6 REVIEW OF SELECTED ELEVATED-TEMPERATURE STRUCTURAL FEATURES TESTS

As noted previously, the properties of concrete can be significantly affected by changes in temperature. Concrete's thermal properties are more complex than for most materials because not only is the concrete a composite material whose components have different properties, but its properties depend on moisture content and porosity. While the properties of the steel reinforcement are relatively well understood, the interaction with concrete is not (i.e., at ambient temperatures the bonding between the reinforcement and concrete is considered complete when structural analyses are conducted; however, with an increase in temperature and/or load the bond deteriorates). Prediction of the performance of concrete structural elements at elevated temperatures is further complicated due to the presence of cracks that form. At high temperatures, correlation of cracking patterns predicted by analytical procedures with experimental results is difficult.[172] Due to the problems involved in the analytical treatment of concrete structural members, especially at elevated-temperature conditions, model tests (structural features) are often used to develop data under representative conditions. The results of these tests are then used in both the validation and the refinement of computer codes. However, the availability of data from elevated-temperature experiments in which concrete members have been subjected to controlled conditions is very limited. Available results are primarily concerned with testing of specific structural features in support of development of analytical procedures, or model tests related to gas-cooled or breeder reactor development. Also, a number of fire tests results are available (see Sect. 2.2.1). Results obtained from several of these studies are summarized below.

6.1 Structural Features Tests

At Shimizu Construction Co., Ltd.,[173] two series of experiments were carried out using 16 reinforced concrete beams to verify the thermal stress design method (TSDM) for reinforced concrete members. The thermal stress under consideration was the one that occurs only when the flexural deformation caused by a temperature gradient across the member cross-section is restrained, whereas the longitudinal deformation caused by the mean temperature change was not included. Nine tests were conducted with heating (T-tests) and seven tests without heating (S-tests). Pertinent parameters for each of these tests are presented in Table 7. A schematic of the test setup is shown in Fig. 121. In the T-test series, the temperature difference between the bottom and top surfaces of the beam was maintained by circulating hot water (60°C or 80°C) and cold water (15°C), respectively. When a steady-state temperature was attained, axial load (N) was applied by Jack No. 1 in Fig. 121 and a restraining bending moment (M*) by Jack No. 2. Figure 122 presents a typical pattern used for heating and application of loads. The S-tests used the same loading device at atmospheric temperature. In these tests, the relationship between bending moment and curvature was investigated up to yielding of the steel to compare results with the T-tests. Figure 123 presents the state of cracking in Test T7 of Table 7. The values adjacent to the cracks in the figure show the extent of crack propagation at the particular magnitude of bending moment noted. Conclusions from the study were that (1) where structural portions considered are not affected by the boundary conditions of the structure in the case of a comparatively short period of loading, the TSDM calculated thermal stresses correlate well with results obtained from both the T- and S-tests; and (2) under the same experimental conditions with respect to loading, thermal stress effects can be evaluated from the moment-curvature relationship obtained at normal temperature without necessarily performing the heating experiment.

Table 7 Pertinent Parameters for Reinforced Concrete Beam Tests (Shimizu Construction Co., Ltd.)

Kinds of test	Test beam number	Reinforcement (number and size)	Reinforce-ment ratio (%)	Material age of concrete (day)	Compressive strength of concrete (kgf/cm²)	Tensile strength of concrete (kgf/cm²)	Modulus of elasticity of concrete ($\times 10^5$ kgf/cm²)
Test–T	T1	4–D16	0.50	40	258	24	2.48
	T2	4–D16	0.50	33	267	23	2.32
	T3	4–D16	0.50	32	294	29	2.50
	T4	4–D19	0.71	56	305	28	2.66
	T5	4–D19	0.71	40	300	24	2.52
	T6	4–D19	0.71	28	282	24	2.42
	T7	4–D19	0.71	27	296	24	2.60
	T8	4–D22	0.97	21	275	24	2.49
	T9	4–D22	0.97	39	296	24	2.53
Test–S	S1	4–D16	0.50	29	281	22	2.64
	S2	4–D16	0.50	26	238	21	2.39
	S3	4–D19	0.71	26	275	26	2.28
	S4	4–D19	0.71	27	282	24	2.47
	S5	4–D19	0.71	33	257	26	2.34
	S6	4–D22	0.97	22	281	23	2.40
	S7	4–D22	0.97	35	286	26	2.40

Source: K. Irino et al., "Studies on Thermal Stress Design Method for Reinforced Concrete Members of Nuclear Power Plant," Paper J4/5, Vol. J, *Trans. of 7th Ind. Conf. on St. Mech. in Reactor Technology*, pp. 209–219, Chicago, Illinois, August 22–26, 1983.

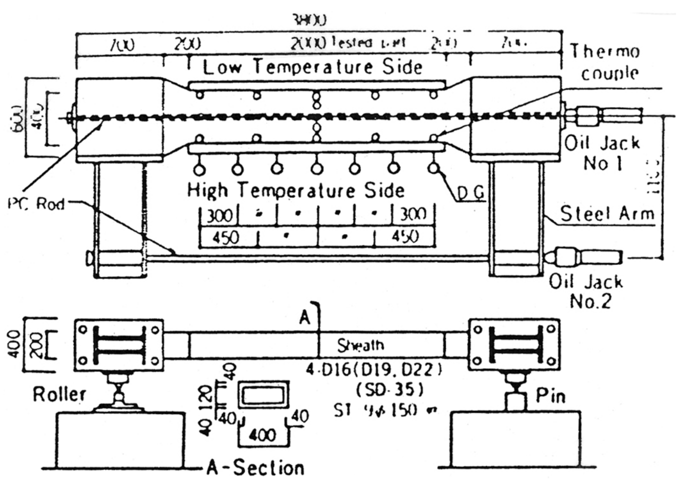


Figure 121 Test setup for investigating effect of thermal gradients on RC beam performance. *Source*: K. Irino et al., 'Studies on Thermal Stress Design Method for Reinforced Concrete Members of Nuclear Power Plant," Paper J4/5, Vol. J, *Trans. of 7th Ind. Conf. on St. Mech. in Reactor Technology*, pp. 209–219, Chicago, Illinois, August 22–26, 1983.

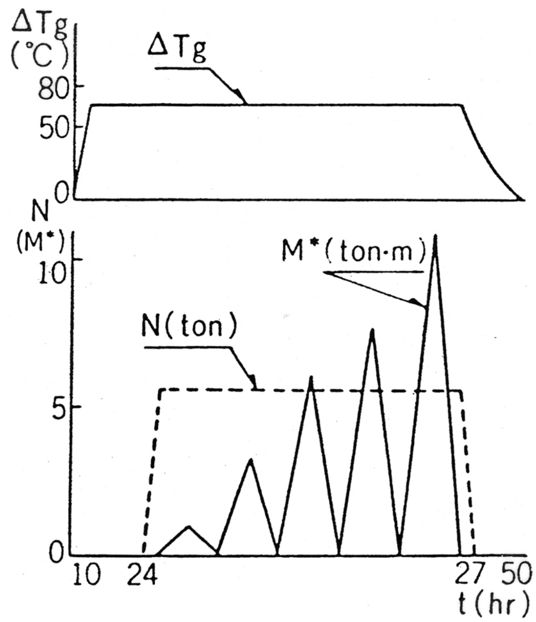


Figure 122 **Typical pattern used for heating and application of load to RC beam specimens.** *Source*: K. Irino et al., 'Studies on Thermal Stress Design Method for Reinforced Concrete Members of Nuclear Power Plant," Paper J4/5, Vol. J, *Trans. of 7th Ind. Conf. on St. Mech. in Reactor Technology*, pp. 209–219, Chicago, Illinois, August 22–26, 1983.

