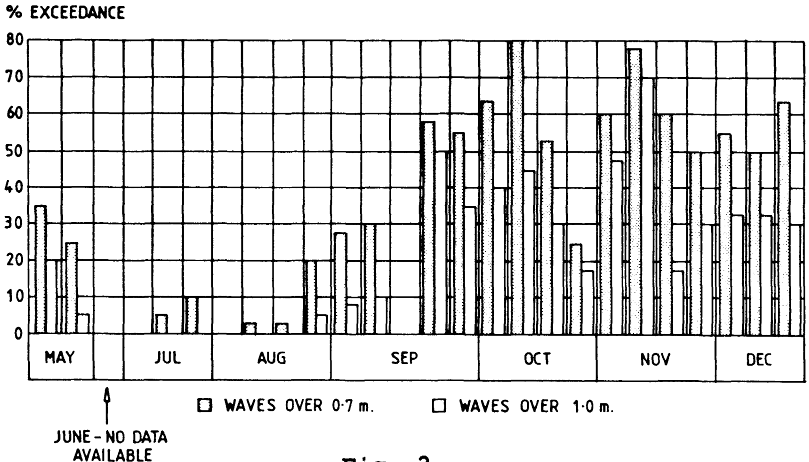


Fig. 3

mainstation complex and the preliminary investigation and final design of the offshore cooling water tunnels.

As outlined in the Paper, the cooling water tunnels required considerable preliminary investigation and analysis of data to confirm that the most cost-effective tunnel construction method identified could be adopted in the prevailing conditions at Sizewell.

Figure 1 shows the results of the trial trench monitoring plotted to show slope stability and trench siltation, indicating that during the summer months the trench sides form stable slopes, and that siltation would be acceptable and could be dealt with over the construction period. Fig. 2 is a plot of the average monthly wind speed over a ten-year period adjacent to Sizewell, from which it can be seen that there is a period during the summer when the wind speeds are significantly lower than the annual average, and during which marine operations could be undertaken. Fig. 3

shows a corresponding decrease in wave height during the same months measured at Sizewell, confirming that acceptable environmental conditions could be expected during a typical summer season. The wave records were also investigated to identify the extent and frequency of periods when waves did not exceed the acceptable working limit of 0.7 m for unit placing and 1.0 m for dredging and general marine work. The results confirmed that there were likely to be sufficient occasions when offshore work could be carried out during the calmer weather.

On the basis of these and other investigations including site geology, programme timing, unit stability, and comparative costings, as outlined in the Paper, NDA concluded that the submerged tube tunnel scheme was a cost-effective and practical proposal for the construction of the offshore tunnels for Sizewell 'B' Power Station.

In reply to Mr Saunderson, at Sizewell only very minor differential settlement was anticipated. The formation is very dense sand with N values between 50 and 100, and the tunnel loading, when flooded and backfilled, does not exceed the original overburden. As the stone bed was placed in a continuous manner to produce a substantially uniform foundation and the tunnel units (100 m long) are able to span over minor variations, differential settlements are minimized. It was expected that the unit when initially placed would not induce any significant settlement, but after flooding the uncompacted stone bed under each unit would compact up to 20 mm. Experience on site did not invalidate these assumptions although minor siltation of the bed affected the final settlement readings. It was found that the sides of the rubber seal slid down the carefully prepared concrete face of the previously placed unit, but the top and bottom of the seal, being over 350 mm wide hollow sections, were not stiff enough to overcome the friction and flexed to take up the differential movement. However, the contact pressure induced by the jacks during joint compression ensured that the joint satisfied leakage criteria under these conditions.

Replying to Mr Jordan's question, divers employed by the contractor were in attendance during all the underwater operations, and after completion of each operation independent underwater inspection was carried out by Nuclear Electric divers. The work was always undertaken during acceptable environmental conditions, and therefore the actual handling of the units during lowering was under tight control. Sea bed anchors, sheaf blocks, mooring lines and sophisticated control of winches ensured accurate control of the units. Positioning by sighting towers and land-based survey stations, confirmed by diver observation as the temporary beam and Vee-notch guidance system was engaged during final lowering, ensured an accuracy at adjacent ends within 50 mm. At the free end, the unit was located to within 150 mm of the true line. After flooding and final tunnel completion, pre-cast concrete covers were placed over the joints to protect the rubber seal from backfill loads. These units allowed for a misalignment between units of up to 50 mm and each one was successfully placed without incident, confirming agreed tolerances had been achieved.

As far as Mr Drysdale's question is concerned, considerable measurement of the existing dispersal of the Sizewell 'A' discharge was undertaken, together with float tracking and current measurement, to establish the water movement regime offshore. This information was used to validate, as far as possible, a mathematical computer simulation developed by CERL, which was then used to predict the discharge plume parameters for Sizewell 'B'. Regarding water tightness of the structure, testing of each unit by inspection (by water testing of end bulkheads) and by water spray was carried out before unit launch. During storage prior to the tow to site, all units were monitored for leakage and found to be essentially water-tight. Full-scale testing was carried out on a section of the joint between units during development of the seal to demonstrate that when compressed the leakage was within acceptable limits. It should be noted that when flooded the differential head in the tunnel is nominal, and that once in position the tunnels will not be dewatered. Thus, minor seepage through the joints is tolerable provided that no fine backfill material is drawn into the tunnel. (The differential head on the tunnel is such that leakage will draw water into the intake tunnel and discharge water from the outfall tunnel).

The cooling water tunnels at Sizewell are required for the conveyance of cooling water to condensers of the turbo generator sets. As such they are no different from any other intake or outfall tunnel, and therefore in response to Mr Hills question, the use of boronated concrete was not considered necessary or in fact appropriate.

22. Long-term properties of concrete in nuclear containment structures

S. N. FIELD, MA(Cantab), MICE, and P. B. BAMFORTH, BSc, PhD, MICE, Taywood Engineering Ltd

SYNOPSIS. Over the last thirty years a large volume of testing has been carried out on concretes used in prestressed concrete pressure vessels and similar structures. The main aim of the work has been to provide the designers with a prediction method for elastic moduli and creep deformation which takes into account temperature and age at loading. This paper summarises and reviews the results from the six concretes tested by Taywood Engineering Ltd (T.E.L.), comparing mixes with and without PFA.

INTRODUCTION AND SCOPE

1. The prestressed concrete pressure vessel (PCPV) for nuclear reactors is almost unique in concrete civil engineering structures in that a detailed knowledge of the deformation behaviour of the concrete is an integral part of the design. To provide the design data on concrete properties, extensive testing has been undertaken on most of the vessel concretes, and in the thirty years since the concept of the PCPV has developed, a wealth of data has been collected. The pressurised water reactor of Sizewell B utilises a prestressed concrete containment structure which has several features in common with the PCPV, including a requirement for performance data on the concrete of which it is built. Testing of this concrete has also been undertaken.

2. This paper constitutes a review of the data accumulated to date. Aspects addressed include:-

a] How do the concretes tested compare and contrast?

b] What insight has been gained into the associated concrete technology that is useful in other fields?

c] What philosophy now (after thirty years experience) determines the scope of testing?

BACKGROUND TO PCPV's AND THEIR DESIGN

3. Although several different shapes of PCPV's have been used, most important features are common to all. The Heysham 2 vessel is described below as typical, and is illustrated in Fig. 1.

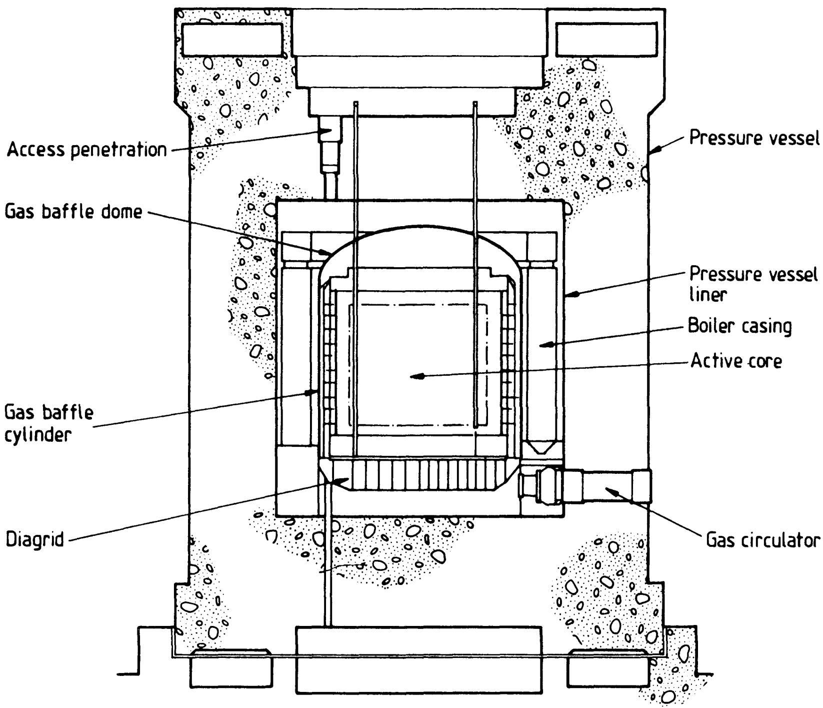


Fig. 1 Heysham 2 prestressed concrete pressure vessel

4. The internal cavity consists of a cylindrical space housing the reactor core, the boilers and the gas circulators. In normal operation, the gas circulates at 4.6 to 5 MPa, and this pressures acts on the vessel via a gas-tight liner. The gas pressure induces direct tensions together with bending and shear stresses. The gas circulates at temperatures of up to 635°C, but insulation and cooling pipes on the liner prevent the concrete reaching temperatures in excess of 60 or 70°C. The difference in temperature between the inside and outside of the vessel contributes additional stresses. To resist the tensions in the concrete induced by the mechanical and thermal loads, the vessel is heavily prestressed (by 3,600 vertical tendons). A typical field stress due to prestress would be 13 MPa.

5. The basis of design of a PCPV is to ensure that the vessel has a largely elastic response to all load combinations experienced during the life of a reactor. Field stresses are limited to 0.33 or 0.4 times the nominal compressive strength of the concrete, and to 1.4 or 2.8 MPa where tensile, depending on load case. Due to the different rates of creep of hot and cold concrete, prestress and pressure loads redistribute away from the hotter inside of the vessel towards the cooler exterior. It is the necessity

of predicting with reasonable confidence the extent of this load transfer (and ensuring that the permissible field stresses are not exceeded in the interim) that makes knowledge of the concrete deformation behaviour necessary.

6. For calculation of in-service stresses, the designer must also have knowledge of thermal properties such as expansion coefficient, conductivity and diffusivity. Estimates are also required of Poisson's ratio, (elastic and creep) and also long term shrinkage. The elastic stress distribution is not the only limit state of interest to the vessel designer. He must also demonstrate inter alia an adequate margin of safety to gross over-pressurisation of the vessel.

7. The first UK vessels were built at Oldbury (commencing in 1962). These were followed by Wylfa (1963), Dungeness (1966), Hinkley Point B (1967), Hunterston B (1967), Hartlepool (1968), Heysham A (1970), Heysham 2 (1978) and Torness (1978) (ref. 1). Understanding and design techniques were evolving continuously during this period. (refs. 2, 3, 4).

8. The most common method used for design of a PCPV is described in BS 4975 as the "effective modulus solution". The method makes use of the fact that all stresses are small in relation to the local strength, and that the loading of the vessel can be idealised as a small number of sets of load increments (each set being associated with some major event in the life of the vessel such as application of prestress, or thermal stresses from the first vessel heating). The distribution of elastic stresses for each load set at some time after loading is calculated (usually by a finite element computer programme or similar) using an effective modulus appropriate to each element. The effective modulus at some time after loading is defined as the reciprocal of the total specific strain (strain in response to a step load of unity) Total strain is defined as the sum of the initial elastic deformation and any subsequent creep. Once stresses for each load case have been calculated, several sets of stresses may be combined by superposition. For example, stresses at proof test would be the sum of the elastic stresses from the internal gas pressure computed via appropriate effective moduli (actually the short term elastic modulus), combined with the stresses from the prestress calculated via a different set of effective moduli reflecting the younger age at which the vessel was prestressed and the duration of the interval between prestress and proof test.

CREEP TESTING

9. The form of testing undertaken is determined by the need to be able to estimate the effective modulus at any time after any load case at any position in the vessel and for any subsequent local temperature history. In particular, the creep curve depends on the age and temperature of the

concrete at loading and the subsequent temperature of storage.

10. The first PCPV programmes in which the deformation behaviour of concrete was measured were Oldbury and Wylfa (refs 5 and 2). The programmes took the form of maintaining sealed concrete cylinders 300mm x 150mm diameter under a constant load at a constant temperature whilst strain was measured. For Wylfa, data was obtained at temperatures and ages-at-loading of 20, 40, 65 and 93°C, and 7, 28, 60, 180, 400 days and 12½ years respectively. The data was presented to the designers in the form of sets of curves of predicted specific strain against time under load (Fig 2).

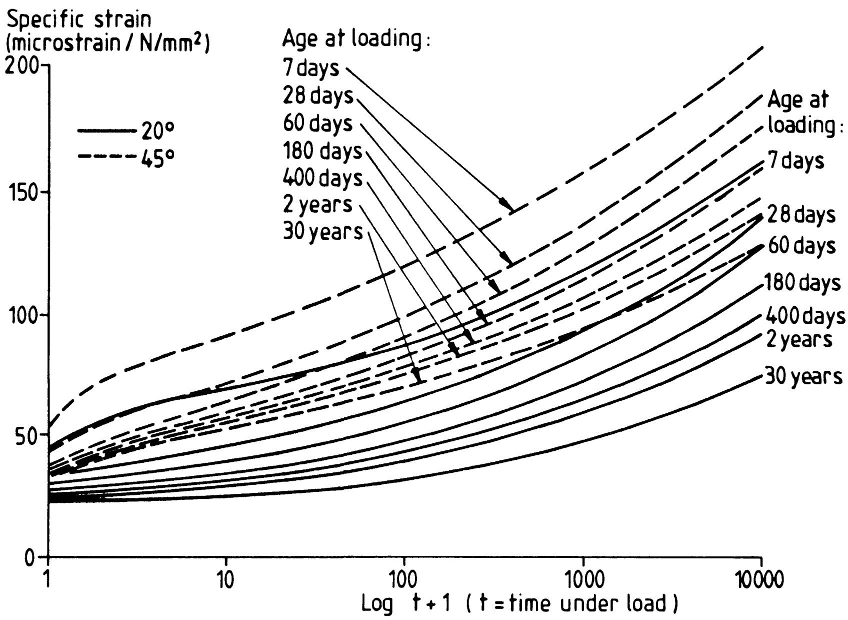


Fig. 2. Specific strain data for Wylfa at 20°C and 45°C.

11. Several features of this figure should be noted. Total strain is plotted, rather than attempting to isolate creep strain from elastic strain. The abscissae are plotted on a $\log_e$ (t + 1) scale (t is time under load), which also allows the initial elastic strain to be shown.

Using the Creep Data.

12. The concrete property test programme usually runs concurrently with vessel design, and only a limited duration

of data can be obtained before the design is finalised. Systematic monitoring of creep specimens has not generally been continued for longer than about two years after loading. This leaves the designer with a need to extrapolate perhaps two years of creep data to predict deformation after thirty years. In the early stages of the Wylfa programme, linearity of strain with log (time) was assumed. Longer term data revealed that this approach tended to underestimate strains at longer periods under load (say greater than 1000 days). It was later determined that a log-log plot (with straight line extrapolation up to thirty years), was a better fit of the measured data .

13. Creep is strongly dependent on the age at loading, and to obtain predictions of the effects of loading vessel concretes in late life (i.e. up to age 40 years), some scheme of extrapolation of data from young concrete must be employed. Linearity of effective modulus (for some time under load and temperature) with log-age-at-loading was assumed. Data supporting this approach was obtained from Wylfa specimens loaded at an age of 12½ years (ref. 6), and from the Hartlepool site specimens.

14. To generate an appropriate effective modulus for a vessel location for any time after load and age-at-loading for a particular load case, the effects of temperature must be considered. For concrete heated long before being loaded, allowance can be made by adjusting (on a maturity basis) the age-at-loading used to select data. Where the temperature varies significantly after loading, appropriate moduli can be selected from the experimental data (all of which is obtained at a constant temperature) to provide conservative upper and lower bounds on vessel behaviour.

General Features of Creep Data

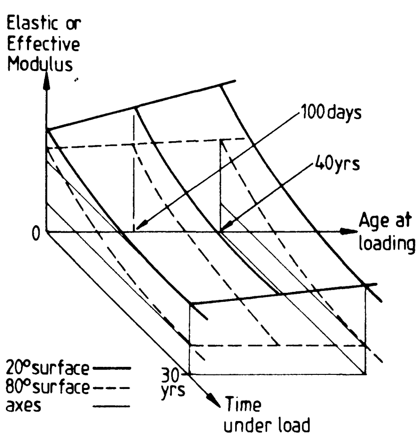


Fig. 3 Creep "surface" for 20°C and 80°C

15. Presentation of creep data is made difficult by the number of variables involved. Fig. 3 shows schematic creep "surfaces" for 20°C and for 80°C. The data is in the form of effective moduli (reciprocal of specific strain) plotted against time under load and age-at-loading. The moduli on the $t = 0$ plane are all initial elastic moduli.

16. Important aspects to note from Fig. 3 include the shape of the creep curve, (creep deformation after 30 years comparable with the initial elastic

deformation), the construction of curves extrapolated outside the range of experimental data (both for late ages of loading and for long periods under load), the temperature dependence (with greater deformation, i.e. lower moduli, at higher temperatures), and finally, the interaction between modulus, temperature and age-at-loading (the absence of age-at-loading effects in specimens loaded and maintained at high temperatures).

MODELLING THE CREEP CURVE

17. The discussion below is largely in terms of effective moduli: the formal definition is as follows:-

$$E_{\lambda T t} = \frac{\delta_L}{\varepsilon_{e\lambda} + \varepsilon_{ct}} \quad (1)$$

where

λ	=	age at loading
T	=	temperature of loading and of subsequent storage
t	=	time under load
$E_{\lambda T t}$	=	effective modulus appropriate to age λ temperature T and time under load t
δ_L	=	loading stress
$\varepsilon_{e\lambda}$	=	elastic stain on loading at age λ
ε_{ct}	=	subsequent creep strain up to time t

18. A number of effective moduli at differing times under load define the creep curve:-

$E_{\lambda T 0}$ (abbreviated to E_0)	=	initial elastic modulus for concrete loaded at age λ and at temp T
$E_{\lambda T 10}$ (abbreviated to E_{10})	=	effective modulus for age-at--loading λ and temperature T after 10 days under load.
$E_{\lambda T 100}$ (abbreviated to E_{100})	=	ditto but 100 days under load.