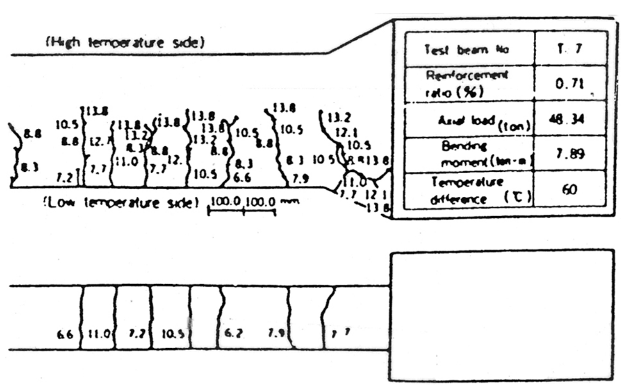


Figure 123 **Typical cracking pattern (Test T7 Table 7).** *Source*: K. Irino et al., 'Studies on Thermal Stress Design Method for Reinforced Concrete Members of Nuclear Power Plant," Paper J4/5, Vol. J, *Trans. of 7th Ind. Conf. on St. Mech. in Reactor Technology*, pp. 209–219, Chicago, Illinois, August 22–26, 1983.

An additional series of tests was conducted in the research laboratories of Shimizu Construction Co., Ltd.[174] to demonstrate the decreasing trend of bending moments and axial forces caused by cracking and creep in reinforced concrete structures. The three types of test articles (A, B, and C) used in the experiments are shown in Fig. 124. Test parameters are summarized in Table 8. A thermal gradient was applied to the column section of the models by circulating water at 80°C and 20°C in rubber bags attached to the hot and cold surfaces, respectively. The experimental setup for both applying loads and restraint is shown in Fig. 125. Models A and C had thermal deformations constrained, whereas Model B had the restraint at the model base removed so that thermal deformation could freely occur. Results of the investigation indicate that (1) where restraint is imposed, the reduction in the axial force is 3 to 5 times larger relative to that obtained for the restraining moment just after the start of testing; (2) the lower the beams rigidity (reinforcement ratio), the greater the decrease in the restraining effect against thermal deformation of a column member; and (3) both the restraining axial force and restraining moment gradually decrease with time after start of heating, eventually approaching a constant value.

Researchers at the Technical Research Institute of Ohbayashi Corporation[175] conducted heating and heating-plus-seismic loading tests at temperatures to 175°C using various concrete structural members (i.e., beams, cylindrical walls, H-section walls, and 1/10-scale models of the inner concrete (I/C) structure in a fast breeder reactor). The concrete members with relatively simple cross-sections were tested to assess the characteristics of thermal stresses and thermal cracks and the behavior of these members under combined loads. Heating and heating-plus-loading tests of the I/C structure were performed to confirm the structural performance under design loading conditions. Thirteen reinforced concrete beam specimens were tested to investigate thermal stresses and ultimate bending and shear strengths. Test variables were

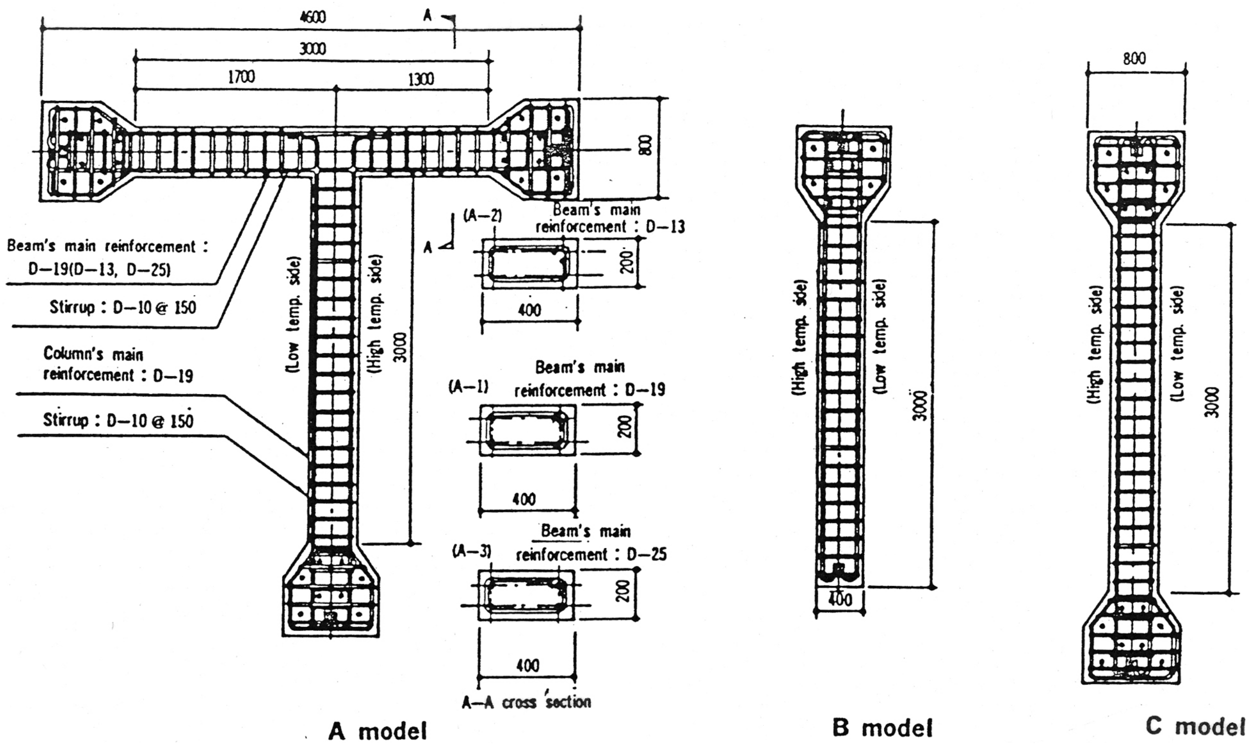


Figure 124 Test articles used to demonstrate the decreased trend of bending moments and axial forces due to cracking and creep in RC structures. *Source*: G. T. Ikoma and N. Tanaka, "Restraining Force and Moment of Reinforced Concrete Beam Column Under a Sustained Long-Term Temperature Crossfall," Vol. H, *Trans. of 9th Ind. Conf. on St. Mech. in Reactor Technology*, pp. 201–208, Lausanne, Switzerland, August 17–21, 1987.

Table 8 Test Parameters for RC Structural Element Tests (Shimizu Construction Co., Ltd.)

Test pieces		Reinforcement: Number and Diameter		Reinforcement ratio (one side) (%)	Restraint	Concrete strength in the air (kgf/cm²): At the start (normal temp.) Compressive strength	At the start (normal temp.) Young's modulus	At the end (normal temp.) Compressive strength	At the end (normal temp.) Young's modulus	At the end (80°C) Compressive strength	At the end (80°C) Young's modulus
A model	A-1	B	4D19	0.7	Yes	241.2	2.33×10^5	243.5	2.27×10^5	244.3	2.24×10^5
		C	4D19	0.7							
	A-2*	B	4D13	0.4		159.5	1.86×10^5	152.0	1.72×10^5	151.4	1.27×10^5
		C	4D19	0.7							
	A-3	B	4D25	1.2		229.6	1.81×10^5	260.2	2.25×10^5	234.3	1.80×10^5
		C	4D19	0.7							
B model	B	C	4D19	0.7	No	241.2	2.33×10^5	243.5	2.27×10^5	244.3	2.24×10^5
C model	C-1*				Yes	159.5	1.86×10^5	152.0	1.72×10^5	151.4	1.27×10^5
	C-2					229.6	1.81×10^5	260.2	2.25×10^5	234.3	1.80×10^5

B: Beam, C: Column (1 kgf=9.8N)

(*): Test piece in which cracks occurred before testing

Source: T. Ikoma and N. Tanaka, "Restraining Force and Moment of Reinforced Concrete Beam Column Under a Sustained Long-Term Temperature Crossfall," Vol. H, *Trans. of 9th Ind. Conf. on St. Mech. in Reactor Technology,* pp. 201–208, Lausanne, Switzerland, August 17–21, 1987.

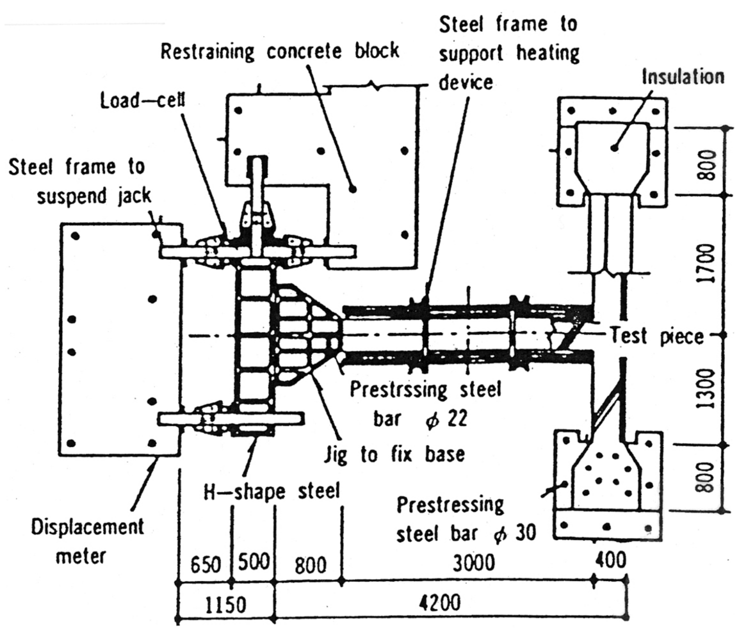


Figure 125 Setup for applying loads and restraint to test articles in Fig. 124. *Source*: T. Ikoma and N. Tanaka, "Restraining Force and Moment of Reinforced Concrete Beam Column Under a Sustained Long-Term Temperature Crossfall," Vol. H, *Trans. of 9th Ind. Conf. on St. Mech. in Reactor Technology,* pp. 201–208, Lausanne, Switzerland, August 17–21, 1987.

temperature (room, 90°C, and 175°C), loading conditions (pure flexure and combined flexure and shear), size (80 by 70 by 400 cm and 40 by 35 by 200 cm), and reinforcement ratio. Eight specimens were heated at the upper surface only, two specimens were heated at both upper and lower surfaces, and three specimens were unheated. Seven reinforced concrete cylindrical specimens (200 cm ID, 20 cm wall thickness and either 200 cm or 350 cm high) were tested to investigate thermal stresses and ultimate shear strengths. Variables in the tests were temperature condition (room, 90°C, and 175°C), loading conditions (torsional and lateral loadings), and reinforcement ratio. Five of the specimens were heated at the inner surface. H-section wall specimens (flange wall; steel plate concrete, web wall; reinforced concrete) were tested to investigate the structural behavior of a wall when adjacent walls were heated. Test variables were temperature (room and 175°C), size (660 cm long by 480 cm wide by 550 cm high, and approximately 330 cm long by 240 cm wide by 275 cm high), and web wall reinforcement ratio. Three of the specimens were heated at the outer surface of one flange wall. Two 1/10-scale I/C specimens were tested to investigate behavior of the I/C structure. One was heated and the other unheated. The loading conditions were selected to simulate design seismic loads as well as thermal loads (T_{max} = 110°C) for a sodium-leakage accident condition. Temperature dependence of concrete material properties were also evaluated for use in the nonlinear finite-element analyses of the test articles. Investigation results showed that (1) thermal deformations and stresses in specimens subjected to temperatures in excess of 100°C were markedly influenced by the temperature dependencies of the materials, especially thermal shrinkage of the concrete; (2) at early load stages for cylindrical specimens subjected to torsional or lateral loads, the thermal stresses and cracks that developed had prominent influence on behavior; however, at the ultimate stages of loading for the heated and unheated specimens, there was little difference in behavior (this was also true for the H-section wall specimens subjected to lateral loads even though a thermal strain of ~1000 $\mu\varepsilon$ occurred in vertical reinforcement in the web wall); and (3) for the I/C structures, the behavior of the heated and unheated models was similar to that observed for the cylindrical and H-section specimens, and the ultimate strength of the I/C models was about four times greater than the design seismic load.

A second study conducted at the Technical Research Institute of Ohbayashi Corporation[176] investigated the effects on temperature distribution, moisture migration, and strain variation due to heating of a simulated section of a mass concrete wall. Cube specimens 1500 mm in dimension, such as shown in Fig. 126, were tested either with or without venting systems. Five surfaces of each specimen were sealed