The effective modulus after t days under load (E_t) can be represented as follows:-

$$E_t = \frac{(E_{10})^2}{E_{100}} \, t^{-\log\left(\frac{E_{10}}{E_{100}}\right)} \qquad (t > 10 \text{ days}) \quad (2)$$

19. Eqn 2 reflects the fact that after about ten days under load, curves of effective modulus against time become linear when plotted on a log-log basis. The Heysham 2 and Torness programmes produced data at durations of load of up to 1400 days, supporting this approach. Eqn 2 is

also used for extrapolation beyond 100 days under load. Prior to ten days under load. any convenient interpolation between E_o and E_{10} can be used. The the full creep curve (for some temperature and age-at-load), from initial elastic deformation through to predicted thirty year creep, can thus be characterized by three numbers, E_o, E_{10} and E_{100}

REVIEW OF SIX VESSEL CONCRETES

Scope

20. T.E.L. have deformation data on all stations for which they have undertaken the concrete property testing, including the Sizewell B containment. Table 1 summarises the constituents and mix designs for the six concretes.

Table 1 Materials, mix designs and strengths of concrete subjected to creep testing by T.E.L.

	WYLFA	HARTLEPOOL	HEYSHAM A	HEYSHAM 2	TORNESS	SIZEWELL B
MATERIALS						
Cement	Padeswood	Weardale	Ribble	Ribble	Dunbar	Mason
PFA				FiddlersF	Longannet	Ironbridge
Sand	Igneous	Glacial	Glacial	S Dredged	Pit	S Dredged
Coarse agg	Limestone	Dolerite	Hornfels	Dolerite	Dolerite	S Dredged
Admixture	Ligno.	Ligno.	Ligno.	Ligno.	Ligno.	Ligno.
QUANTS/M^3						
Cement	410	420	435	270	340	215
PFA	-	-	-	90	115	140
Water	175	205	190	165	175	140
Sand	600	630	620	705	720	660
Coarse Agg	1215	1280	1190	1315	1090	1230
Density	2400	2535	2435	2545	2440	2385
w/c	0.42	0.49	0.44	0.46	0.39	0.40
a/c	4.40	4.52	4.15	5.56	3.98	5.30
PFA rep%	-	-	-	25	25	40
Strgth(MPa)	50	60	55	52.5	60.5	61

N.B.1 Water/cement and aggregate/cement ratios (w/c and a/c) refer to total cementitious content (inc PFA)

N.B.2 PFA rep% is the percentage of the total cementitious content that is pulverised fuel ash.

N.B.3 Mixes used in actual PCPV construction differ slightly (significantly for Heysham A) from laboratory mixes.

Data Presentation

21. Making comparisons between six sets of creep surfaces in the form of Fig 3 is not practicable and drastic steps must be taken to simplify the data. As noted in para 19, all creep curves can be defined by three values E_o, E_{10} and E_{100}. The effect of a variable such as temperature can be discussed in terms of its effect on these key moduli. To further reduce the data volume, no further reference will be made to E_{10}. (Although E_{10} is a

necessary parameter in fully describing the shape of the creep curve, its behaviour is an amalgam of that for E_o and E_{100}).

22. Standard conditions (namely 20°C, 80°C and age-at-loading 100 days) are used where practicable. For example, evaluation of the effects of temperature on E_o and E_{100} (interpolated where necessary) is made on specimens all loaded at age 100 days. Interpolation has been used where appropriate (eg where 100 days was not one of the ages of test in the experimental programme).

General Deformation

23. Fig. 4 shows E_o and E_{100} data at 20°C, and age-at-loading 100 days. The three concretes containing PFA are all stiffer than the concretes without PFA, the difference being more marked after a period under load. The effect of PFA in reducing creep has been previously reported (ref. 7). Strength, aggregate stiffness, and paste volume may all also be expected to affect creep, but the effect of PFA in reducing deformation and creep now seems strongly indicated.

24. Fig. 5 shows E_o and E_{100} against combined volume of water and Portland cement in the mix (i.e. not including PFA). The data is compatible with the view that neither PFA nor its hydration products tend to creep.

Temperature

25. Fig 6a shows the ratio of E_o at temperature to E_o at 20°C. Fig 6b shows similar data for E_{100}. It can be seen that when the data is presented in this way, all six concretes are broadly similar in behaviour. Higher temperatures make the concrete softer, and have a greater effect on E_{100} than E_o. E_{100} represents the "elastic plus creep" strain, and as creep is widely viewed as a thermally activated process, these effects are unremarkable. The use of PFA in 3 of the 6 concretes does not seem to influence the effects of temperature. This is consistent with the view expressed in para 24 - that there is a possibility that PFA plays no part in the creep process.

Effect of Age

26. Fig 7a shows the ratio of E_o for concrete loaded at a variety of ages, to the value of E_o on concrete loaded at age 100 days. All data is at 20°C. As expected, the older the sample when tested, the greater its elastic modulus. The apparent absence of an ageing effect between ages of load of 10 and 100-200 days is a common feature of heat cycled concretes (ref. 7). The form of relationship used for extrapolating late age-at-loading moduli is largely determined by the Wylfa 12½ year measurements, but is corroborated by Washa et al (ref. 8), on concretes 10½ years old when first loaded.

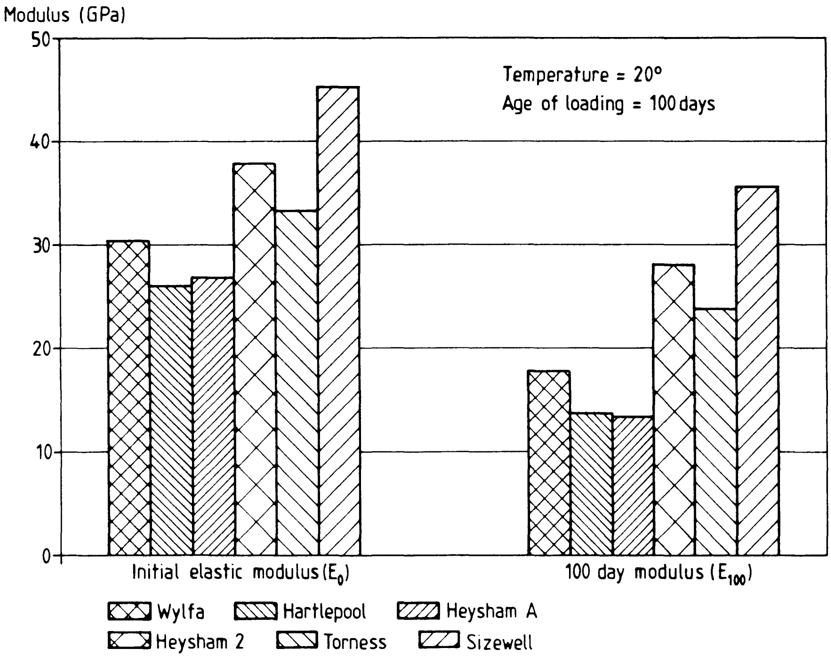


Fig. 4 Initial elastic modulus, and effective modulus after 100 days under load

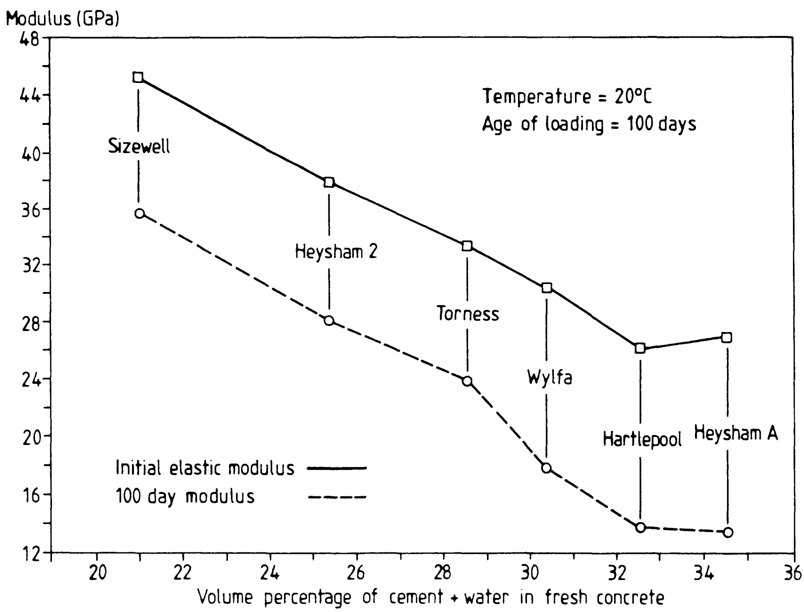


Fig. 5 Effect of paste volume (water + OPC only) on initial elastic modulus, and on effective modulus after 100 days under load

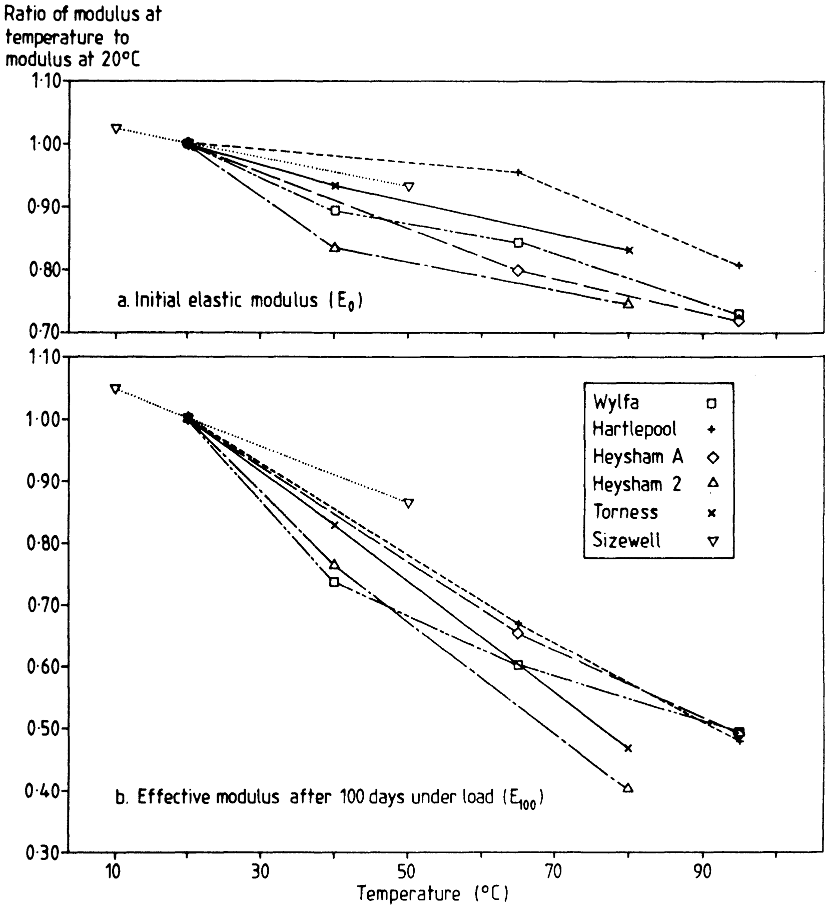


Fig. 6 Variation of modulus (initial elastic, and 100 day effective) with temperature

27. Fig 7b shows data for E_{100} (again at 20°C) in a similar form to that of Fig 7a. Some scatter is apparent and again it will be seen that the results from the Wylfa 12½ year data play an important part in determining how results obtained from young specimens are extrapolated to predict the effects of late-life stress changes. The need for more testing on specimens aged 10 years or more is clearly indicated.

Interaction of Temperature with Age of Loading

28. Fig 7d shows data for E_{100} from concrete loaded at several ages. All data is at 80°C and the data is plotted in a similar form to that of Figs 7a and b. Comparison of

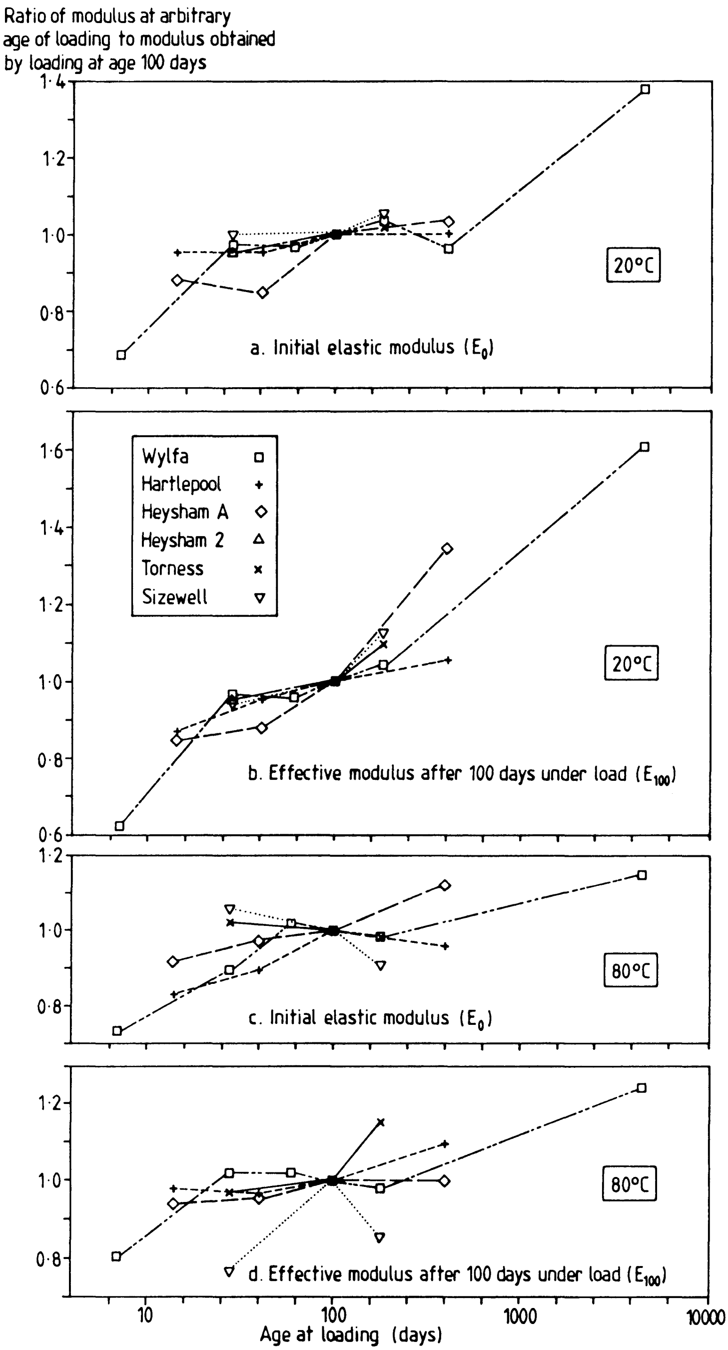


Fig 7 Variation of moduli at 20°C and 80°C with age at loading

Figs 7b and 7d show that whilst there is a significant age-at-loading effect at 20°C, there is little or no effect in specimens loaded and maintained under load at 80°C. Elastic modulus (shown in similar format) in Fig 7c also fails to increase with age in specimens tested at 80°C. (NB storage of all specimens prior to test is under sealed conditions at 20°C).

29. There are no clear reasons for this effect, although speculation is possible. Heating concrete must cause internal thermal stresses and some degree of microstructural damage. The effect of the "damage", (reduction of both elastic and creep moduli with increase in temperature) is greater in old specimens. It seems plausible that young creep-prone concrete is better able to accommodate damage, from temperature excursions, and that old concrete, whilst initially stiffer, is internally disrupted by heat, with a concomitant softening.

Thermal Properties and Shrinkage

30. Thermal expansion coefficients have been measured on all six concretes. Values are an average obtained from two or three temperature cycles between ambient and 80 or 95°C. Table 2 shows the coefficients obtained.

31. Thermal conductivity was determined by the methods of BS 874 for five of the concretes. (Table 2). Samples were tested in the "as received" condition, (sealed from casting) simulating mass vessel concrete, and also after drying out (to give a "worst case" for transient thermal stresses.)

Table 2 Thermal Properties

Station	Thermal expansion coefficient (Microstrain/°C)	Thermal conductivity W/m K	
		as received	dried out
Wylfa	8.5	1.9	1.6
Hartlepool	9.0	1.7	1.4
Heysham A	8.5	-	-
Heysham 2	10.5	1.8	1.3
Torness	9.0	1.6	1.2
Sizewell	13.0	2.4	-

32. Autogenous shrinkage (i.e. concrete movement that is not due to stress, temperature change or moisture movement) must be allowed for in design. For Wylfa (ref. 2) 400 microstrain was quoted as an upper limit for shrinkage in late-life concrete in the vessel maintained at elevated temperature.

33. Due to a lack of continuity in the concrete property test programme, little long term shrinkage data has been obtained. "Shrinkage" strains measured on unloaded control

specimens up to two years old, have steadily reduced from station to station, possibly reflecting improvements in the technology of sealing specimens. Worst-case values quoted to designers have been reduced commensurately, (currently 100 microstrain after 30 years). Validation of this approach is obtained from the results of the routine surveillance of the Wylfa, Hartlepool and Heysham A vessels - no significant loss of prestress has occurred due to unexpectedly large autogenous shrinkage of the vessel concrete.

Additional Testing

34. Various additional testing has been carried out, which for reasons of lack of space, cannot be fully described here. For Wylfa, Hartlepool and Heysham A, specimens of concrete were cast on site. These served to demonstrate that the laboratory data was representative of the behaviour of site concrete.

35. For Hartlepool, specimens seven years old were loaded, and then unloaded after 24 days under load. At 20°C, recovery moduli during unloading were similar to those that would be predicted for loading at that age: at 65°C, after only 24 days at elevated temperature, the concrete had become much stiffer during unloading than would have been predicted for that temperature and age.

36. For Heysham 2 and Torness, unloading of specimens maintained under load for a long period (4 years) generated an unloading modulus at 20°C comparable to the virgin loading modulus predicted at that age. For specimens loaded and maintained at load at temperatures of 40°C or greater, not only did the unloading modulus not reflect the usual softening effect of elevated temperature, but the concrete had become stiffer than a hypothetical specimen maintained at 20°C.

37. The long-duration creep data for Heysham and Torness have already been mentioned. For Heysham, certain specimens were heated whilst under load. No evidence was found that this process induced extra deformation (termed transitional thermal creep in the literature). For three stations, measurements were made of instantaneous Poisson's Ratio. Values of 0.25 (Heysham 2) 0.28 (Torness) and 0.23 (Sizewell) were recorded at ambient temperatures.

CONCLUDING REMARKS

38. The concrete property test programmes to date have resulted in the development of a clear picture of the interaction of temperature and age-at-loading with basic creep. Two minor areas of uncertainty remain however:-

39. The elastic moduli with which the vessels respond to proof testing (largely an "unloading" condition) tend to be slightly greater than would be predicted from the concrete property test programme. The lack of experimental data at late ages-at-load (particularly for concretes containing PFA) has already been remarked upon. Suitable specimens are

available for testing. There has also been speculation at various times that two moduli (one for loading and a stiffer one for unloading) are required to adequately describe concrete behaviour. The limited data described in paras 35 and 36 do not support this view, fortunately, as such non-linear behaviour would call into question the applicability the principle of superposition in design.

40. Temperature effects are not yet fully quantified. Evidence is gathering that maintaining a specimen at high temperature results in increased stiffening (possibly including healing of thermally induced damage) that is greater than the simple increase of maturity would indicate.

41. For future structures, it is useful to consider the volume of testing required in the light of knowledge gained to date. Some testing must take place - the differences in moduli in Fig. 4 are very significant to the designers. Creep testing at 20°C and at least one age of load is clearly indicated. Although the relationships in Figs 6 and 7 provide a model from which effective moduli for any temperature and age of load can be estimated from one creep test at 20°C, concrete is not always predictable. The authors believe that testing at a limited number of conditions should continue to be carried out to confirm the established pattern of temperature and age effects.

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23. Long-term performance of PCPVs for nuclear reactors

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SYNOPSIS. In the 1950s, progressive developments in gas cooled reactor technology reached the point where these reactors could no longer be contained in steel pressure vessels with an appropriate level of assurance concerning their long-term integrity. To respond to this situation the prestressed concrete pressure vessel (PCPV) was conceived and developed, fulfilling the dual function of pressure vessel and nuclear radiation shield. The technologies of civil engineering were thereby introduced into an area formerly dominated by mechanical engineering. This paper reviews the performance records of PCPVs and compares these with the original predictions, in respect of movements, concrete stresses and the loads carried by the vessel prestressing systems. It shows that the allowances incorporated in the design of PCPVs for their long-term behaviour were, in general, sensibly conservative and that these structures can be confidently expected to remain in a satisfactory condition for the foreseeable lives of the reactors they contain.

INTRODUCTION

1. All nuclear reactors are enclosed in pressure vessels whose primary function is to contain the fluid used to transfer heat from the reactor to the primary heat exchangers. Reactors cooled by gas require much larger pressure vessels than those cooled by water and the early gas-cooled reactors were generally contained in steel spheres. Immediately outside these spheres were massive concrete structures which provided a shield against nuclear radiation emitted by the reactor.

2. Progressive increases in the size and operating pressure of gas-cooled reactors demanded ever larger and thicker steel pressure vessels. Eventually a point was reached where the demands outstripped the current state of steel technology and the era of prestressed concrete pressure vessels (PCPVs) was born. These structures were able to fulfill both the pressure containment and radiation shielding functions previously met by separate structures. They also contained the heat exchangers,

previously housed in separate structures, with consequent benefits to both operational efficiency and safety.

3. Over a period of about twenty five years, 27 PCPVs were built in five countries, the majority being in the UK and France. The earliest vessel, at Marcoule in France, was taken out of service in 1985 after an operational life of 28 years. Some of the other reactors contained in PCPVs have also been shut down, for reasons unconnected with the performance of the PCPV itself.

4. Throughout their lives, all PCPVs have been regularly inspected to ascertain whether their behaviour is in accordance with design predictions and to confirm that they are fit for continued operation. No other structures are subjected to such close scrutiny and the records provide a unique history of the in-service performance of large prestressed concrete structures.

TYPES OF PCPV

5. The majority of PCPVs are essentially vertical cylinders, with internal diameters in the range 10-24m, internal heights from 15m to 40m and wall and cap thicknesses generally between 3m and 6m. There are three exceptions; the vessels at Marcoule (France), the earliest PCPVs, are horizontal cylinders; the vessels at Wylfa (U.K.) are spherical, with an internal diameter of about 30m; the vessels at Hartlepool and Heysham 1 are vertical cylinders with thickened walls in which are formed eight cylindrical cavities for the boilers, with the intention that these could be removed for maintenance or replacement should the need arise.

6. All the PCPVs are massive structures with concrete volumes ranging from 12000 to 25000 cubic metres and up to 2500 tons of prestressing steel. Their primary dimensions are shown in Fig. 1.

REQUIREMENTS AND PROVISIONS FOR IN-SERVICE SURVEILLANCE

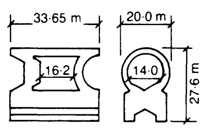
U.K.

7. Introduction Nuclear Electric plc (NE) is responsible for the operation of seven Nuclear Power Stations with PCPVs in England and Wales; each station has two PCPVs. These stations are at Oldbury-on-Severn, Wylfa, Hinkley 'B', Dungeness 'B', Hartlepool, Heysham 1 and Heysham 2. Scottish Nuclear Electric (SNE) operates two stations with PCPVs at Hunterston 'B' and Torness.

8. Surveillance Programme The Nuclear Site Licence Conditions, under which NE are the Licensees, requires that PCPVs are inspected on a regular basis. These requirements are detailed in the Maintenance Schedule applicable to each station. Each reactor is shut down once every two years so that each year there is an inspection of one vessel at each site.

9. This normally coincides with the scheduled shut-downs for reactor maintenance when the reactor and the internal parts of the vessel are inspected and serviced. A report

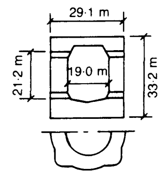
UK



Type of reactor: Magnox
No. of units: 2
Date of criticality: 1958
Working pressure: 147 N/mm^2
Gross therm/net elec. o/p: 260/36 MW

Marcoule G2.G3

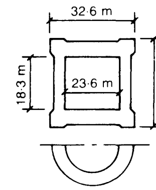
F



Type of reactor: Magnox
No. of units: 1
Date of criticality: 1966
Working pressure: 2·65 N/mm^2
Gross therm/net elec. o/p: 1580/480 MW

Chinon – A3

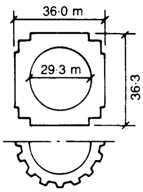
UK



Type of reactor: Magnox
No. of units: 2
Date of criticality: 1967
Working pressure: 2·52 N/mm^2
Gross therm/net elec. o/p: 750/300 MW

Oldbury

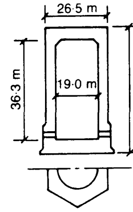
UK



Type of reactor: Magnox
No. of units: 2
Date of criticality: 1969
Working pressure: 2·65 N/mm^2
Gross therm/net elec. o/p: 1800/590 MW

WYLFA

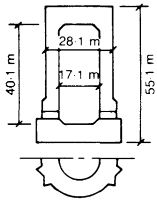
F



Type of reactor: Magnox
No. of units: 2
Date of criticality: 1969
Working pressure: 2·70 N/mm^2
Gross therm/net elec. o/p: 1680/480 MW

St. Laurent

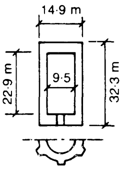
F



Type of reactor: Magnox
No. of units: 1
Date of criticality: 1972
Working pressure: 4·5 N/mm^2
Gross therm/net elec. o/p: 1954/640 MW

Bugey – 1

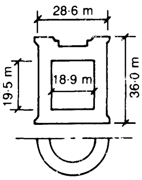
USA



Type of reactor: HTR
No. of units: 1
Date of criticality: 1974
Working pressure: 4·86 N/mm^2
Gross therm/net elec. o/p: 842/330 MW

Fort St. Vrain

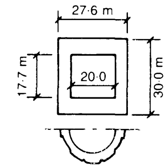
UK



Type of reactor: AGR
No. of units: 2
Date of criticality: 1974
Working pressure: 4·04 N/mm^2
Gross therm/net elec. o/p: 1440/625 MW

Hinkley 'B'/Hunterston

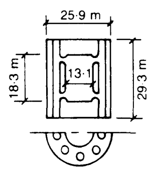
UK



Type of reactor: AGR
No. of units: 2
Date of criticality: 1983
Working pressure: 3·30 N/mm^2
Gross therm/net elec. o/p: 1480/600 MW

Dungeness 'B'

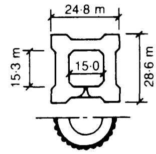
UK



Type of reactor: AGR
No. of units: 4
Date of criticality: 1983
Working pressure: 4·04 N/mm^2
Gross therm/net elec. o/p: 1600/625 MW

Hartlepool/Heysham 'A'

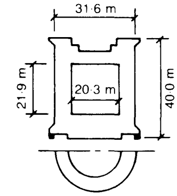
D



Type of reactor: HTR
No. of units: 1
Date of criticality: 1983
Working pressure: 3·92 N/mm^2
Gross therm/net elec. o/p: 760/298 MW

Schmehausen

UK



Type of reactor: AGR
No. of units: 4
Date of criticality: 1987
Working pressure: 4·14 N/mm^2
Gross therm/net elec. o/p: 1660/815 MW

Heysham/Torness

Figure 1: Prestressed Concrete Reactor Vessels. Typical Outlines.

is submitted to HM Nuclear Installations Inspectorate of the Health and Safety Executive summarising the results of the examinations carried out on the vessel. The report may recommend repairs or improvement work.

10. Activities Each inspection includes the following:

(a) Concrete Surface Examination A visual survey of the accessible vessel surface is undertaken and any increase in cracks or new cracks are reported. Cracks which are considered to be propagating are measured and a graph plotted to determine the propagation rate.

(b) Tendon Anchorages A sample consisting of a minimum of 1% of all anchorages is inspected for signs of damage, corrosion, slippage, or other forms of deterioration.

(c) Tendon Load Checks All the U.K. PCPVs have unbonded prestressing systems. Load checks are carried out by "lifting-off" a number of tendon anchorages in a sample consisting of a minimum of 1% of all tendons. These load checks are tabulated and presented in graphical form so that trends can be compared with design expectations and the maximum permissible limits of loss of load.

(d) Corrosion Examinations Strands or wires are removed from a number of tendons and inspected for signs of deterioration in their protective coating and underlying corrosion or mechanical damage. They are tested to determine their mechanical properties and cumulative records of any corrosion found are kept in a statistical format for comparison with permissible levels.

(e) Foundation Settlement A precise survey of levels is undertaken and the results presented in graphical form so that current and extrapolated settlements can be compared to design expectations and maximum permissible limits.

(f) Vibrating Wire Strain Gauges (VWGs) Embedded VWGs are read at monthly intervals. At approximately 5 yearly intervals, a full report is presented comparing VWG readings with a theoretical strain analysis, which takes account of time-temperature dependent tendon relaxation and concrete creep, as further evidence of continued operation within expected limits.

(g) Vessel Temperatures Thermocouples measuring concrete temperatures are read regularly and checked for conformity with the operating rules for the vessels. Particular attention is paid to hot-spots, areas around steam pipework penetrations, and the stand-pipe zone.

(h) Vessel Cooling Water System Routine visual inspections of tendon anchorages and pipework penetrations are carried out in order to detect evidence of water leaks from the vessel liner cooling system.

(i) Deflections On AGR vessels only, measurements of top slab deflections are read and compared with readings taken during the Proof Pressure Test of the vessel.

France

11. Introduction At the beginning of 1990 EdF operated 4 natural uranium gas graphite reactors: Chinon A3,

St Laurent A1 and A2 and Bugey 1. Two of these, Chinon A3 and A1, have now been taken out of service. Elsewhere, by agreement between EdF and HIFRENSA, the surveillance of the Vandellos 1 reactor in Spain is carried out by EdF.

12. Extent of Surveillance The vessels have been surveyed, since their construction, with the help of monitoring equipment as described below. In addition there is an annual visual examination of the external surface of the vessels.

(a) Foundation Settlement The settlement and deformation of the vessel foundations are measured by surveying instruments. These measurements also provide some information on the tilt of the vessel.

(b) Change of Shape of the Structure The variation of diameter and the tilt of the vessels are obtained using plumb-lines and the variation of height by means of vertical invar wires. Horizontal invar wires are used to measure the change of diameter of the base and of the top slab of some of the vessels.

(c) Internal Concrete Strains Local strains in the body of the concrete are measured by a system of VWGs orientated in different directions and distributed at appropriate locations within the vessel concrete.

(d) Concrete Temperatures The thermal state of the vessel is measured partly by the VWGs and partly by a network of thermocouples.

(e) Tensile Forces of Prestressing Cables A limited number of prestressing cables were injected with grease and instrumentated with load cells at their anchorages.

13. The output of the above instrumentation is recorded once a month, except settlement measurement, which is generally recorded once a year. In addition, at the time of programmed shut-downs and start-ups, the instrumentation is read at different values of vessel pressure. All the results which are obtained are examined at the time and then reviewed and summarised every two years in a surveillance report, in which a statement on the behaviour of the vessel is presented.

14. For those vessels which have been shut-down, a reduced programme has been established, in which part of the instrumentation is read at less frequent intervals.

15. Interpretation of Results

(a) Initial Processing Initially the output of the monitoring instrumentation is translated into dimensional changes of the vessel. For example, the readings from the VWGs are interpreted as local strains in um/m. This first stage constitutes the acquisition of basic data.

(b) Statistical Treatment of Data The intended objective is to increase the extent of information that can be obtained from the basic data. The method which is used also facilitates the detection and correction of erroneous or biased measurements.

16. The surveillance results obtained for a vessel are related to the conditions in which they are obtained,

particularly, the thermal state, the internal pressure and elapsed time since initial measurements. Experience has shown that basic data provides, to a first approximation, data on three principal types of behaviour:

- one associated with irreversible trends of time-dependent phenomena characterised by a function $f_1(t)$.
- two associated with reversible effects, i.e. variations of internal pressure $f_2(P)$, and temperature $f_3(\emptyset)$.

17. Each vessel parameter (X) is therefore represented by the formula: $X = f_1(t)+f_2(P)+f_3(\emptyset)+\epsilon$, where ϵ is the component of X caused by experimental errors and negligible secondary effects. Each of these functions can be expressed in the following manner:

$$f_1(t) = b_1e^{-t} + b_2t; \qquad f_2(P) = b_3P$$

for $f_3(\emptyset)$ an analysis has shown that only a limited number (5-6) of characteristic temperatures are needed in the function $f_3(\emptyset)$ which can therefore be written:

$$f_3(\emptyset)=b_4\emptyset_1+\ldots\ldots+b_{m+3}\emptyset_m \quad (m\leq 6)$$

18. The complete expression is therefore:

$$X = A+b_1e^{-t}+b_2t+b_3P+b_4\emptyset_1+\ldots+b_{m+3}\emptyset_m+\epsilon ,$$

where A is a constant dependent on initial conditions.

Coefficients A and b_1 are calculated from a representative sample of output obtained at different conditions of pressure and temperature (at least 30 observations). A statistical method is used based on linear regression using the method of "least squares".

19. Assessment of Time-Dependent Effects In order to identify time-dependent effects, a comparison is made of results obtained on different dates. The method adopted uses the results of the statistical analysis, allowing for thermal and pressure corrections and expressing the values which are obtained at identical conditions of temperature and pressure. These are expressed by the formula:

$$X' = X - (b_3P+b_4\emptyset_1+\ldots\ldots +b_{m+3}\emptyset_m),$$

for which: $X' = A + b_1e^{-t} + b_2t +\epsilon$.

The term X' is therefore independent of variations of temperature and pressure.

SURVEILLANCE RECORDS

U.K.

20. Concrete Surface Cracks less than 0.5mm in width not associated with changes in section or areas of high stress combination are categorised as not significant. Cracks between 0.5mm and 1.0mm in width are monitored for abnormal crack development. Cracks greater than 1.0mm in width are assessed utilising demec gauges, photography

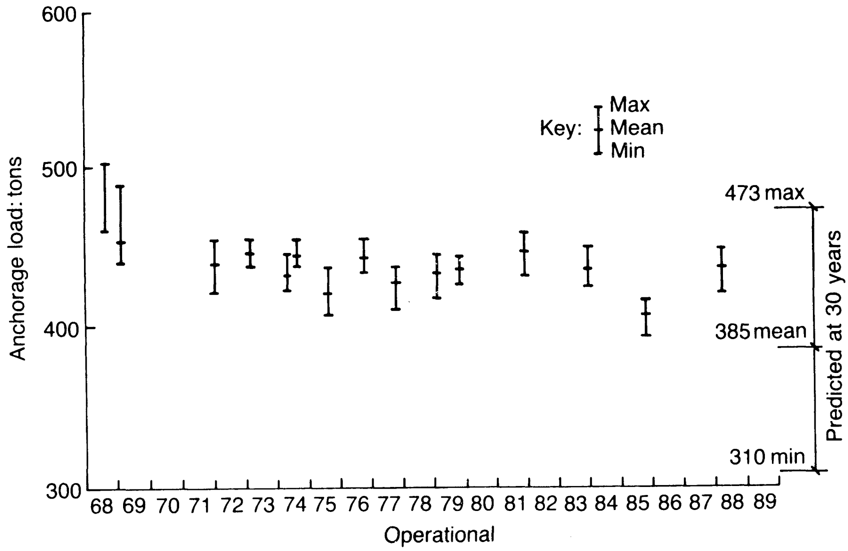


Fig 2. Lift-off loads for top cap tendons; Wylfa PCPV 1.

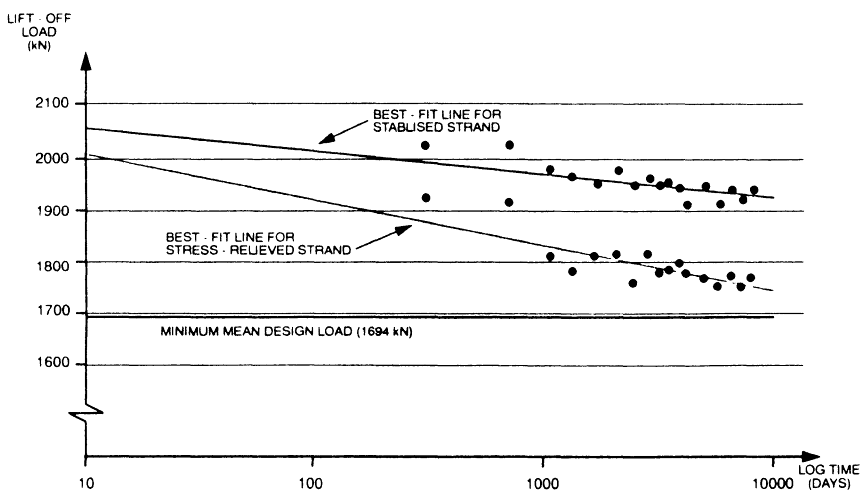


Fig 3. Lift-off loads for helical tendons; Oldbury PCPV 2.

etc., reference being made to the vessel analysis for possible high areas of stress.