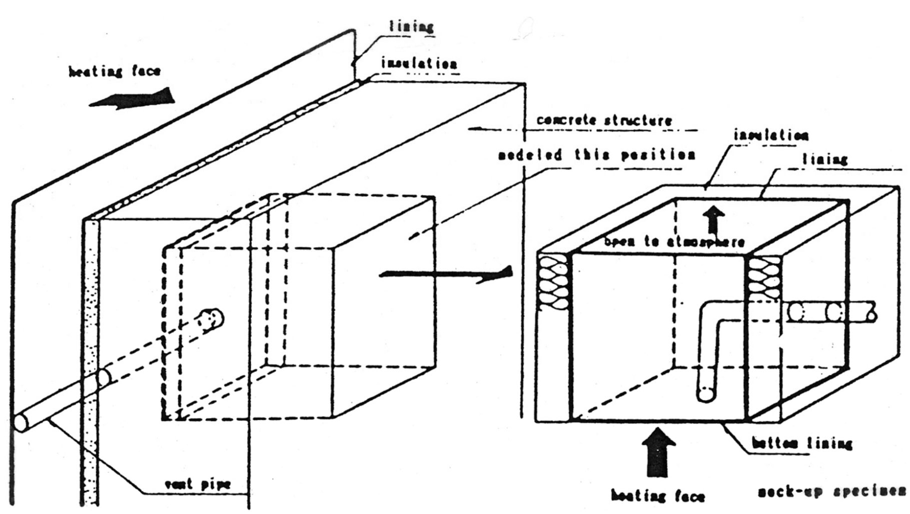


Figure 126 Simulated section of mass concrete wall. *Source*: T. Takeda et al., "Experimental Studies on Characteristics of Concrete Members Subjected to High Temperature," Vol. H. *Trans. of 9th Intl. Conf. on St. Mech. in Reactor Technology*, pp. 195–200, Lausanne, Switzerland, August 17–21, 1987.

and insulated with glass wool. During a test the bottom surface of the specimen was heated to 175°C over a 2- to 3-h period, and the temperature was maintained at this level for 91 d. Table 9 summarizes the testing conditions for this series of tests. Items measured during a test included temperature, moisture, concrete strain, water discharge from the venting system, and compressive strength and modulus of elasticity of concrete after heating. Figure 127 presents details and measurement positions for a typical

Table 9. Summary of Conditions for Simulated Mass Concrete Wall Section Tests

Items	Conditions	Items	Conditions
1) Types of specimen	Specimen with a venting system and without a venting system Two specimen in total	8) Exposure condition during heating	Top surface of the specimen is orposed to air
2) Shape and dimension of specimen	150 ×150 ×150 cm Cube	9) Temperature Control method	Electric capacity controller and temperature controller
3) Age when heated	Greater than 91 days	10) Measuring method	
4) Heating period	3 months	a. Moisuture content	Electrode method
5) Heating temperature	Surface temperature of the concrete of the bottom lining inside is constantly set at 175°C	b. Temperature	C-C thermo-couple for high temparature
		c. Inside strain	Embedment type strain gage
6) Heating method	Electric panel heater	d. Water discharge from vent pipe	Store the cooled vapor discharged from venting system
7) Curing conditions until heating begins	In-situ curing	e. Strength and elastic modulus	Core specimen

Source: T. Takeda et al., "Experimental Studies on Characteristics of Concrete Members Subjected to High Temperature," Vol. H. *Trans. of 9th Intl. Conf. on St. Mech. in Reactor Technology*, pp. 195–200, Lausanne, Switzerland, August 17–21, 1987.

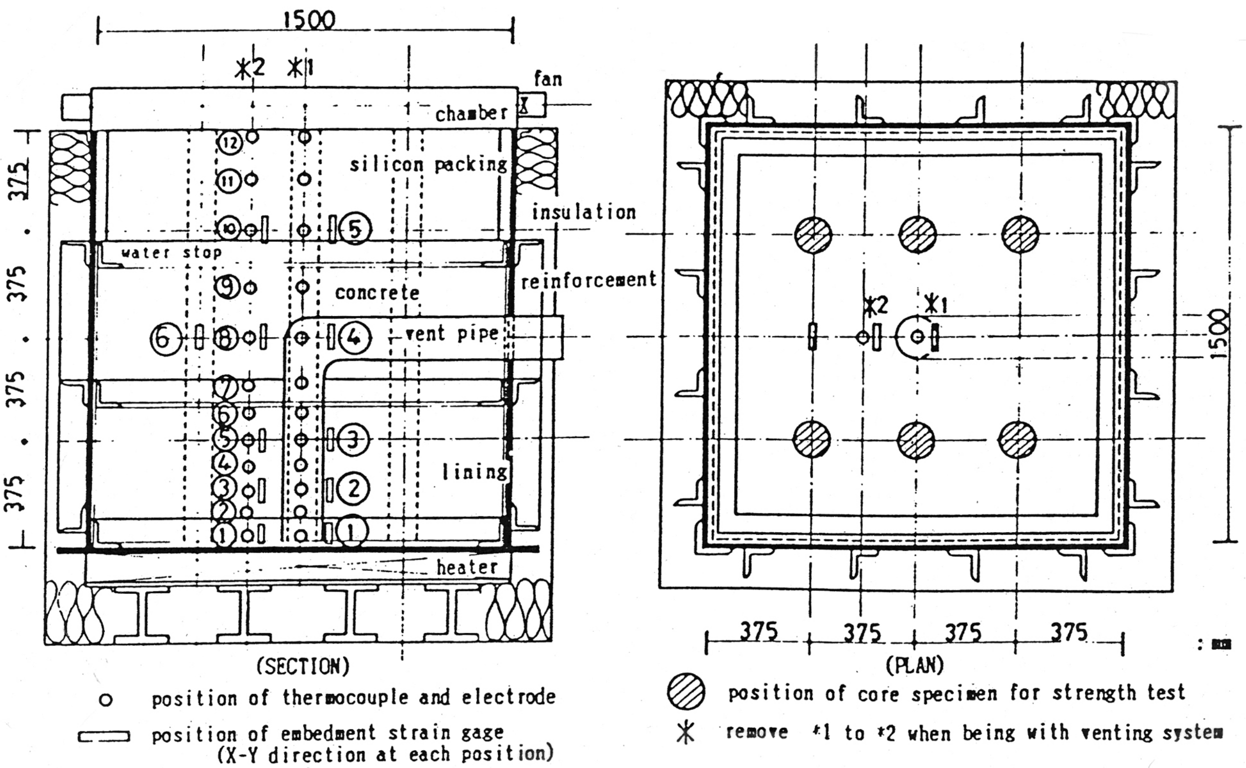


Figure 127 Details of simulated section of mass concrete wall and measurement positions. *Source*: T. Takeda et al., "Experimental Studies on Characteristics of Concrete Members Subjected to High Temperature," Vol. H, *Trans. of 9th Intl. Conf. on St. Mech. in Reactor Technology,* pp. 195–200, Lausanne, Switzerland, August 17–21, 1987.

specimen. Concrete temperature distributions at various times since the start of heating for a vented and nonvented specimen are presented in Fig. 128 and show that the temperature increase in the specimen without venting was slightly less than that for the specimen with venting, but after equilibrium was attained, the temperature distribution in the two types of specimen was almost identical. The moisture content of the specimen without a venting system decreased at a slower rate than that for the specimen with venting, Fig. 129. At 91 d after heating, the moisture distribution showed similar patterns for the two types of specimens, but the high moisture content zone was greater for the nonvented specimen. Water discharge from the venting system, shown in Fig. 130, increased relatively rapidly for the first 7 d of heating and then gradually decreased with a total of 150 L (70 L/m^2 of bottom liner) discharged over the