21. An assessment has been made of the crack measurements taken during the service life of the Oldbury and Wylfa vessels. A comparison was made between crack widths when the PCPV was pressurised and unpressurised. The results of this work confirmed that a small number of cracks were propagating but that the growth rate was so small (less than 25 microns per year) that it was of no structural significance. The change in crack width with pressure was insignificant.

22. Concrete surface examinations have shown that surface cracking on PCPVs is confined to drying shrinkage and thermal strain effects and no instances of structurally significant cracking have occurred.

23. Tendon Load Checks The main purposes of load checking are to verify that the mean prestressing load on the vessel is within the design limits and to detect any unexpected high loss of mean load or of individual tendon loads. For this reason a representative number of tendons are checked each year. They are generally taken from each of the major groups of tendons in a vessel and the sample size is statistically significant but not less than 1% of the total number of tendons in a vessel.

24. As a variety of prestressing systems are used in the vessels, the method of load checking differs in detail from one station to another. However, at all stations load checking is carried out by 'lift-off' methods using a prestressing jack.

25. The simplest method employed determines the load to trap and free a feeler gauge. This load is taken to represent the measured load at the tendon anchorage. Alternatively the lift-off load is found using displacement transducers mounted between the fixed and moveable components of the anchorage and a graph plotted of load against movement. As the load is initially increased, the deflection increases by a small amount prior to actual lift-off, due to elastic deformation of the anchorage components. At lift-off the curve becomes non-linear and thereafter is that due simply to extension of the tendon. The lift-off point is taken as the commencement of the non-linear part of the load-deflection curve. This method was developed by Taylor Woodrow Ltd for Wylfa, and a similar method is used for measuring the tendon loads at Hartlepool and Heysham 1.

26. The results of the lift-off load measurements for the top cap tendons of Wylfa PCPV 1 are shown in Fig 2. The range of measured loads is partly due to geometrical variations within this group of tendons.

27. Two different types of strand were used for the tendons of the Oldbury PCPVs and the results of lift-off load measurements of geometrical similar tendons are shown in Fig. 3. The lower line represents the lift-off loads measured on tendons which are made up of stress-relieved

strand, which would be expected to have a higher relaxation rate than stabilised strand. The results show that the claims made for the stabilised material have been borne out in practice, since the relaxation losses are considerably lower than those of the stress-relieved strand. Fig. 3 also shows that extrapolation of tendon loads indicates that load will be well above the design minimum load at the end of station life, and, therefore, restressing should not be necessary.

28. The data obtained from periodic jack lift-off load checking and load cell monitoring of the unbonded prestressing systems used in the UK PCPVs have shown that the design predictions of prestressing losses due to steel relaxation and creep of concrete have been conservative.

29. Tendon Corrosion The most frequent problem encountered with unbonded prestressing systems has been corrosion, particularly pitting attack. The pit depths measured on the external hoop tendons of the Wylfa PCPVs are shown in Fig. 4.

30. Pit depth measurement is made by means of a micrometer probe, and in some cases, by sectioning. Further examinations, where considered necessary, are carried out using electron microscopy and chemical analysis of corrosion products.

31. The major incidents involving corrosion have occurred during construction. In these cases, pitting corrosion had occurred and the main causes were a combination of moisture, chloride contamination and impressed electrical currents probably due to improper earthing of DC welding machines. DC welding is now prohibited in any PCPV construction areas and earthing of AC equipment must meet appropriate standards.

32. As a result of these experiences, later PCPV construction specifications were extended to include more stringent requirements for the storage, transport and installation of prestressing tendons. Particular emphasis is placed upon the use of approved greases or waxes and the control of humidity in long term site storage areas. In addition dry ambient air conditions have been established for tendons installed in the vessels.

33. As a rule stations sited on the coast are more likely to suffer from this type of corrosion. Statistical analysis of the severity of corrosion allied to mechanical and chemical testing has been used to set acceptance standards and, to date, wholesale replacement of tendons has not been necessary on any UK PCPV. Indeed, there is little reason to believe that replacement of unbonded systems is likely to be required over the normal design lifetime of PCPVs provided present standards are maintained. It would appear, however, that corrosion cannot be entirely eliminated and the inspection programme should include an adequate degree of sampling to confirm freedom from stress corrosion cracking, and analytical support to determine pit depth/strength relationships.

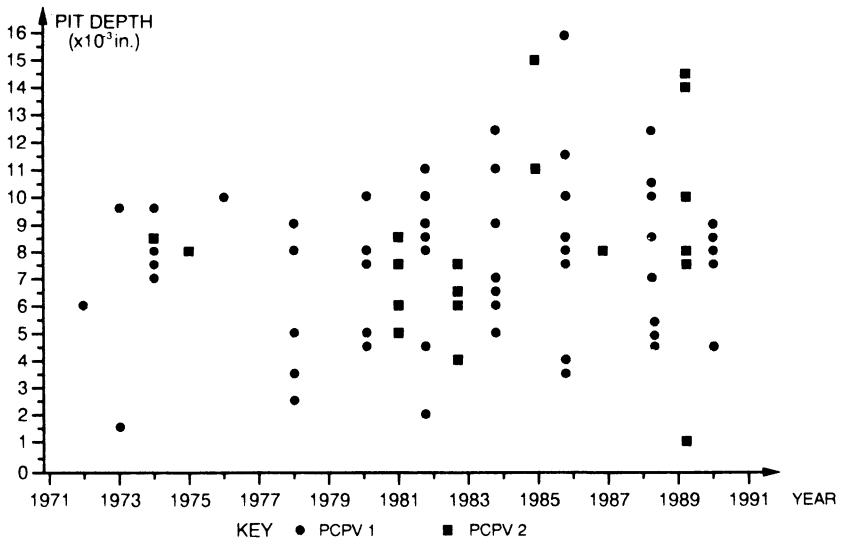


Fig 4. Pit depths measured on external hoop tendons; Wylfa.

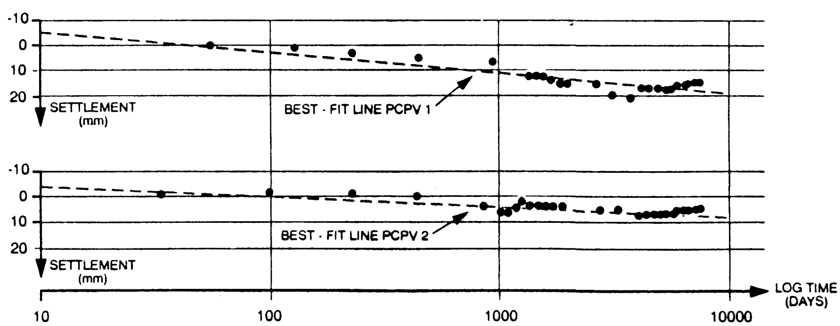


Fig 5. Mean settlement of PCPVs 1 & 2; Hinkley B.

34. Foundation Settlement Levelling is carried out using a precise optical level and invar staff, with closing errors generally less than 0.1mm. The allowable limits on PCPV foundation settlement and tilt are large compared to the movements that have been measured to date. The main concern from uneven settlement of the PCPV is associated with the structural strength of the reactor core and its supports together with ensuring the uninhibited passage under the influence of gravity of the reactor control rods.

35. Foundation settlement surveys have revealed few problems, although in one instance remedial work was required when unexpectedly large differential settlement occurred. The settlement records for the PCPVs at Hinckley B, shown in Fig. 5, are typical of most UK PCPVs.

36. Vibrating Wire Strain Gauges The original purpose of the cast-in strain gauges was to demonstrate that the vessel behaved elastically and in accordance with design predictions during the proof pressure test. However they have proved of considerable value in understanding the operational behaviour of vessels and are now monitored regularly throughout the operational lifetime of each vessel. Readings are taken at intervals of not less than 1 month and at the start and end of each reactor shut down and start-up cycle.

37. Continuous strain histories are maintained and typical plots comparing measurements with prediction are included in the annual surveillance reports.

38. Strain gauge readings have shown that theoretical predictions of operational deformations of been borne out during actual service. Fairly high numbers of gauges were found to fail or to malfunction in early stations but improvements in moisture proofing and electrical circuit testing during construction have now reduced gauge failures to acceptable levels.

France - General

39. Behaviour of Vessels in Response to Varying Pressure Variations of pressure arising from reactor start-ups cause an overall expansion together with some local deformations, an increase of diameter and, sometimes, some slight tilt of the structure. The reversible variations of diameter observed at mid-height of the barrel of all vessels are about 0.7mm/bar.

40. Behaviour of Vessels with Respect to Time The results obtained over the 20-25 year operational lives of the vessels shows that they have behaved satisfactorily. Changes caused by shrinkage have considerably slowed, and completely stopped on the reactors at Chinon, Bugey and Vandellos. In contrast, surveillances of the two reactors at St Laurent have shown that, over the last 15 years, they have been expanding at a linear rate; the vessel at St Laurent A1 being more affected than that at St Laurent A2. In consequence the results for the vessels at Chinon,

Vandellos and Bugey are presented separately from those for St Laurent A1 and A2.

Chinon, Vandellos and Bugey

41. Foundation Settlement Only the reactor at Bugey is provided with means of monitoring foundation movements. The results obtained since construction not showing any significant trends

42. Overall Structural Behaviour The overall decrease in diameter has remained extremely small, of the order of 2-3mm at most, generally at mid-height of the barrel; at the present time they are almost completely stable. Furthermore, the tilt measured since construction has been very modest, amounting to 1mm/10m at Chinon and Vandellos and 2.5mm/10m at Bugey.

43. Internal Concrete Strains Concrete strain measurements have shown, in the vertical and tangential direction, after about 20 years, an average shortening of 150-200um/m in the barrel sections of the vessel.

44. Most vessels have suffered a substantial loss of VWGs (47% at Bugey, 27% at Vandellos and 39% at St Laurent A2) and, to date, there is no certain explanation for this. Accelerated irradiation tests, carried out on laboratory test apparatus, have shown some degradation, but the extent is not sufficient to explain the extent of the VWG failure.

45. Concrete Temperature The general trend of temperature measurements shows that, during reactor operation, the concrete temperatures in the three vessels lie between 25-40^{o}C. Average gradients through the concrete, between the internal and external surfaces, are about 10^{o}C. However, at Vandellos, in the bottom cap, the data from some thermocouples shows that the operating temperatures are of the order of 50-70^{o}C.

46. Variations of Prestress Load At Chinon, there has been practically no change of prestressing tendon load since 1969. Before that, a fault in the dynamometer assemblies prevented rigorous monitoring of prestress losses during the first three years of operation. The nominal tensions which have been measured are between 870 and 900kN, which represents a reduction of tension since construction of about 230-260kN, from the nominal initial load of 1130kN.

47. At Vandellos, the reduction of tension recorded on the monitored cables, since vessel commissioning in 1971, ranges from 35kN to 285kN, from an average initial tension of about 2100kN. The lowest recorded tendon force, at the end of 1990, was 1790kN.

48. At Bugey, the average reduction of load, between 1971 and 1990, has been about 160kN, compared with an average initial load, when the vessel was commissioned, of about 2350kN.

49. No significant change of force has been observed at these two stations for the past ten years.

50. Extent of Cracking The surveys at Chinon has

identified about a dozen cracks which can be related to the states of mechanical stress of the vessel. They are evenly distributed over the four external faces of the vessel. Their lengths vary between 0.5m and 6m and their widths do not exceed 0.4mm. They are, for the most part, horizontal. At Bugey, in contrast, the extent of vessel cracking in primary areas of the structure is practically non-existent. There are insufficient recent surveys of Vandellos to permit a meaningful statement on cracking.

St Laurent A1 & A2

51. Foundation Settlements The vessel for Reactor A1 has shown practically no settlement since construction, whereas Vessel 2, shows an average settlement of 12mm since 1981, accompanied by a tilt of 1.5mm per 10m. This has clearly slowed down since 1985.

52. Concrete Temperature The measured temperatures on these two vessels are directly compared to those observed on the vessel at Vandellos, with values ranging from 25-40°C, up to a maximum of 50-60°C in the bottom cap. In addition, there is evidence that on this type of vessel, the variation of temperature measured during a reactor start-up is about 20°C in the bottom cap, between 10 & 15°C in the region of gas circulators and between 5-8°C in the barrel.

53. Concrete Strains, Overall Structural Deformation, Prestressing Tendon Loads As mentioned above, the surveillances of these two vessels show that they are expanding at a uniform rate without any sign of slowing down. The vessel at St Laurent A1 is more affected by this phenomena, almost 95% of the VWGs show some expansion, compared with 60% on Vessel 2. The observed rate is also higher on Vessel 1. 46% of the gauges are indicating between 4-10μm/m/year, compared with 45% showing between 0-0.6μm/m/year on Vessel 2.

54. These indications are confirmed by the change of diameter measured at mid-height of the barrel of 0.5mm per year at A1 and 0.3mm per year at A2.

55. Furthermore, there is a small increase in force in the majority of prestressing cables, with magnitudes generally of the order of 50kN since construction of the two vessels. However, on Vessel 2, one dynamometer installed on a vertical cable shows an increase of about 200kN since construction.

56. Laboratory analyses have been carried out to attempt to find an explanation for this expansion. They have shown that the elements potentially necessary for the development of an alkali-silica reaction are probably present in the concrete. In addition, dimensional measurements have been made on a sample of concrete held for one year in water at 70°C, then for a further year in water at 40°C, but they have not provided evidence of significant change of length, probably because of the slight rate of development (of the order of 10μm/m/year on

average).

57. State of Cracking The extent of cracking on the two vessels at St Laurent is greater than that for the three other vessels. Surveys have shown about 400 cracks on each, about 30 of which are instrumentated to monitor their width. Since these instruments were installed in 1985, an increase in width has been observed, reaching a maximum of 0.2mm. Furthermore, each annual survey provides evidence of the growth in length of about 20 to 30 cracks as well as several new cracks. However, the change of width of the instrumentated cracks in response to variations of internal pressure (between atmospheric and operating pressures) has remained less than 0.1mm.

CONCLUSIONS

58. The majority of the reported deficiencies found during in-service inspections of the U.K. PCPVs have arisen from factors such as adverse construction and operating environments causing corrosion of prestressing materials, malfunctions of operating equipment causing contamination by water of chemicals and less than 100% inspection of components at the construction stage.

59. The results of the measurements which has been made during the lifetime of the French PCPVs have shown that they have also behaved satisfactorily throughout this period, even if those at St Laurent have experienced a small degree of concrete expansion.

60. The continuation of measurements after shut-down of the two vessels (that at St Laurent A2 is planned for 1992) at a reduced level will perhaps permit confirmation of the reduced rate of expansion which was observed at the time of a prolonged outage of Vessel 2, between the beginning of 1980 and the end of 1982.

61. The overall conclusion gained from the surveillances of both the UK and French PCPVs is that their performance has been extremely good. These structures more than satisfy the stringent standards the public have a right to expect in the design and construction of nuclear installations. On the infrequent occasions when incidents have occurred or shortcomings have been found in the structure and its component parts, these have had an insignificant effect on either safety or serviceability.

ACKNOWLEDGEMENTS

This paper is published with the permission of Nuclear Electric, Taywood Engineering and EdF.

Fig. 1 is taken from 'The design and construction of prestressed concrete reactor vessels, Addendum 1990,' published by The Institution of Structural Engineers for Federation Internationale de la Precontrainte.

24. Programme development for the controlled removal of contamination and activated materials during the decommissioning of nuclear facilities

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SYNOPSIS. Over the past decade Taywood Engineering Limited has been carrying out a development programme on the use of explosives for the controlled removal of activated and/or contaminated parts of nuclear facilities undergoing decommissioning. Factors likely to affect the concrete removal have been investigated including concrete removal from re-entrant corners, curved surfaces and ducts, and from behind reinforcement and steel liners. Procedures for achieving a required level of material removal without impairing the remaining structure has been developed. The application of finite element methods to this class of problem with emphasis on numerical modelling of the concrete material has been investigated.

INTRODUCTION

1. The use of explosives for the controlled cutting and partial removal of concrete structures is a technique being developed for the decommissioning of nuclear facilities (refs 1-4). Here the technique is being developed for the controlled removal of activated and/or contaminated materials from the insides of biological shielding structures whilst relying on the outer layers of non-activated or contaminated materials to contain the activity. As the development programme has progressed, the explosive technique has been shown to have many advantages for several tasks in the controlled dismantling of nuclear facilities, (refs 4-6).

2. Contrary to general opinion, the explosive technique uses, comparatively, only small amounts of energy in controlled material removal. This energy is, however, released in a very short time interval (tens of microseconds) resulting in the attainment of very high power and momentum densities for very short periods of time. This gives rise to impulsive rise in ambient air pressures (blast waves), structural shocks and debris impacts. For best results in nuclear facility decommissiong tasks, the controlling parameters to these factors must be regulated to ensure

safety.

3. To assist in the technology development programme, investigations have been carried out on the controlled use of explosives to cut and remove selected parts of biological shielding and/or reactor pressure vessel structures without impairing the overall containment integrity (refs 1, 3). This has included investigations into the cratering characteristics of explosive charges buried in concrete, concrete removal from re-entrant corners, curved surfaces and from behind reinforcement and steel liners and the drilling capacity of shaped charges.

4. To substantiate and add credibility to the development work being carried out, analytical studies have been performed to investigate the effects of firing an explosive charge buried in a concrete mass. Two commercially available computer program packages, ADINA and DYNA3D, have been used in these analytical studies. The analytical results were compared with experimental results obtained from field trials.

5. A particle size distribution analysis has been carried out on the debris produced by the explosive technique. To provide assurance, cross-checking and back-up to recorded data, various particle sizing equipments, which cover the size range of 0.01 - 15 microns, were used for the dust particle size assessment.