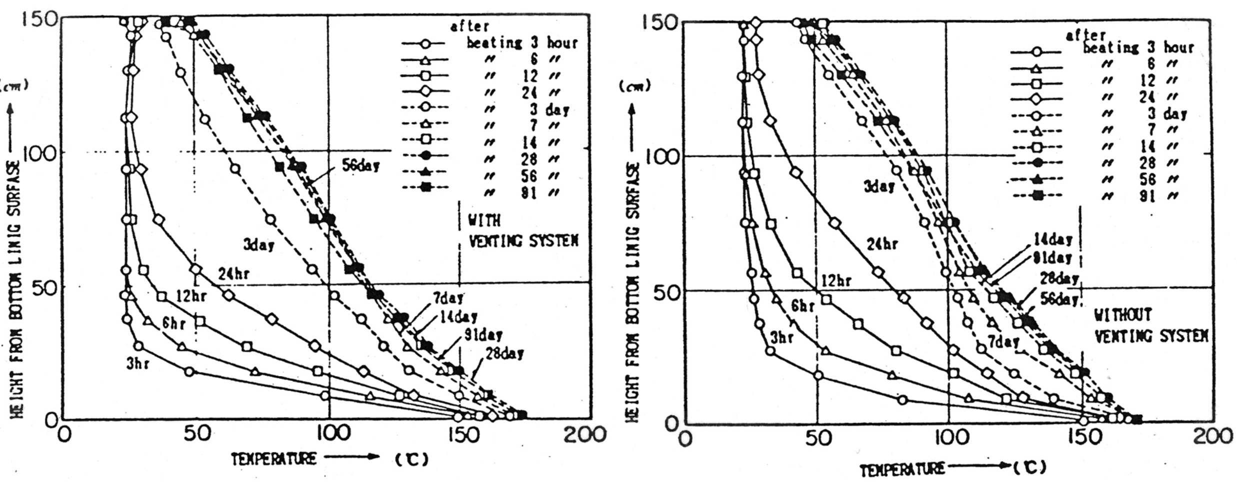


Figure 128 Temperature distribution at various times in simulated mass concrete wall with and without a venting system. *Source*: T. Takeda et al., "Experimental Studies on Characteristics of Concrete Members Subjected to High Temperature," Vol. H, *Trans. of 9th Intl. Conf. on St. Mech. in Reactor Technology*, pp. 195–200, Lausanne, Switzerland, August 17–21, 1987.

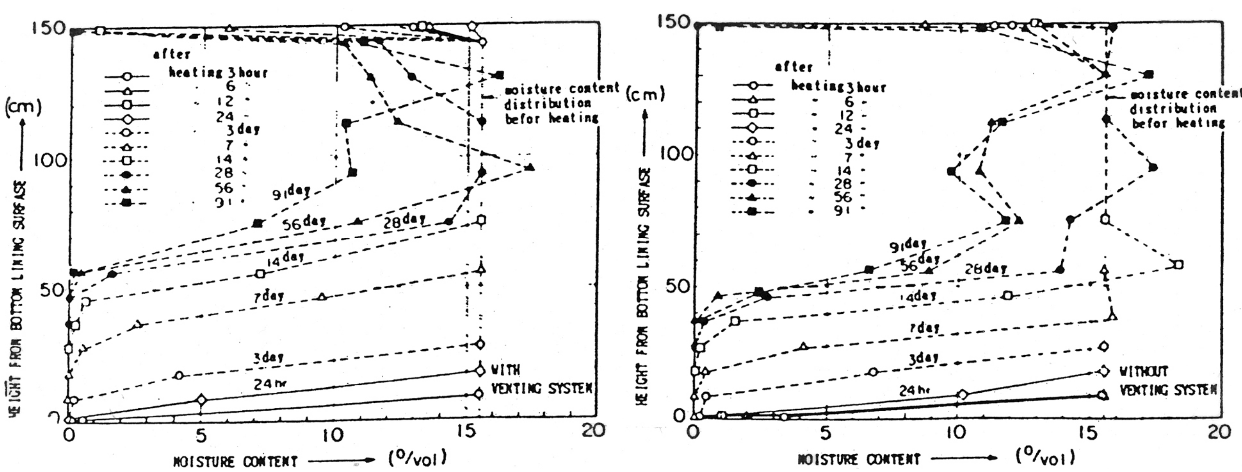


Figure 129 Moisture distribution at various times in simulated mass concrete wall section with and without a venting system. *Source*: T. Takeda et al., "Experimental Studies on Characteristics of Concrete Members Subjected to High Temperature," Vol. H, *Trans. of 9th Intl. Conf. on St. Mech. in Reactor Technology*, pp. 195–200, Lausanne, Switzerland, August 17–21, 1987.

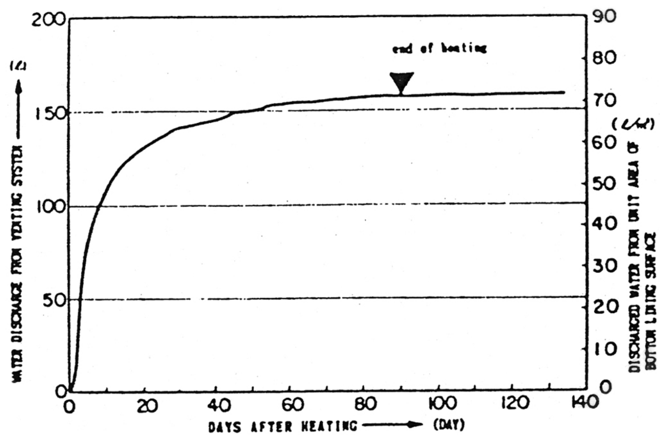


Figure 130 **Water discharge from vent pipe of simulated mass concrete wall section.** *Source*: T. Takeda et al., "Experimental Studies on Characteristics of Concrete Members Subjected to High Temperature" Vol. H, *Trans. of 9th Intl. Conf. on St. Mech. in Reactor Technology*, pp. 195–200, Lausanne, Switzerland, August 17–21, 1987.

91-d test duration. As the temperature increased, the concrete strains near the bottom liner (heated face) increased, and as heating continued the concrete strains at the unheated face increased with time, Fig. 131. Core samples removed from the specimens at conclusion of a test and tested at room temperature were used to determine the effect of heating on the concrete's compressive strength and modulus of elasticity. Test results for strength and modulus of elasticity are shown in Figs. 132 and 133, respectively. Reference values for strength and modulus of elasticity obtained from water-cured and sealed control specimens are

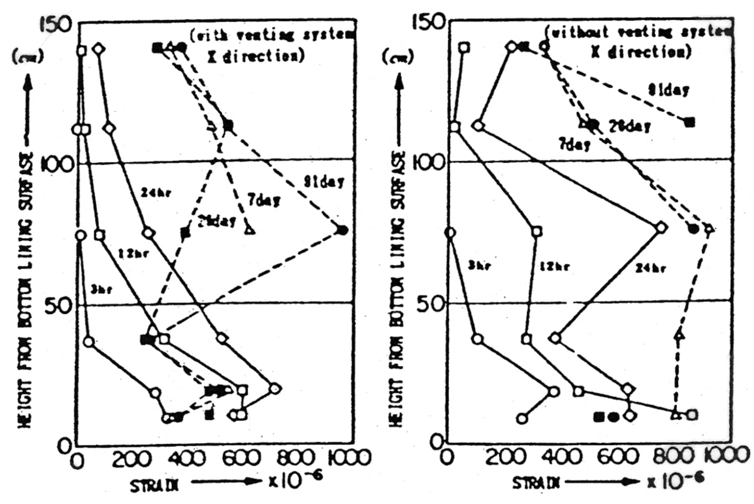


Figure 131 **Change in strain distribution with time in simulated mass concrete wall section with and without venting.** *Source*: T. Takeda et al., "Experimental Studies on Characteristics of Concrete Members Subjected to High Temperature," Vol. H, *Trans. of 9th Intl. Conf. on St. Mech. in Reactor Technology*, pp. 195–200, Lausanne, Switzerland, August 17–21, 1987.