DEVELOPMENT PROGRAMME

6. For the complete decommissioning of nuclear facilities, it is essential that removal of the activated and contaminated material precede the general demolition of the concrete structures. This requires working from the inside outwards, without inflicting damage to the external shell of non-active concrete and losing containment integrity. Using explosives to dismantle these structures means that the activated and/or contaminated material will have to be removed in thin layers to ensure that venting cracks do not develop.

7. Initial tests investigated parametric effects to establish the influences of charge weight, depth of burial, concrete strength on the cratering characteristics of concrete. Using optimum conditions for depth of burial and charge weight (selected from Fig. 1) the effects on material removal of parameters such as charge separation distances, charge array pattern and firing sequence were investigated. Tests were also carried out to assess the most effective layout of charges for stripping concrete off in layers to form a composite crater.

8. Almost all tests were carried out using quarter scale replica modelling. The earlier models represented cores taken through a biological shield of two metres thickness. The models, therefore, were of 0.5 meters thickness with diameters of either 0.5 or 2.0 metres. During this series of

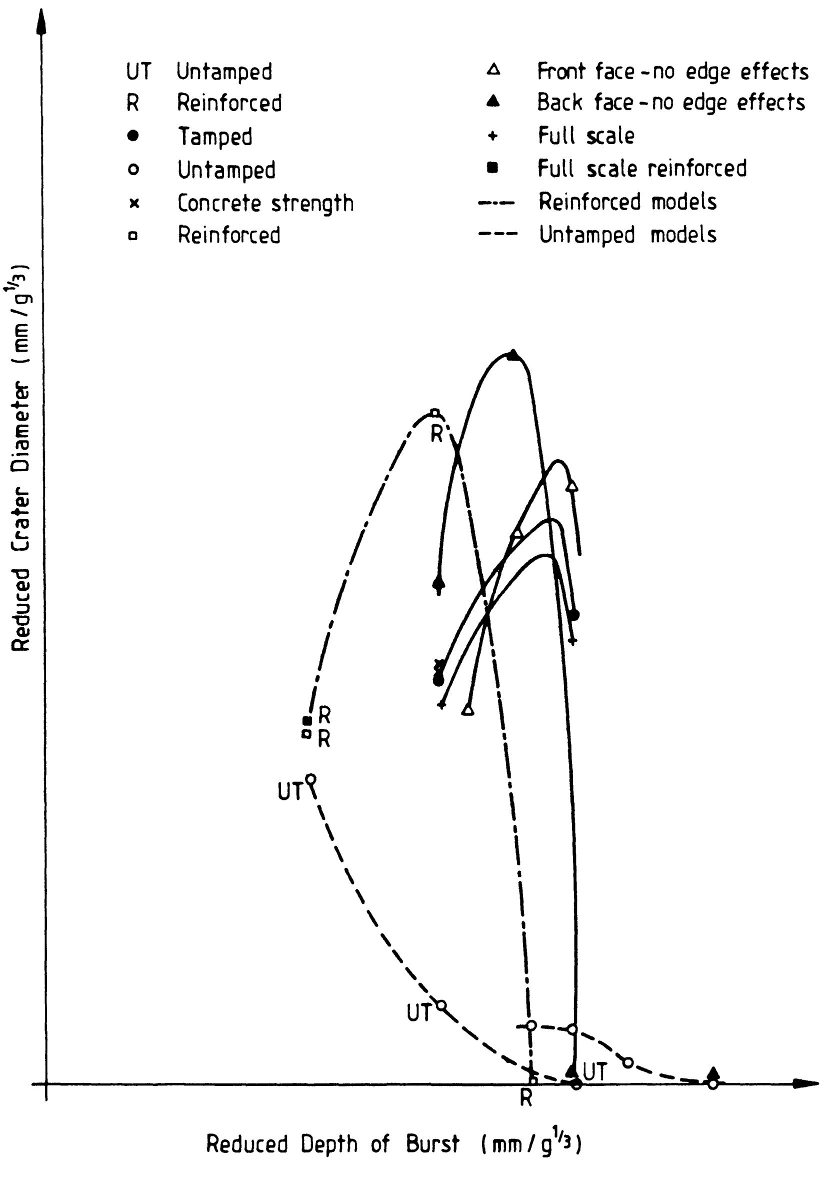

Figure 1. Reduced Crater Diameter vs. Reduced Depth of Burst

tests, a number of full scale tests were performed to confirm the validity of the modelling techniques and the replica scaling laws being used.

9. Representative quarter scale ring models of a biological shield was constructed to investigate curvature effects on the cratering characteristics, charge placement, multiple firing and multiple layer stripping characteristics. This was followed by tests on a closed right cylinder reinforced concrete model simulating at a quarter scale some biological shield characteristics. The model had an external diameter of 3.1 metres, an internal diameter of 2.1 metres and internal height of 3.12 metres and two end caps each 0.35 metres thick. Investigations were carried out on multiple layer stripping, charge hole preparation and the separation between charges to control the type of crater contours produced. The general observation made was that the tests on the complete cylinder essentially confirmed all the preliminary studies and demonstrated the feasibility of using small explosive charges for decommissioning purposes.

SPECIAL APPLICATIONS

10. Material removal from re-entrant corners, or near penetrations, gas ducts, etc, have always been recognised as posing special problems. The problems posed by geometrical shapes have been investigated through a series of field trials. These tests have included investigations into charge positioning to achieve material removal from re-entrant corners and the effects of charge orientation, separation from the re-entrant corner and firing sequence on the cratering characteristics. The feasibility of using multiple charges to facilitate the removal of material from re-entrant corners was demonstrated (see Fig. 2).

11. When material removal from positions adjacent to lined or unlined penetrations, gas ducts, etc, was investigated it was found that for smaller diameter penetrations the structural shape gives rise to a passive restraint against the formation of a crater. This results in smaller sized craters being produced in comparison with larger penetration results. It has been shown that it is feasible to remove material from the region of lined gas ducts although shock wave reflections can influence crater formation. Crater dimensions in the regions away from these ducts are only marginally affected by the presence of the ducts.

12. The influence of the presence of steel reinforcement has been found to be complex. The lateral reinforcement together with the shear bars stiffens the structure and thus prevents the concrete fracturing. On the other hand, if the lateral reinforcement layers are not restrained in the transverse direction, then they tend to enhance the lateral fracturing from shock wave transmission properties. This results in larger scabbed areas for laterally reinforced structures. Crater dimensions are, however, in general marginally reduced

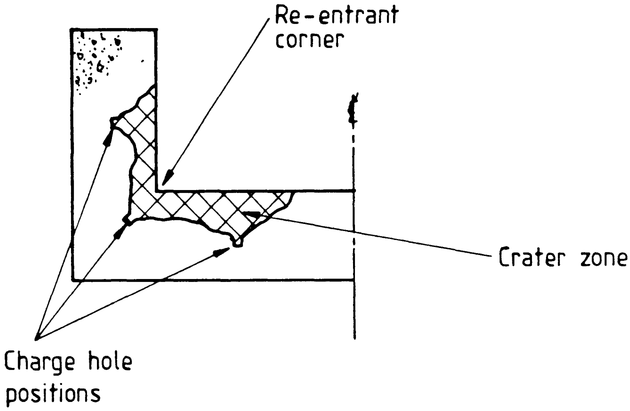

Figure 2. Section showing removal of concrete from a re-entrant corner

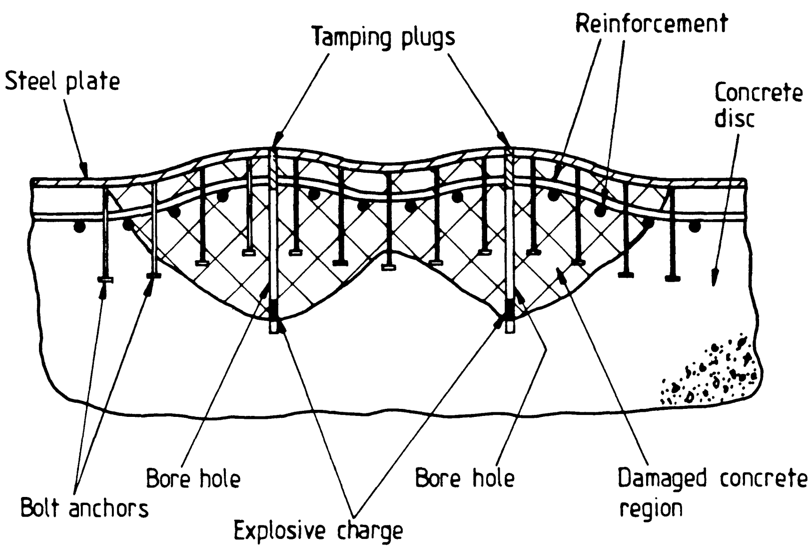

Figure 3. Sketch of simulation of a steel lined concrete face after firing of multiple charges

for reinforced concrete surfaces in comparison with unreinforced surfaces.

13. The investigations have also shown that the concrete cover on reinforced faces of concrete structures can be stripped off by firing charges beneath the reinforcement mesh. The amount of cover stripped off will depend on the depth of burial of the charge and the separation between charges. Once the reinforcement mesh has been exposed, conventional methods can be used to cut off the exposed reinforcement.

14. Typical pressure vessels will have steel liners on the inside surface of their containment structures to provide gas-tight membranes. In the decommissioning of such structures it will be necessary to remove both liners and the activated parts of the containment structures beyond them. To meet this requirement for liner removal, investigations have been carried out to determine the efficiency with which the liner and its anchoring system may be debonded from the concrete by firing explosive charges buried in the concrete below and adjacent to the liner.

15. A section through the biological shield was modelled by a circular concrete disc having a steel plate front face to represent the liner (see Fig. 3). This steel plate was anchored to the concrete disc, in a similar manner to a steel lined concrete structure, using small bolt anchors. Charges were fired in pre-prepared holes to investigate their effectiveness in debonding the liner and its anchoring system from the concrete.

16. Post test investigation and subsequent sectioning of the model indicated that the liner and its anchoring system had been debonded from the concrete in the local areas of the charges, with considerable break up of the concrete beneath. It was concluded that the feasibility of using the technique had been demonstrated. It was postulated that together with a pre-sectioning of the liner into small areas or post firing cutting of the liner, a complete removal of the liner and its anchoring system can be achieved.

17. The explosive technique requires the provision of holes drilled into active concrete. When drilling from the inside, the operation must of necessity be by remote control and the selected technique could generate considerable quantities of secondary waste. Drilling from the outside is complicated by the fact that some radioactive material could be wasted out necessitating special treatment of the effluent material. A number of the problems envisaged can, however, be overcome through the use of shaped charges to drill the charge holes from the inside of the nuclear facilities. This was investigated through a combination of field trials and computer integration, using an explosive jet/target interaction model, to optimise on the shaped charge design, size and stand-off distances to produce the required charge holes.

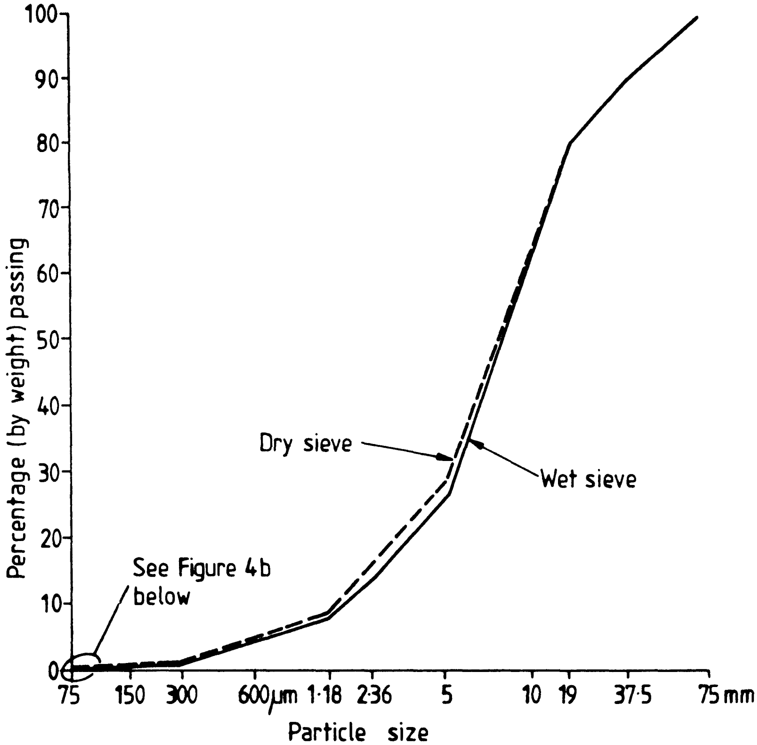

Figure 4a. Hand collected Debris Size Distribution

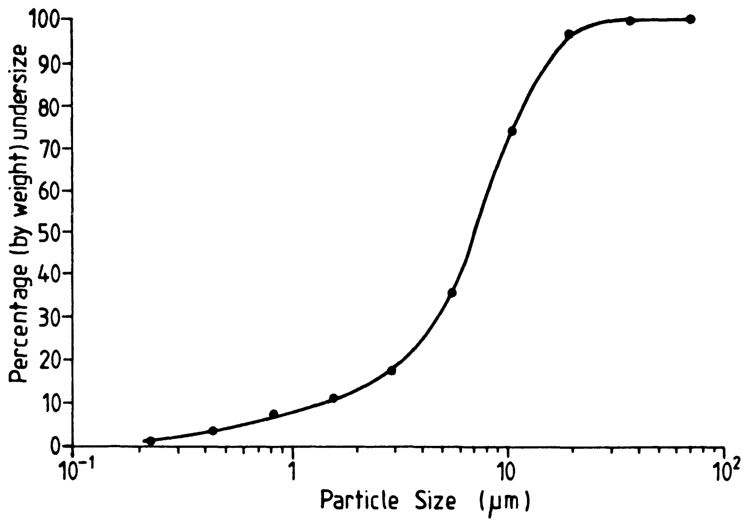

Figure 4b. Aerosol Size Characterisation

Figure 4. Particle Size Distribution

18. Particle size distribution analyses have been carried out on the debris produced by the explosive technique to assess the particle size distribution and to identify whether there are any aerosol size particles which might demand use of specialised filtration systems. In an initial investigation non-airborne debris was collected for particle size distribution analysis. No effort was made to collect any boulder size material cratered off. Particle sizes down to fine sand were analysed using both wet and dry sieving methods. Particles passing through the 75 microns sieve size were analysed using sedimentation process.

19. For the investigations into the size distribution of the airborne particles, a PVC enclosure was erected around one of the concrete blocks to help contain the generated dust from the blasting. Various particle sizing equipment, with their associated extraction systems, were used to cover the size range 0.01 - 15 microns. This involved the use of a combination of an electrical aerosol analyser, a light scattering particle size analyser and an Andersen Cascade impactor to study the entrained particulate. In addition, a sample of air was drawn through a high-efficiency glass fibre filter paper to give the total mass concentration of the entrained particulate.

20. The test results indicated that small quantities of submicron particles were present in the airspace after the explosives were fired (see Fig. 4). Further investigation into the source of the submicron particles suggested that these may have originated mostly from the explosives used and not from the concrete material. The results obtained from subsequent firings of the explosives alone in air appeared to substantiate this theory.

SHALLOW SURFACE MATERIAL REMOVAL

21. The use of scaled models in this development work has meant that very small explosive charges have been used to achieve the removal of comparatively very thin layers of material. Typically layer thicknesses from a few tens of millimetres up to 100 millimetres have been removed. It is therefore possible to use this technique for the decontamination of concrete surfaces of nuclear facilities or for the controlled removal of active concrete layers of facilities into defined activity zones.

22. The development work has shown that both the long borehole technique, where the boreholes lie in a plane parallel to the layers being stripped off, and the short borehole techniques, where the boreholes are drilled parallel to the layers, may be used (see Figs. 5 and 6). It has also been shown that with the correct positioning of charges it is possible to achieve the removal of near surface reinforcement layers at the same time.

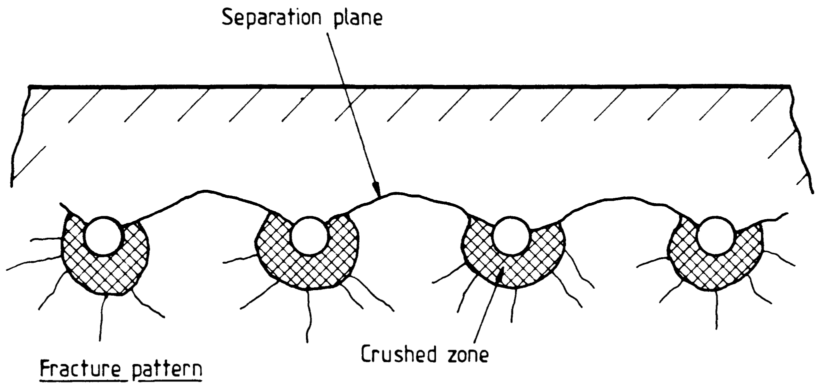

Figure 5. Long Borehole Technique

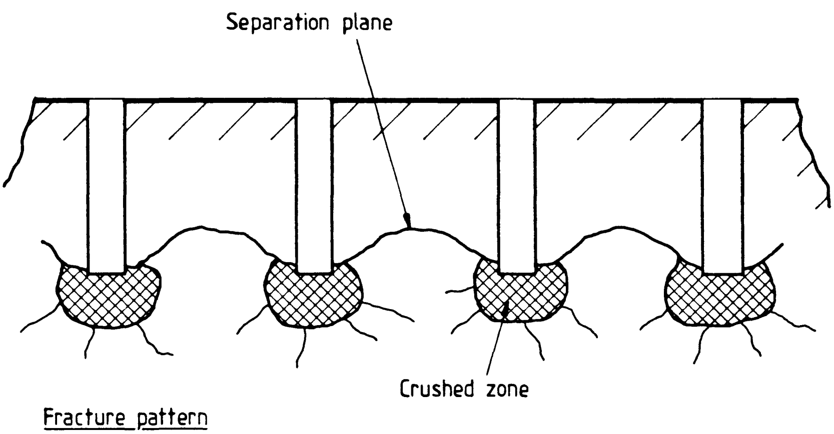

Figure 6. Short Borehole Technique

23. It was found that in direct comparisons with each other, the long borehole technique was shown to be marginally more effective than the short borehole technique. This results from the fact that it is comparatively easier to tamp or contain the explosive forces to achieve better cratering with the long borehole technique than with short boreholes. The main disadvantages with the long borehole technique comes from difficulties in the provision of boreholes on the inside of "closed" structures and in the production of long, straight boreholes (the ends of the drill bits start to deviate from straight lines when they get very long). However, on structures where there is an easy access to the surfaces to be removed for long boreholes to be drilled, then this technique can be used effectively. The lengths of the boreholes can be kept acceptably short by arranging to achieve the removal in bands.

CONCRETE DAMAGE ASSESSMENT

24. A concrete model has been loaded with an idealised transient and the response of the structure analysed using a finite element formulation ADINA (see ref. 7) and a finite difference formulation DYNA3D (see ref. 8). The ADINA formulation considered a two dimensional axisymmetric simulation whilst the DYNA3D considered the problem as a plane strain calculation (see ref. 4).

25. The material description was kept the same for the two analyses. This assumed that there are two material types:-

i) a high pressure region close to the charge represented by a small layer of mesh elements which surrounds the charge hole and represented by "compaction" material models (see ref. 9). A single deviatoric failure curve for compression and tension meridians. The deviatoric surface is represented by

$$\frac{J_2}{\sigma_2^2} = 0.491\,(P/\sigma_c)^2 + 0.745\,(P/\sigma_c) + 0.02$$

where P (mean pressure) = $\dfrac{\sigma_1 + \sigma_2 + \sigma_3}{3}$

and $J_2 = 1/6\,\{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2\}$

J_2 is the second invariant of stress

σ_1, σ_2 and σ_3 are three principal stresses

σ_c is the crushing limit of the material.

ii) a low pressure outer region having material modelled by concrete models having better definition of volume compaction data at lower pressures. The deviatoric behaviour in the concrete model is defined by the Ottosen failure criteria (see ref. 10) as

$$\frac{AJ_2}{\sigma_c^2} + \frac{\lambda\sqrt{J_2}}{\sigma_c} + B\frac{I_1}{\sigma_c} - 1 = 0$$

where $\lambda = \lambda(\cos 3\theta) > 0$

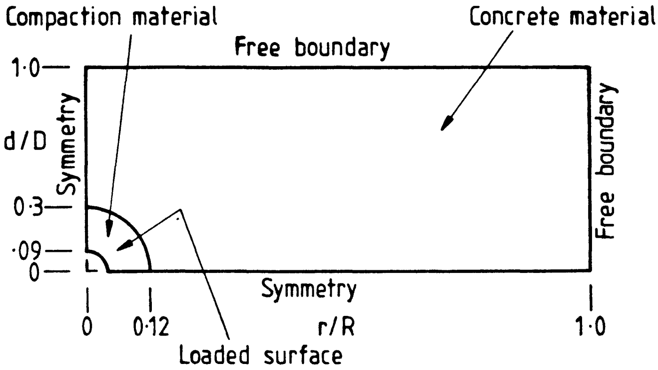

Figure 7a. Analytical Model for Test Specimen No. 1

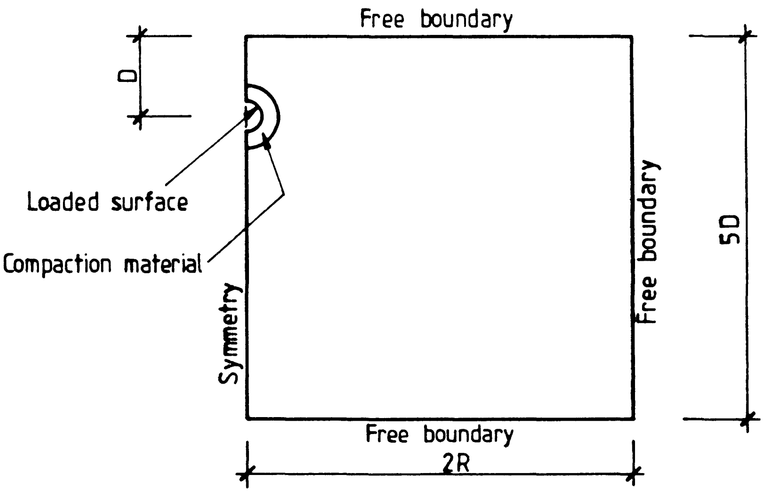

Figure 7b. Analytical Model for Test Specimen No. 2

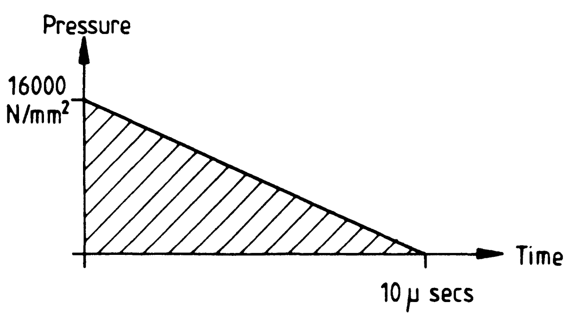

Figure 7c. Pressure Transient Applied to Loaded Surface

Figure 7. Theoretical Modelling

A and B are constants
and I, (first invariant of stresses) = $\sigma_1 + \sigma_2 + \sigma_3$

26. The results from both analytical techniques showed the formation of craters having characteristics similar to the crater produced on the physical model during field trials. The general conclusion drawn was that with better definition of concrete properties for the high rates of loading experienced under blast loads, the agreement between predicted and model test results could be improved.

REMOTE CONTROL SYSTEM

27. In order to reduce radiation exposure and occupational hazards during decommissioning tasks in nuclear facilities and to allow intervention at an early stage after shutdown, remotely controlled autonomous systems will be needed for the biological shield dismantling operation. The explosive cutting technique being investigated for decommissioning tasks will need such remotely controlled manipulation and/or robotic system to be developed. As a very first attempt in addressing this problem, basic desk top studies have been started to define the likely requirements such a system will need for the explosive cutting technique.

28. The general requirements for such a remote controlled manipulator are seen to include the following:

a) Sufficient reach and dexterity to position various tool packages associated with the cutting technique to all positions within the biological shield.

b) The capability of working in the operational environment envisaged for a decommissioning exercise.

c) Handling of all tools required for the drilling of charge holes, cutting of reinforcement and where necessary steel liners.

d) Control of the pre-drilling of charge holes, placing, tamping and initiation of cratering charges.

e) The safe handling of controlled quantities of explosive charges required for the dismantling techniques.

f) Identification of and retrieval and/or destruction of any misfired explosives.

g) The safe handling of the debris arising from the pre-drilling of charge holes and subsequent cratering processes.

h) Position and orientation determination, surface profile mapping, steel reinforcement detection and potential hazard identification

i) Sufficiently robust to withstand shock and vibration loadings imposed on it by the explosive cutting technique.

j) A versatile, reliable and compact control system.

k) Programmable features such that the manipulator could be used as a robot to perform repetitive tasks.

l) Visual coverage of the environment by means of TV cameras preferably with remote controlled zoom facilities.
m) Cameras and sensors for radiation levels etc., which can be detected by the manipulator and aimed at any specific areas in order to have more detailed information.

CONCLUSIONS

29. On the basis of the results obtained from this programme of work, the following conclusions have been drawn.
a) Small explosive charges may be fired in a controlled manner to crater off definable volumes of concrete. The volume of concrete cratered off is dependent on the depth at which the explosive charge is placed and the charge weight. As the depth of burial is increased, the volume of material ejected, as indicated by the crater diameter, increased almost linearly until the charge is insufficiently powerful to produce a crater and a camouflet (bulb shaped hole formed inside the material) is formed.
b) An equivalent thickness of concrete, for radiation protection, may be stripped off in layers sequentially using small explosive charges. This could be achieved without losing structural integrity and with full control with respect of safety.
c) Concrete cover on reinforced faces may be stripped off by firing charges beneath the reinforcement mesh. The presence of reinforcement (especially transversally restrained bars) produces slight reduction in crater dimensions.
d) Small explosive charges may be fired as point or line charges to remove shallow depth surface materials for decontamination purposes or to separate active material into different activity level debris.
e) Small explosive charges may be fired in a controlled manner to debond a steel liner and its anchoring system from the surrounding concrete and to break up a layer of the concrete adjacent to the liner. Charge positions and numbers can be designed to remove material from re-entrant corners and adjacent to penetrations and gas ducts.
f) Shaped charges with small explosive weights can be designed and used to produce boreholes suitable for placing cratering charges. The shaped charge designs can be modified to produce charge holes of specified dimensions.
g) Compared to other cutting techniques, only small volumes of dust are generated during cutting of concrete with explosives. The large majority of the aerosol content of this duct would appear to come from the explosive itself.
h) The feasibility of using a finite element and finite difference formulation to analyse the response of concrete to explosive loads have been demonstrated. It was concluded that the comparison with field trails can be improved with better definitions of concrete properties under high rates of loading.

It is considered that sufficient knowledge has been obtained of the potential for the explosive cutting technique to justify full-scale field trials being undertaken.

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The author also acknowledges the help given to him in carrying out the work by his colleagues. Special acknowledgement to Hunting Engineering Limited for their contribution in running the site trials and designing the shaped charges, the Computer Applications Group of Taylor Woodrow for the finite element analysis, UKAEA Winfrith for the dynamic relaxation analysis and UKAEA Harwell for the aerosol characterisation analysis.

25. Preliminary work for stage 2 decommissioning of B16 pile chimney

E. M. WRIGHT, British Nuclear Fuels plc Decommissioning Unit, and R. F. MATHEWS, MA(Cantab), MICE, W. S. Atkins – Northern

SYNOPSIS Planning of the second stage of decommissioning of the two pile chimneys at Sellafield started while work was underway on the first stage, which involved removal of the sections above the filters. The second stage requires the removal of all radio-active parts and the dismantling of the filter and diffuser sections, and has to be completed by 1997. The planning involved studying the many possible options and their effects on both radiological and industrial safety.

This decommissioning project employs a high proportion of civil engineering and construction techniques, which are then developed to eliminate the hazards from radioactive dusts, and to minimise the effect of radiation on operatives working on the project. Much of this equipment is modified forms of standard construction equipment and includes cutting equipment and remotely operated vehicles.

The initial phases of the work involve:-

1. Provision of a waste packaging and access building, which is a simple portal framed, sheet clad structure, designed to be moved from B16 to B6 during the life of the project.
2. Provision of temporary ventilation systems to control the dust generated by the work. These are containerised so that they can be easily moved from B16 to B6.
3. Cutting of 3 m square access doorway through the 1.5 m thick reinforced concrete wall of the chimney, using diamond drilling and bursting techniques.
4. Provision of Remotely Operated Vehicle (ROV) to act as a tool carrier for lining stripping work. This machine is a radio controlled JCB 812s provided with its own lighting and CCTV systems.

5 Removal of the thermal lining from the floor and lower walls of the chimney, by stripping away the aluminium lining and glass fibre insulation using the ROV and cutting up the supporting steelwork using shears mounted on the ROV.

6 Installation of precast concrete walls which separate the pile reactor core from the chimney flue. These walls are specifically designed for rapid installation to reduce the dose up-take of the operatives.

INTRODUCTION

1. The two Pile Chimneys were built at Sellafield between 1947 and 1950 to discharge cooling air from the two pile reactors, and they operated until 1957, when both reactors were shut down following the fire in Pile 1. The two chimneys, given reference numbers B6 and B16 were identical and each served its own reactor. The addition of filters at the top of the chimneys at a late stage of the construction necessitated a complex design. Each chimney was 125 m high and consisted of seven sections (Ref Fig 1), foundations, main shaft, diffuser section, filter section, concentrator section, upper shaft and access shaft. The foundations, main shaft and diffuser section are all built of reinforced concrete, while the remainder are steel frame structures clad in brickwork and asbestos cement sheets. A more detailed description of the structures, and their purpose is set out in reference 1.

2 Planning work for decommissioning the two chimneys started in 1987 and the resulting overall project requirements were that the sections above the filters of both chimneys should be removed by the end of 1992, and that the filter and diffuser sections should be removed by the end of 1997. At the start of the project it was determined that the work on each chimney should be split into two stages. Stage 1 was to be the relatively straightforward work of removing the sections above the filters, and the radiologically and physically complex work of removing the filter and diffuser section was to be stage 2. It was also decided that work on B16 should precede that on B6 which had been radioactively contaminated during the Windscale fire in 1957. A full description of the initial studies and stage 1 work is also given in reference 1.

3 Stage 1 work was completed on B16 at the end of 1989. In order to allow work on stage 2 to start in early 1990 planning work for stage 2 started in early 1989.

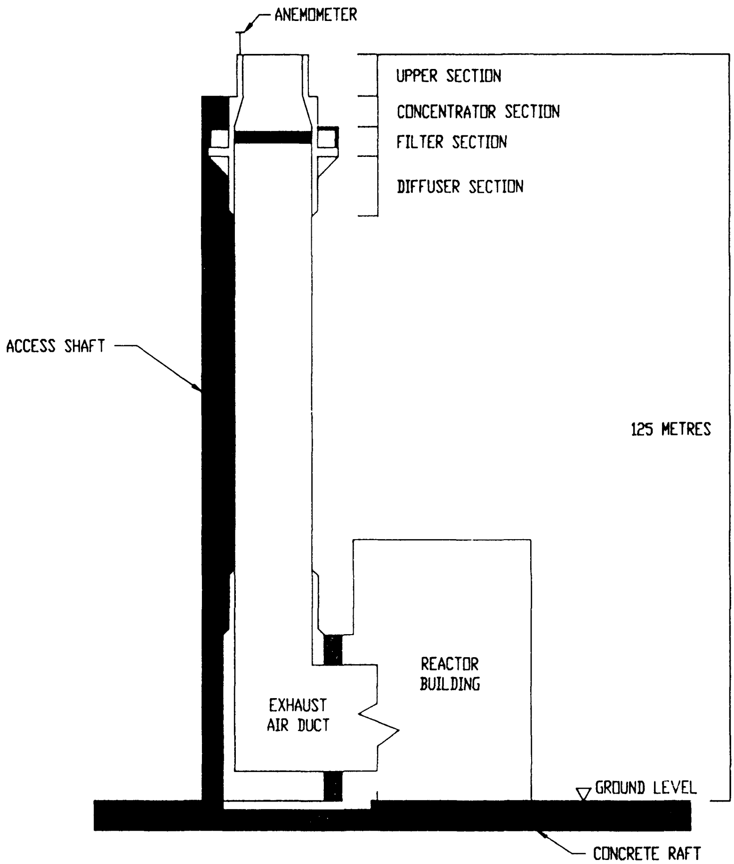


FIG 1 GENERAL ELEVATION

PLANNING AND STUDIES

1 The planning for the work started by looking at the radiological problems involved in the fire contaminated B6. Material samples were taken from the chimneys and insitu radiation monitoring proved that radiation levels inside the chimneys were 5 mSv/hr (MilliSieverts per hour) for B6 and 0.2 mSv/hr for B16. Working times inside the bases of the chimneys were limited to 5 minutes per day in B6 and 2 hours per day in B16. Therefore two primary decisions were made. The first was that the work should be divided into a period of removal of radioactive material using remotely operated equipment under contained conditions. This would then be followed by physical demolition using traditional manual methods. The second decision was that the easier working conditions of B16 should be used to trial and develop any specialist equipment required for B6.

2 The planning studies culminated in the issue of a report detailing the options available for the project and recommending a best method. The best method had to take into account the divergent requirements of radiological safety, industrial safety, technical soundness and cost. The proposals were then subjected to a HAZOP 1 study (Hazard and Operability study). This study, attended by the project manager, engineers, and industrial and radiological safety advisors, allowed the proposed work methods to be considered in a structured manner to ensure that they were safe and practical.

3 Once the method of working had been agreed for each section of the work, outline design was carried out and the project budgets prepared. Because no data existed for work of this nature cost predictions were based on estimated manhours for each task with materials costs where applicable. The estimate of manpower requirements was then used to extrapolate the anticipated radioactive dose-uptake for the workforce on each task. The final cost of £78 million and the total dose uptake of eight man sievert indicate the difficulty of the project. A general philosophy of the work has been to treat the project as a construction project in reverse, wherever possible using equipment and techniques which are well proven in the civil engineering construction industry, but modified to suit the specialist requirements of a nuclear decommissioning project. In some cases this has not been possible and special purpose equipment to handle radioactive waste material is being designed and developed by the project team.

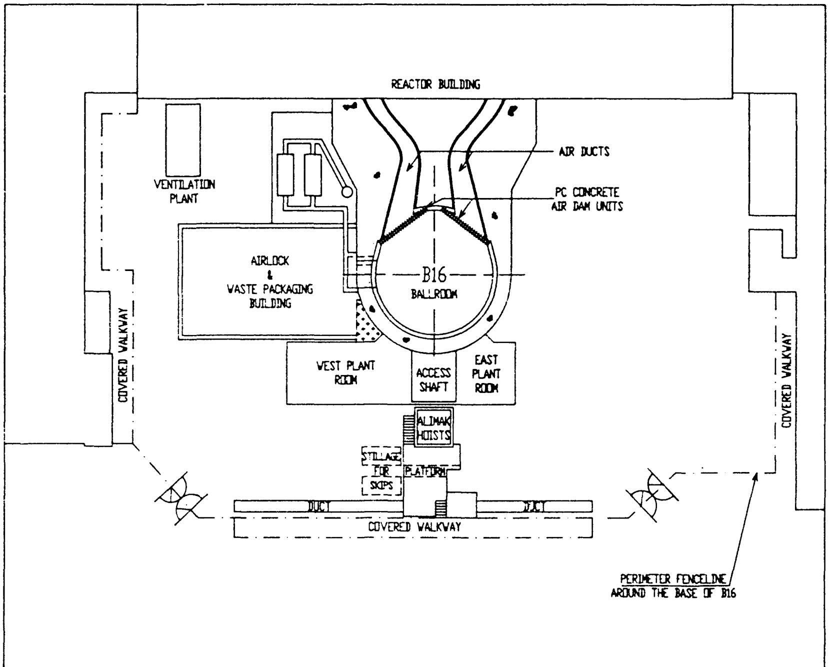


FIG 2 AREA PLAN

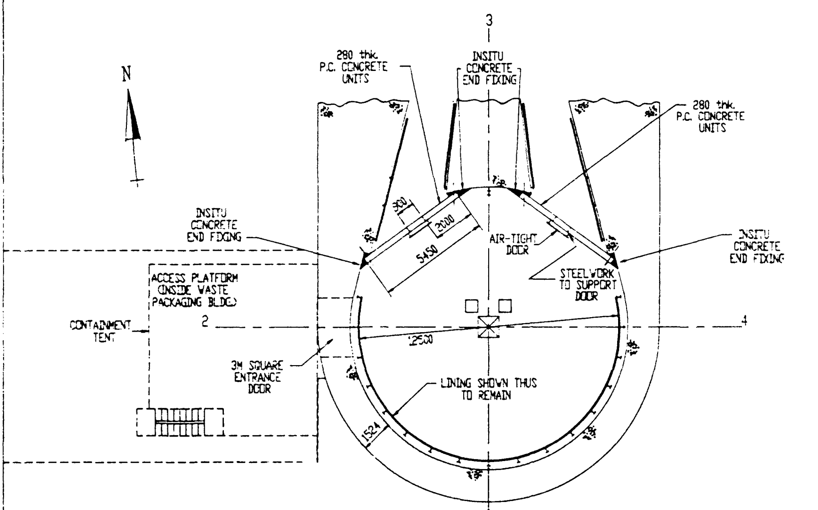


FIG 3 SECTIONAL PLAN THROUGH CHIMNEY AT 11.0 M. LEVEL

4 Stage 2 of the project was divided into twelve sections:-

(a) Provision of waste packaging building and base vent plant
(b) Breaking into chimney base
(c) Installation of air dams
(d) Filter washing trough removal
(e) Provision of headgear, top vent plant and cover
(f) Filter removal
(g) Stripping of the flue liner
(h) Decontamination of the inside of the chimney
(i) Provision of scaffolding and working platforms
(j) Dismantling of the filter section
(k) Dismantling of diffuser section
(l) Removal of access shaft.