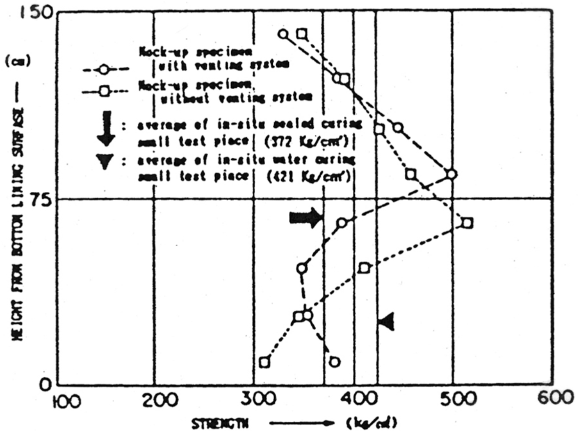


Figure 132 Compressive strength test results at selected locations in simulated mass concrete wall section with and without venting. *Source*: T. Takeda et al., "Experimental Studies on Characteristics of Concrete Members Subjected to High Temperature," Vol. H, *Trans. of 9th Intl. Conf. on St. Mech. in Reactor Technology*, pp. 195–200, Lausanne, Switzerland, August 17–21, 1987.

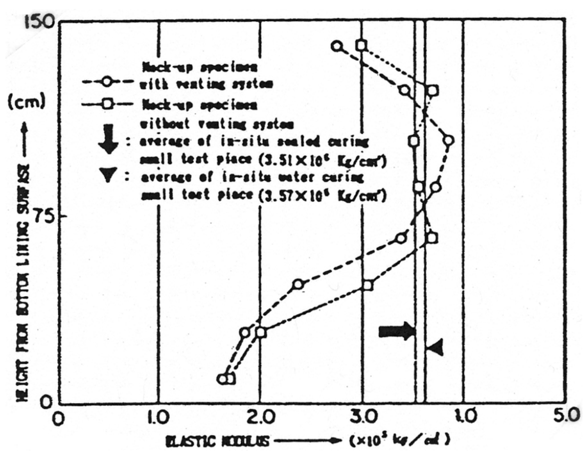


Figure 133 Modulus of elasticity test results at selected locations in simulated mass concrete wall section with and without venting. *Source*: T. Takeda et al., "Experimental Studies on Characteristics of Concrete Members Subjected to High Temperature," Vol. H, *Trans. of 9th Intl. Conf. on St. Mech. in Reactor Technology*, pp. 195–200, Lausanne, Switzerland, August 17–21, 1987.

also shown in the appropriate figure. The effect of the elevated temperature was most significant on the concrete modulus of elasticity, which decreased up to about 40%, relative to sealed control specimens, near the bottom face of the specimen. Compressive strength results at all locations in the test specimens exceeded the design strength (240 kg/cm^2).

At the Central Research Institute of Electric Power Industry (CRIEPI),[177] reinforced concrete structures were subjected to elevated temperatures (room to 300°C) to determine the effect on their behavior of (1) change in physical properties of materials, (2) difference in coefficients of thermal expansion between steel reinforcement and concrete, and (3) creep and drying shrinkage of concrete due to water movement. Table 10 summarizes the status of the theoretical and experimental investigations (as of September 1987). The overall objective of the investigations is to develop elevated-temperature design methods for reinforced concrete structures. Figures 134–139 present schematics of the test articles utilized in the extensive CRIEPI test program. Objectives of the various experimental studies are (1) temperature stress tests (Figs. 134 and 135)—measure temperature stress directly and comprehend the influence of creep and drying shrinkage of concrete on temperature stress in reinforced concrete members; (2) shear resistance tests (Fig. 136)—evaluate influence of internal stress, caused by coefficients of thermal expansion of concrete and rebar, on shear transfer behavior, and confirm the shear resistance capacity of reinforced concrete at elevated temperature; (3) material creep test (Figs. 137 and 138)—determine creep and drying shrinkage of concrete at various temperature conditions; (4) flexural creep test of reinforced concrete beams (Fig. 139)—evaluate the influence of creep and drying shrinkage of concrete on the long-term flexural behavior of reinforced concrete beams; (5) flexural test of reinforced concrete beam with lap splice—determine influence of internal stress on the strength and deformation capacities of a lap splice section of a reinforced concrete beam; and (6) anchorage and bond tests—evaluate the influence of the internal stress on the anchorage strength of reinforced concrete. Throughout the test program, an ordinary

Table 10 Identification/Status (September 1987) of Experimental and Analytical Investigations at CRIEPI

	Finished	Continued
Experiment	(1) Investigation of temperature dependence of physical properties of concrete and reinforcement (2) Flexural behaviour of RC beams at elevated temperatures up to 500℃ (3) Flexural behaviour of RC beams with axial compressive stress at elevated temperatures up to 200℃	(1) Creep of concrete at elevated temperature (2) Flexural creep of RC beams at elevated temperatures (3) Temperature stress test of RC beams (4) Shear resistance of RC members at elevated temperatures
Theoretical Investigation	(1) Temperature dependence of physical properties of concrete and reinforcement (2) Estimation method of flexural behaviour of RC beams at elevated temperatures (3) Estimation method of flexural behaviour of RC beams with axial compressive stress at elevated temperatures	(1) Application of Finite Element Method (2) Estimation of creep of concrete material and RC beams (3) Estimation of temperature stress

Source: "High-Temperature Concrete-Testing and Data," *8th CRIEPI/EPRI FBR Workshop,* Palo Alto, California, September 23–25, 1987.