Each of these sections could be handled as an independent sub-project and work could stop safely at the end of any section.

5 Approval to proceed with the preliminary sections of the project, (a to f), was given in April 1990. A target date of December 1990 was set for the installation of the air dams to separate the reactor from its chimney. Therefore, design work for sections a, b and c which would allow the installation of the air dams was rapidly put in hand and a design contract was let by competitive tender to WS Atkins - Northern. This work is described in the following sections.

<u>Provision of waste packaging and access building</u>

1 The inside of the base of the chimney was contaminated during operation of the reactor and the work of removing the lining and installing the airdams was expected to generate considerable quantities of airborne contamination. The optimum method of preventing the escape of this dust was to build an airlock around the access opening which would be built into the base of the chimney (Ref fig 2). This building was designed to allow the waste extracted from the base of the chimney to be sorted and packaged into waste skips in a controlled environment.
After considering the design life and the need for an identical building to perform the same function at the B6 chimney, it was decided that a transportable building should be designed and built.

2 A Contract for waste packaging and access building was let to PML (Pressed Metals Ltd.) of Stanley. The building design employed a series of steel portal frames, clad in fabric reinforced polythene sheet. To avoid excessive excavation, the foundation consists of a simple reinforced concrete raft with an edge beam. Special features of the building include a removable section of the roof to allow large or bulky equipment to be craned into the building, and access doors which can be positioned in different bays to account for the differing site requirements of B6 & B16.

3 To provide access into the base of the chimney which is about 5 m above ground level an independent free standing platform will be provided inside the building. To allow easy adaptation for later stages and transportation to B6, the platform has been designed using Kwikform proprietary falsework, and the airlock was built from reinforced PVC sheet suspended from the building framework.

Base Ventilation Systems

1 In order to control the airflows in the base of the chimney and to guarantee a airflow through the Pile reactor during the installation of the airdams, a ventilation system was required. This system with a capacity of 5.6m3/sec, was designed with containerised HEPA filter units and skid mounted fans, so that the main components could be transferred to B6 following the work on B16. The system was built and installed by CG Firths. Commissioning is scheduled for completion on 12 November 1990.

2 The system extracts air from two 600 mm diameter openings which were diamond cored through the concrete wall of the chimney over the door opening. At the outset there is no ductwork within the chimney, but as work progresses internal ductwork can be provided in order to further control air flow within the base of the chimney. The ductwork is spiral wound and was prefabricated off site and in sections light enough to facilitate installation by hand from the access scaffold. After filtration the air is monitored for radioactivity prior to discharge through a 30 m high exhaust duct which was bolted to the side of the chimney by a team of steeplejacks.

3 An additional problem was that the ventilation system from the adjacent B29 Pond originally discharged into the base of the chimney. To allow concurrent decommissioning of the pond, new HEPA filters were inserted into the B29 ductwork adjacent to the pond and the airflow diverted into an old duct which ran up the access shaft of B16. This duct originally served to provide ventilation air at positive pressure to the filter gallery.

It was surveyed and found to be in such good condition that it could be reused. A short cross connection at the base and a new discharge bend at the top were all made up from redundant sections of the duct. The complete diversion was made up without the costs incurred from new ductwork or disposing of the original.

Access doorway

1 To allow access for men and materials a 3 m square door had to be formed through the 1.5 m thick heavily reinforced concrete wall at the base of the chimney. Consideration was given to forming the opening using a diamond wire saw. This proposal was abandoned because of the difficulty of moving the 27te block of concrete and complications of providing access to the rear of the block to thread the wires A detailed method for removing the wall was chosen and approved prior to tender. This procedure frequently occurs on the site as safety cases have to be agreed prior to the work commencing. Changes cannot readily take place and the contractor has to follow the tendered method statement. The contract was won following a very tight tender by Thermic UK. The opening was formed by first stitch drilling all around the opening to a depth of 1.2 m. A pattern of 200 mm core holes was formed in the block and broken up using hydraulic bursters to a depth of 1.2 m. All this work was carried out without the impediment of protective clothing or the need for containment or ventilation as the remaining 300 mm of concrete formed a radiation and contamination barrier. Once the front part of the opening was formed an airtight steel door was fitted into a rebate formed around the opening. This allowed the last 300 mm of wall to be removed and the inner door of the airlock to be installed under contained conditions. The final wall section was removed using concrete crushers after an initial opening had been formed.

2 The main problem associated with forming the door opening was containing the cutting water, as this was potentially contaminated. Guards around the cutting heads with vacuum extraction were used and a series of gutters were created to provide a final back-up. However some seepage did occur. Because of the relatively clean conditions on B16 the seepage caused no problems but this aspect of the work will have to be improved before the door is formed on B6.

Provision of Remotely Operated Vehicle (ROV)

1 The usual equipment in the civil engineering industry for reducing labour content is the hydraulic excavator. A market search was carried out for suitable remotely operated hydraulic machines. Two were found. The 3te Brokk from Sweden and a converted Hymac used by the RAF for digging up unexploded bombs. After a competitive tender a contract was placed with Ergotel to provide a radio controlled JCB 812s.

This machine had two advantages over the Brokk machine. The first was that it is radio controlled whilst the Brokk uses an umbilical. This meant that there was no umbilical to cut by being tracked over, which would have left the machine irretrievable. The second was that its larger weight (13.2te) meant that it could mount heavier specialist cutting tools at the boom end.

2 The control system was designed and the machine converted tested and delivered in a matter of 17 weeks. The control system is mounted in a road trailer, but the normal manual controls are retained should it be required to operate the machine from the cab. Forward viewing is provided by two cameras mounted on the machine adjacent to the cab, one providing a stereoscopic picture to allow the operator to judge the distance to the workface. A rearward view CCTV is provided as is a sound channel to allow the operator to use as many of his senses as possible.
Thought was given to tilting the operators chair to allow even more feel to the machine but this was abandoned as unnecessarily expensive. A series of aerials is provided to allow the signal to be transmitted to the machine when it is operating inside the chimney base.

3 The machine retains its diesel power plant. Consideration was given to an electrically powered machine, but this would have required the use of an umbilical. Instead the machine will be withdrawn from the base of the chimney and the air tested for carbon monoxide and oxygen deficiency prior to man entry. Because of the restricted operating space within the base of the chimney the separation of men and machine is a basic safety requirement to prevent anyone being crushed by the machine.

4 The main aim of the machine is to deploy specialist purpose made tooling to cut up the lining and to scabble 10 mm of the concrete surface to remove the radioactive contamination. Hydraulic demolition hammers were banned from use on the chimneys in 1989 when the vibration from one caused loose concrete to fall from the filter gallery soffits. Shears, crushers and saws are therefore preferred equipment.

Currently the main tool is a hydraulic shear, supplied by Allied Construction, this tool can cut through steel sections up to 200 mm by 150 mm which were the largest used in the construction of the chimney, and can also act as a grab and grappling tool.

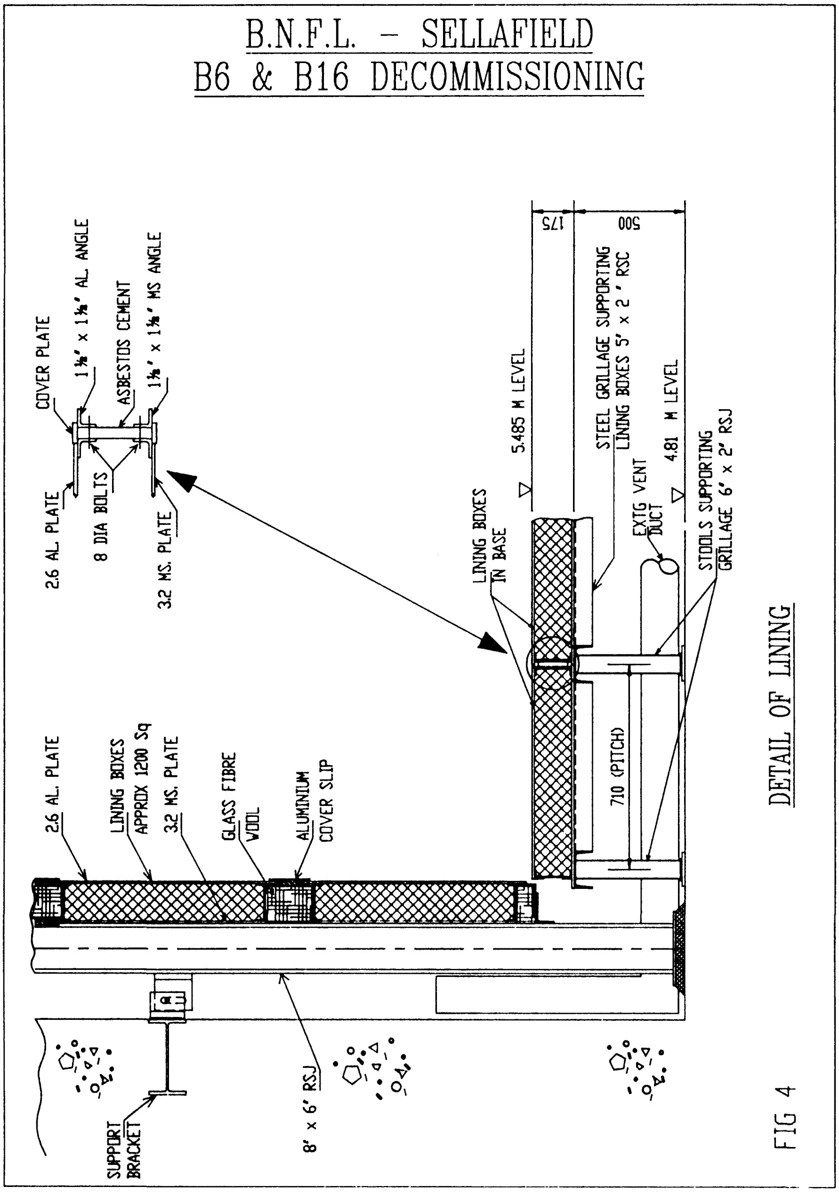
<u>Removal of the Thermal Lining.</u>
As this paper is being prepared this work is just starting, but will be complete prior to the conference on the 20 March.

1 The work of removing the lining and installing of the airdams has been let to PC Richardsons, following preparation by the design team, of a detailed design and a method statement for carrying out the work. The work method is based on the results of trial removal of boxes from within the chimney base. (Ref Fig 4). It was found that the floor was in a very poor condition, being very heavily corroded with the insulation saturated by rainwater. The materials were contaminated with radioactive caesium causing handling problems. On the other hand the wall lining and insulation was in almost perfect condition, with no visible corrosion and the red lead paint still intact. The current radiation level within the chimney base are 0.2 to 0.3 mSv/hr (Milli Sieverts per hour), but sampling indicates that most of the contamination is adhered to the lining. Because of the relatively high radiation and contamination levels, work methods had to concentrate on reducing manhours worked inside the chimney rather than purely on minimising final cost. Therefore the plan is to use the ROV to remove the floor lining and as much of the wall lining as can be easily reached. It is estimated that this will halve the radiation inside the base of the chimney. Men will carry out the more delicate and tricky operations since the cost of designing and building a completely remotely operated machine would be prohibitive.

2 The section of the lining behind the door opening will be removed manually by grinding off the bolt heads supporting the lining boxes to the steel work and removing the boxes whole for disposal. This will allow the ROV to enter the base of the chimney and rip out all the floor lining. It will then proceed to remove the wall lining for a height of 5 m.

3 Waste will be disposed of to the Low Level Waste Repository at Drigg and requires careful handling and control. All waste material will be placed into a plastic lined skip in the base of the chimney. This will be then extracted to the Waste Packaging Building (WPB) where the plastic lining will be sealed and the contents monitored for activity, weighed, and recorded.

The waste inside its plastic lining can then be carefully slid into a transport skip for shipment to Drigg. The sealed skips, the tent inside the building and the ventilation system provide the control to prevent any radioactive material escaping to atmosphere (Ref fig 3). A regime of sampling and laboratory analysis before work starts allows the radiological content of the waste to be determined. This guarantees that everything which will be sent to Drigg complies with the conditions of acceptance.


B.N.F.L. - SELLAFIELD
B6 & B16 DECOMMISSIONING
COVER PLATE
1 ½" x 1 ½" AL ANGLE
ASBESTOS CEMENT
1 ½" x 1 ½" MS ANGLE
2.6 AL PLATE
8 DIA BOLTS
3.2 MS. PLATE
175
500
5.485 M LEVEL
STEEL GRILLAGE SUPPORTING LINING BOXES 5' x 2' RSC
4.81 M LEVEL
LINING BOXES IN BASE
EXTG VENT DUCT
STOOLS SUPPORTING GRILLAGE 6' x 2' RSJ
2.6 AL PLATE
LINING BOXES APPROX 1200 Sq
3.2 MS. PLATE
GLASS FIBRE WOOL
ALUMINIUM COVER SLIP
710 (PITCH)
DETAIL OF LINING
SUPPORT BRACKET
8' x 6' RSJ
FIG 4

5 Once the floor lining has been removed the concrete surface will be cleaned. This will halve the radiation inside the chimney base and allow the remainder of the work to be carried out manually.

6 As the airducts are 16 m high the lining has to be removed from around the ducts for a further metre. The lining will be stripped progressively as scaffold towers are erected on each side of the opening. At the top of each airduct cat heads will be bolted to the concrete wall of the chimney. From these cradles will be suspended to allow rapid man access to the final wall position, which will be at the entrance of the air ducts (Ref fig 3). In addition pulleys will be provided on the cat heads to carry winch wires for lifting the parts of the airdams into position. The winches will be bolted to the floor of the chimney opposite the walls.

Installation of Air Dams

1 In the B16 chimney there are two symmetrical air ducts which connect the Pile reactor to the chimney. (Ref fig 2). Each air duct is 16 metres high and 7 m across the exit into the chimney. A number of schemes for sealing them were investigated. It was decided to use a system of stackable precast concrete beams spanning across the open mouth of the ducts. The most important reasons for choosing this system were:-

a) Radiation shielding
b) Speed of installation
c) Ability to withstand earthquake loading
d) Ease of providing in-situ infill sections to take up size variations in existing concrete structure
e) Ability to remove the lower parts of the walls without affecting the top part. This would allow access into the airducts at a future date should it be necessary for decommissioning the Pile reactor.

2 The final design chosen uses 7 m long precast concrete beams weighing about 1.25 tonnes (Ref fig 5). The blocks have a chevron shaped cross section to aid stacking and alignment, and also to prevent radiation shine through the gaps. Once the beams are stacked in position they will shield the workforce from radiation emanating from the airducts.

3 The precast beams will be delivered to the waste packaging building and hoisted onto the the platform in the airlock using the internal overhead runway. Inside the airlock they will be lowered onto a pair of wheeled bogies and pulled into the base of the chimney. Once inside the chimney they will be manoeuvred to the base of the wall and

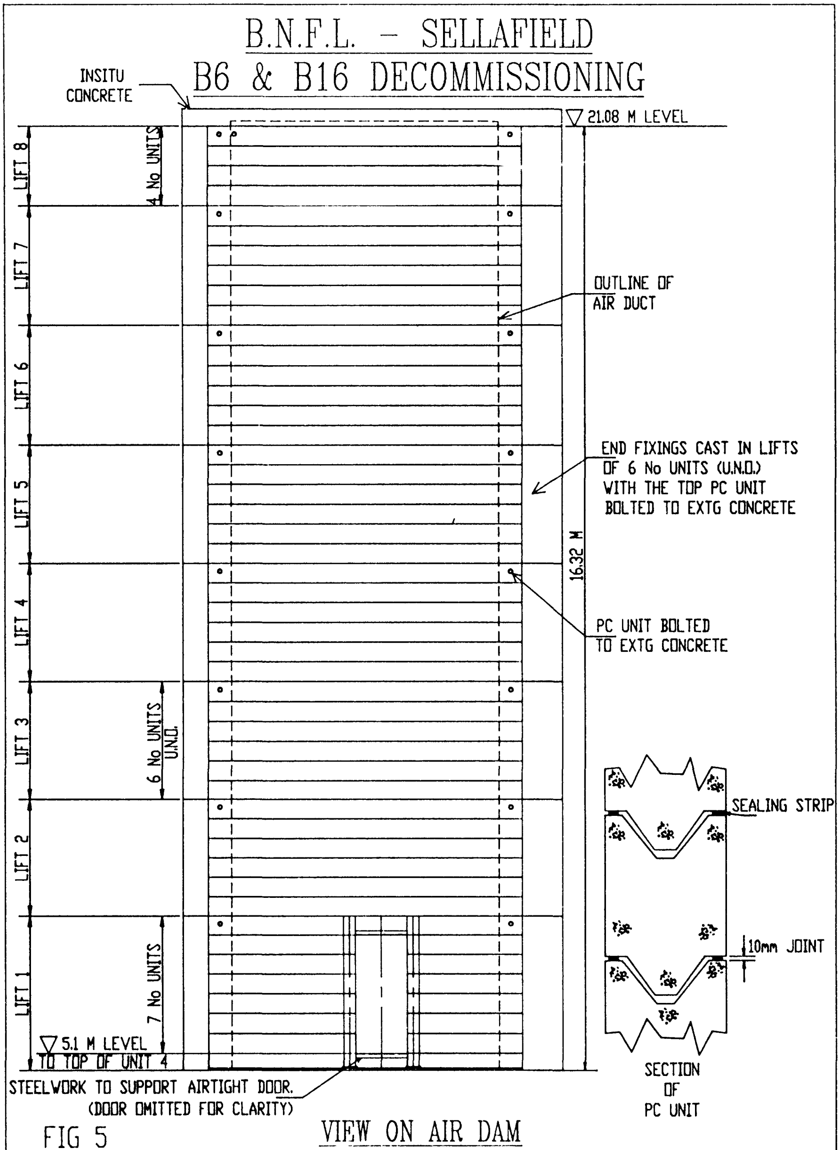


FIG 5

VIEW ON AIR DAM
SHOWING PC UNITS

the winch wires hooked on. They will then be hoisted up the face of the wall using the winches and lowered onto the wall top. Steeplejacks working from cradles will assist the chevrons in locating the beam into its final position. They will then be temporarily bolted back to the concrete of the chimney to stabilise the stack.