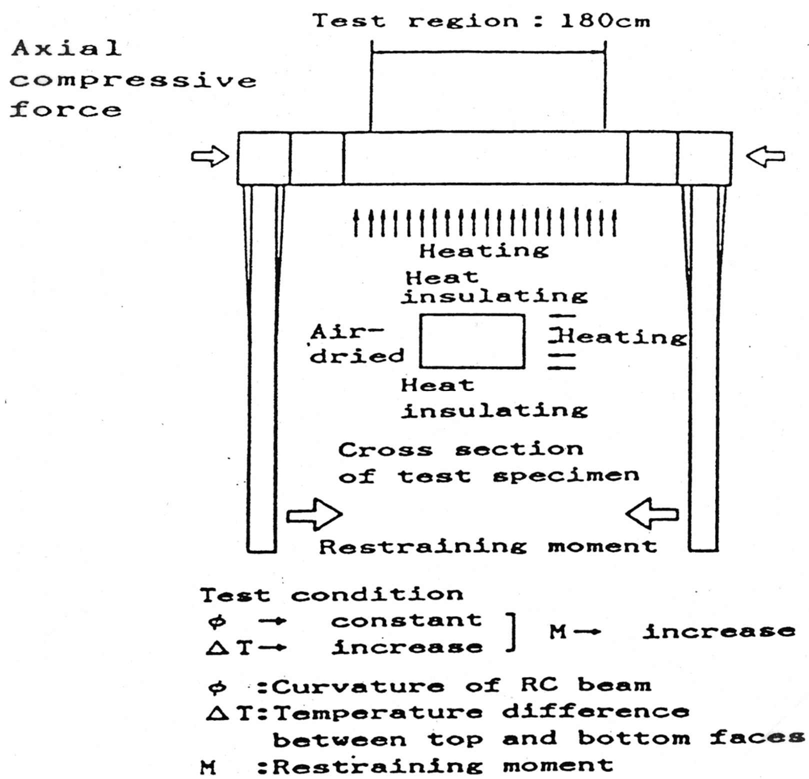


Figure 134 **Temperature stress test sponsored by CRIEPI.** *Source:* "High-Temperature Concrete-Testing and Data," *8th CRIEPI/EPRIFBR Workshop*, Palo Alto, California, September 23–25, 1987.

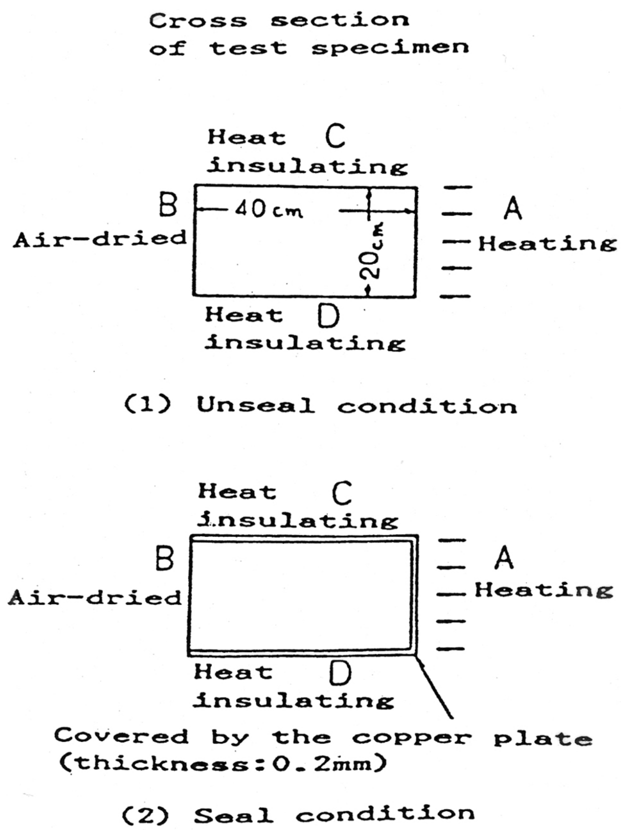


Figure 135 Sealed and unsealed conditions for reinforced concrete beams in temperature stress test series sponsored by CRIEPI.
Source: "High-Temperature Concrete-Testing and Data," *8th CRIEPI/EPRI FBR Workshop,* Palo Alto, California, September 23–25, 1987.

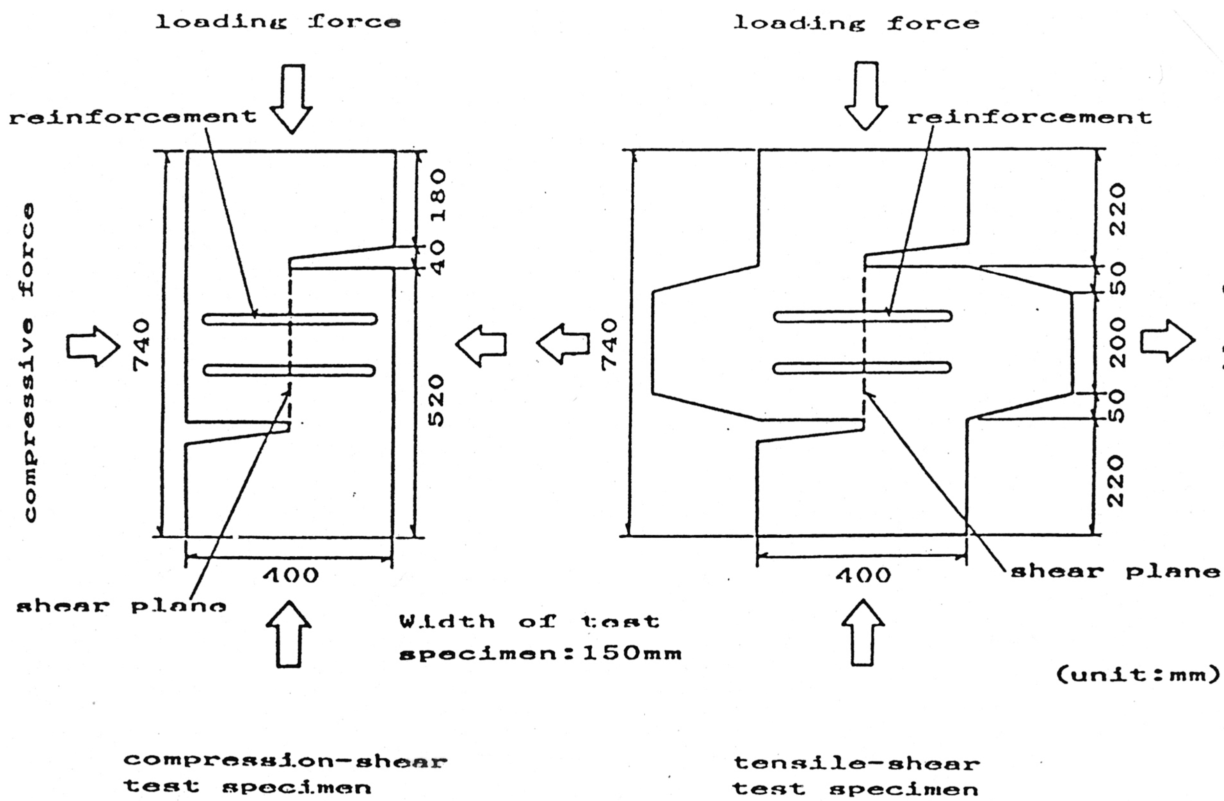


Figure 136 **Shear resistance test article used in CRIEPI test program.** *Source:* "High-Temperature Concrete-Testing and Data," *8th CRIEPI/EPRIFBR Workshop,* Palo Alto, California, September 23–25, 1987.