4 On completion of the wall of precast beams reinforcing bars will be drilled into the existing walls and an insitu joint made at both ends of the beam thereby ensuring structural integrity. A neoprene strip between the units will provide an airtight seal.

5 Each wall will be built in an identical manner and once the walls are complete then the winches and access cradles will be removed to allow the next stage of decommissioning to be carried out.

6 Airtight bulkhead doors in the base of each wall will allow access into the airducts for future inspections. They also allow the ventilation plant at the base of the chimney to draw air through the pile until the ventilation plant, which is being installed for decommissioning the pile reactor, is commissioned.

Further Work

1 Once the air dam installation has been completed the B16 chimney will be isolated from the Pile 2 reactor, which will allow the decommissioning of the two parts to continue independently.

2 An internal platform suspended from winches at the chimney top will allow the lining to be stripped remotely and the internal surfaces to be decontaminated. This will allow manual demolition of the filter gallery and diffuser section to follow using fairly traditional techniques.

CONCLUSIONS

1 The preliminary work on decommissioning the B16 Pile Chimney shows how well tried civil engineering construction and management techniques can be adapted to solve new decommissioning problems in the nuclear industry.

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Discussion

N. THORNTON, W. S. Atkins Northern, Whitehaven
Has Mr Fleischer considered methods of remotely removing and packaging the material which will be removed by explosive scabbing from the inner surface of the structure?

J. R. MAGUIRE, Lloyd's Register, Croydon
Global dynamic tests can often yield useful structural behaviour information. They can improve understanding and can be issued to assess ongoing structural integrity. The technique is in worldwide use in the aerospace and automatic industries, particularly in the US and Japan. Can Mr Dawson comment on why this technique is not generally used in our nuclear industry?

B. SKIPP, Soil Mechanics Associates, Wokingham
I endorse the comments of Dr McGuire. If seismic triggering instrumentation has a sufficiently wide band and if a 'free field' seismograph is provided, ambient and microtremor excitation can yield information to validate numerical modelling, and may even yield 'status' clues in what is designed as a linear system. In Japan this approach is proving worthwhile, but we need not be passive, and here I am on a hobby horse. Something approaching a plane wave travelling across a site and foundation can be generated by vibroseis or even explosive sources. The Czechs have done it.

W. THORP, Multi Design Consultants Ltd, Stockport
Since our real concern is the long-term overall force contribution from the tendons, is there not a gain to be obtained from comparing the overall extensions measured at destress to the extension at restress to reinforce the information obtained from the measurement of lift-off loads?

S. JONES, W. S. Atkins Consultants Ltd, Epsom
How does Mr Dawson think the French pressure vessel decommissioning will vary from British PCVs, given their tendency for grouting in tendons?

P. DAWSON, Paper 23
The Authors are unaware of any attempt to use global dynamic tests to provide data for pre-stressed concrete pressure vessels, though they may be used elsewhere in the nuclear industry, for the examination of mechanical components.

Surveillance of PCPVs is sometimes carried out 'on-load', and in such circumstances the output from global dynamic tests is likely to be masked by vibrations induced by the operating reactor plant. Even when the surveillance is 'off-load', a sufficient amount of reactor plant continues to operate and this would at least complicate if not invalidate the interpretation of dynamic test data.

Pre-stressed concrete pressure vessels are valuable items of plant, housing even more valuable nuclear reactors, which the public hold in some awe. I see no prospect of a station manager agreeing to the use of explosives specifically intended to disturb his plant for experimental purposes, and even if one did, the NII would surely object.

For so long as PCPVs house operational nuclear reactors they will not be used as test beds. The risks, however slight, of the test activities adversely affecting them or their contents are unlikely to be justified by the uncertain benefits of novel monitoring techniques.

In reply to Mr Thorpe, in the majority of pre-stressing tendon surveillance operations the current force at an anchorage is obtained by measuring the force needed to just lift the anchorage off its seating. Tendons are only destressed when it is required to remove wire or strand for corrosion examination. On a few such occasions attempts have been made to measure the recovery of extension, but it has not proved possible to use this measurement to obtain an indication of the force in the tendon before destressing because of the complicating effects of reverse friction during destressing and the lock-off losses which occurred when the tendon was originally stressed.

Concerning Mr Jones' point, during the time between taking the reactor out of service and demolishing the pressure vessel, the French PCPVs will remain fully stressed but at atmospheric internal pressure. A modest level of surveillance will be needed to ensure that they remain in a structurally satisfactory condition during this period. The UK PCPVs could remain fully pre-stressed, but the option will be available to remove some or all of the tendons. The benefits of so doing have yet to be investigated in detail. If the tendons are not destressed and removed, some level of monitoring will be required to ensure that the stressed tendons do not present an unacceptable hazard. If the tendons are to be removed it will be necessary to show that the PCPV, when not pre-stressed, will be in a structurally satisfactory condition to contain the radioactivity present within it. Eventual demolition of both French and UK PCPVs will be a massive undertaking, the prior removal of the tendons from the UK PCPVs making their demolition slightly easier in this respect.

C. C. FLEISCHER, Paper 24
In reply to Mr Thornton, there are already various concepts in existence of remotely controlled manipulators for collecting and packaging materials. AT this stage of our development programme, it is not envisaged that the debris arising from the use of explosives will give rise to insurmountable problems. Basic desk-top studies have already been carried out to define the requirements of a remotely controlled manipulator to remove and package explosive scabbing debris.

The feasibility of using global dynamic testing to obtain useful information on the state of a structure has, however, been recognized in some non-nuclear areas. The ideas here involve obtaining initial signatures of a structure's response to dynamic loading before it is subjected to normal working loads. In subsequent years, the dynamic loading can be repeated whenever required to check whether in-service loads have changed the structure's response signatures. These changes can be traced back to damage locations within the structure. There is, therefore, the feasibility of using this approach to augment surveillance activities of nuclear structures. However, there is a need to demonstrate the applicability of this approach for use on such massive structures as nuclear PCPVs.

Prestressing systems for nuclear power stations

P. DAWSON, EurIng, MA, MICE, Taywood Engineering Ltd

The introduction of prestressed concrete technology to the nuclear industry stimulated three separate and significant developments of prestressing systems. For prestressed concrete pressure vessels there was a need for larger systems than were currently available; the importance of ensuring that these structures would behave satisfactorily and predictably throughout their working life led to major improvements in the control and knowledge of high tensile steel relaxation; and the demands for assured performance promoted substantial advances in anchorage efficiency and quality control. Developments initiated for PCPVs have subsequently been incorporated in the prestressed concrete containment structures which enclose the reactor and steam generating plant for many light water reactors.

This poster display illustrates the history of the progressive advances in prestressing technology in response to the demands of the nuclear industry, including the introduction of low-relaxation prestressing wire and strand, 1000T capacity linear tendons, wire-winding and the corrosion protection of unbonded tendon systems.

OUTPUT OF PCPV REACTORS

As at December 1991 or shut-down, if earlier

($MW\text{-}hrs \times 10^6$)

Marcoule	G2	-	
	G3	-	
Chinon	A3	30	
Oldbury	1	35)	70
	2	35)	
Wylfa	1	60)	114
	2	54)	
St Laurent	1	47)	92
	2	45)	
Vandellos		56	
Bugey		51	
Fort St Vrain		5	
Hinkley B	1	44)	90
	2	46)	
Hunterston B	1	48)	91
	2	43)	
Dungeness B	1	9)	14
	2	5)	
Hartlepool	1	9)	19
	2	10)	
Heysham A	1	13)	26
	2	13)	
Schmehausen		3	
Heysham B	1	5)	10
	2	5)	
Torness	1	7)	12
	2	5)	

Blast and explosion resistant design of nuclear facilities

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INTRODUCTION.

1. Explosions may occur at a facility having an inventory of highly active radioactive materials. The explosion will cause blast pressure waves on the structure and may also cause penetration and perforation by resulting fragmented missiles.

2. The containment structures enveloping the nuclear materials may be subjected to an internal explosion due to an accident or, may be exposed to air blast pressures and ground shocks from an external aggression. These structures should be strong enough to withstand all effects of a postulated explosion scenario. They should be designed to prevent uncontrolled release of radioactivity after an incident.

THE ANALYSIS AND DESIGN METHOD.

3. The state-of-the-art method of design of blast-resistant nuclear structures would normally follow the steps described below.

Evaluation of Pressure-Time History

4. Blast pressures and impulses could be due to bursting gas spheres, TNT or equivalent explosions, Vapour cloud explosion, arcs in electric circuit breakers, ground shocks due to explosions in the ground and air-blast shocks. For each of these explosion scenarios the blast wave loading on the structure can be computed with vented or unvented conditions for internal explosions. A typical pressure-time diagram is shown in Fig.1.

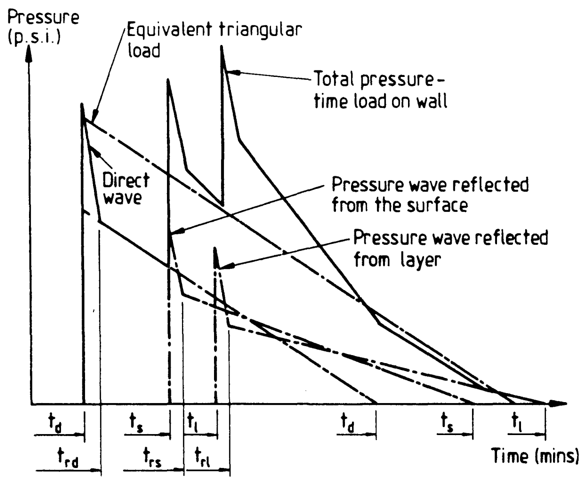

Fig.1. Typical Pressure-Time diagram on a rigid structure.

Global Dynamic Analysis including Soil-Structure interaction

5. A Finite Element idealisation of the structure is made and a dynamic analysis is carried out with soil boundaries modelled by compliant springs. The dynamic loading on the structure is the pressure-time history from the postulated blast. Typically the global internal forces in the structural resisting system will be found from this analysis. The displacement and acceleration time histories at selected points in the structure will also be obtained. Fig 2. shows a typical acceleration time history. In-structure shock spectra at equipment locations will be computed.

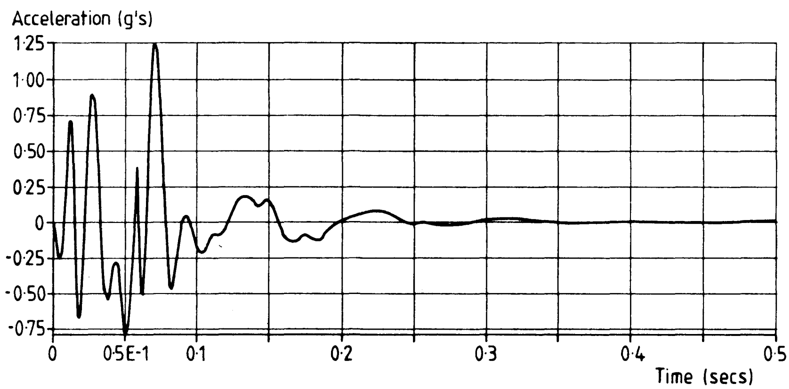

Fig 2. Typical Acceleration Time History.

Check Safe Shut-down Equipment

6. All safety related equipment should have available shock-resistance spectra determined by tests or otherwise. Typical shock resistance spectra are shown in Fig 3. These will be compared with the in-structure shock spectra at the point of attachment.

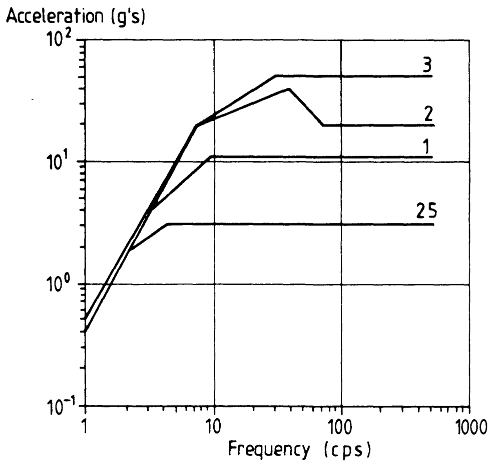

Fig 3. Typical shock-resistance spectra.

Equipment Isolation

7. Equipment isolation by proprietary isolation systems will be required if the safe shut-down equipment cannot withstand the shock loading. A local dynamic analysis will be carried out for each equipment requiring isolation using the global acceleration or displacement time history as the dynamic loading. Typical equipment isolation system is shown in Fig 4.

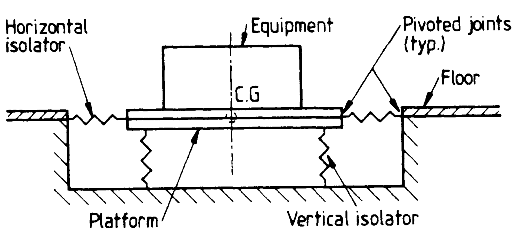

Fig 4. Typical equipment isolation system.

Structural Design

8. Main structural resisting system in a Nuclear facility is generally by reinforced concrete shear walls. These shear walls have low ductility unless the total shear is carried by shear reinforcement. The shear walls are designed from the results of the global dynamic analysis. Local analyses for wall and slab elements subjected to peak blast pressures are carried out normally using the yield line principles with a predetermined level of ductility. Performance criteria for structural integrity of safety related structures must be established before any design work is undertaken. This will establish the levels of ductility that could be adopted in the design without compromising the nuclear shielding and containment requirements. The typical resistance-deflection curve for flexural response of reinforced concrete elements is shown in Fig.5.

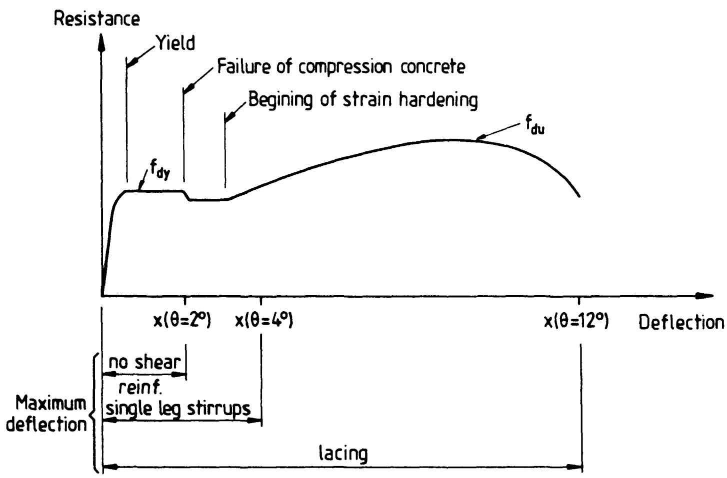

Fig 5. Typical resistance-deflection curve for flexural response of reinforced concrete elements.

9. Local analysis are also carried out to check the extent of penetration, perforation and back-face scabbing of reinforced concrete elements. These calculations are based on empirical formulae. The velocities and masses of missiles or fragments generated by an explosion are also found by empirical means established by tests. Multi-layer target penetration by fragments is carried out by the energy principle.

Parametric studies for seismic analysis of Sizewell 'B' power station

D. J. SHEPHERD and J. M. LLAMBIAS

SYNOPSIS

Response spectra have been generated for Sizewell 'B' using a soil spring method for representing soil-structure interaction. Due to the complexity of the seismic analysis problem, certain simplifying assumptions are made in the analysis route used to generate design spectra. The importance of these effects has been considered by a series of parametric studies which are used to determine whether the effects need to be considered explicitly in the analysis method used to generate design spectra.

Conclusions from these studies are:-

(i) Layering of the site should be considered explicitly in the determination of soil springs/dampers.

(ii) Frequency dependency of soil compliances can be adequately represented by frequency-independent compliances.

(iii) Embedment of the building in the soil reduces the seismic response.

(iv) Foundation lift-off does not have a significant effect on response spectra developed for Sizewell.

(v) The primary soil non-linearities can be represented by equivalent linear methods

(vi) Interaction between buildings due to coupling through the soil medium has been shown not to be significant enough to take account of in generating design spectra in preliminary studies.

(vii) Soil property variations have a significant effect on response spectra developed for Sizewell.

(viii) Structural property variations have an insignificant effect on the global response of buildings on Sizewell.

The development of anchorage details and design methods for stainless steel liners subjected to extreme loads

J. H. MILLS, PhD, Allott & Lomax, and A. R. GOULD

SYNOPSIS

In the nuclear industry stainless steel liners are applied to the walls and floors of concrete cells. The 3mm sheets are welded to steel 'T' sections made into frames that have been cast into the walls and floor screed. These 'T' sections are anchored into the concrete by short lengths of reinforcement bent through holes in their stalks.

When stainless steel liners are used as secondary containment they have to survive various extreme loads such as the accidental release of boiling liquid, or major earthquake. The performance of the liner is critically dependent upon the stiffness and strength of the anchorage details. This paper describes the full scale testing used to develop the anchorage details and the way in which the results were integrated into the design of the liners.

Initially several tests were made on the liner to establish its stress-strain characteristics and the most suitable yield criterion for biaxial membrane stress. Likewise tests were carried out on the weld details to test their ductility.

In the development of the various anchor details over 100 tests were made to obtain their force-deflection relationship up to failure load, and to demonstrate the consistency of these curves. These anchors have to be stiff enough to control the expansion of the liner, but ductile enough to even out the distribution of strains.

The tests results formed the basis of an analysis to demonstrate that there was sufficient ductility to provide an appropriate safety margin against rupture of the liner. The various encast plates and the corners of the concrete cells cause hard spots which prevent movement of the liner, so a large number of analyses were used to develop the arrangement of the stiff but ductile anchors around those areas.