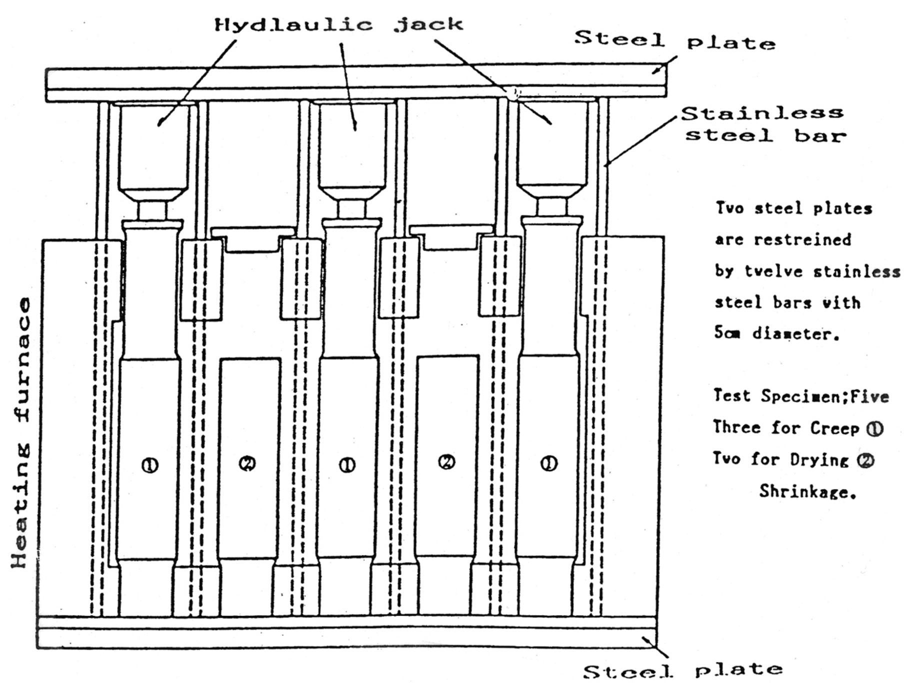


Figure 137 **Creep apparatus used in CRIEPI test program.** *Source:* "High-Temperature Concrete-Testing and Data," *8th CRIEPI/EPRI FBR Workshop,* Palo Alto, California, September 23–25, 1987.

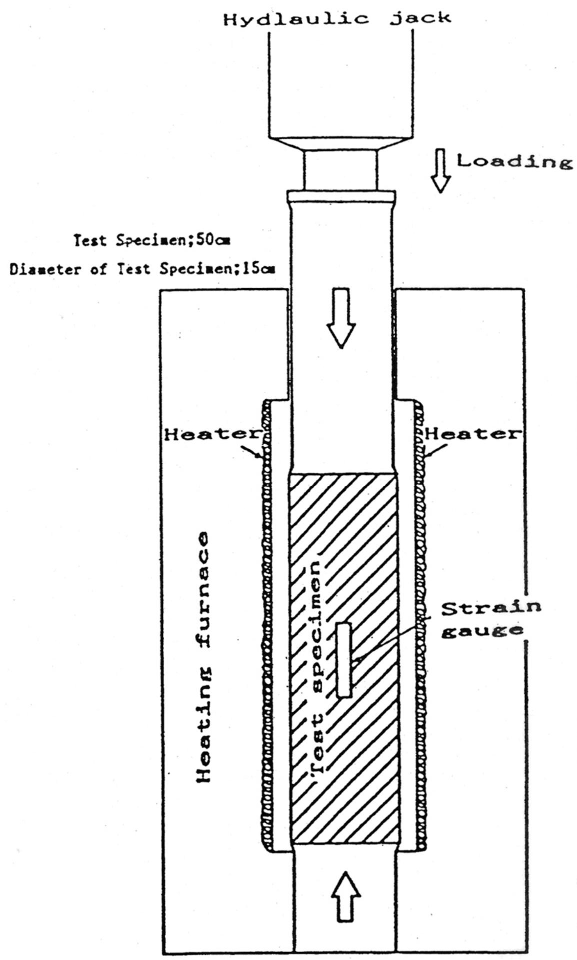


Figure 138 Close-up of creep specimen used in CRIEPI test program. *Source:* "High-Temperature Concrete-Testing and Data," *8th CRIEPl/EPRI FBR Workshop*, Palo Alto, California, September 23–25, 1987.

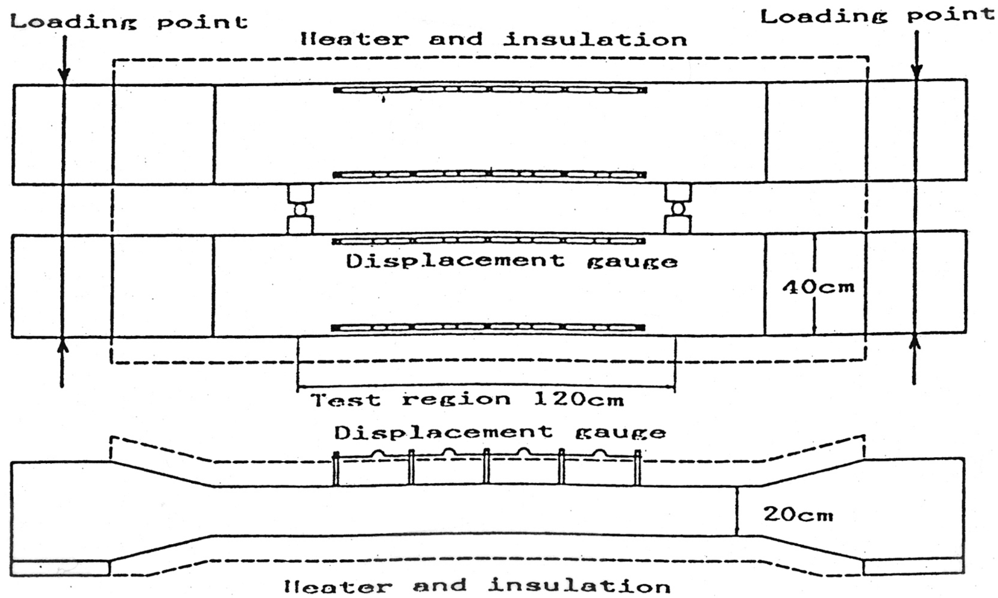


Figure 139 Test setup used for CRIEPI flexural creep tests of reinforced concrete beams at elevated temperature. *Source:* "High-Temperature Concrete-Testing and Data," *8th CRIEPI/EPRI FBR Workshop*, Palo Alto, California, September 23–25, 1987.

Portland cement concrete (greywacke and tuff coarse aggregate; chert, andesite, slate, granite and sandstone fine aggregate) having a compressive strength of 400 kgf/cm^2 was utilized.

Two reinforced concrete beam specimens, Fig. 140, representing portions of the walls or slabs of the fuel storage pool of a boiling-water reactor (BWR) building, were tested to evaluate the effect of thermal

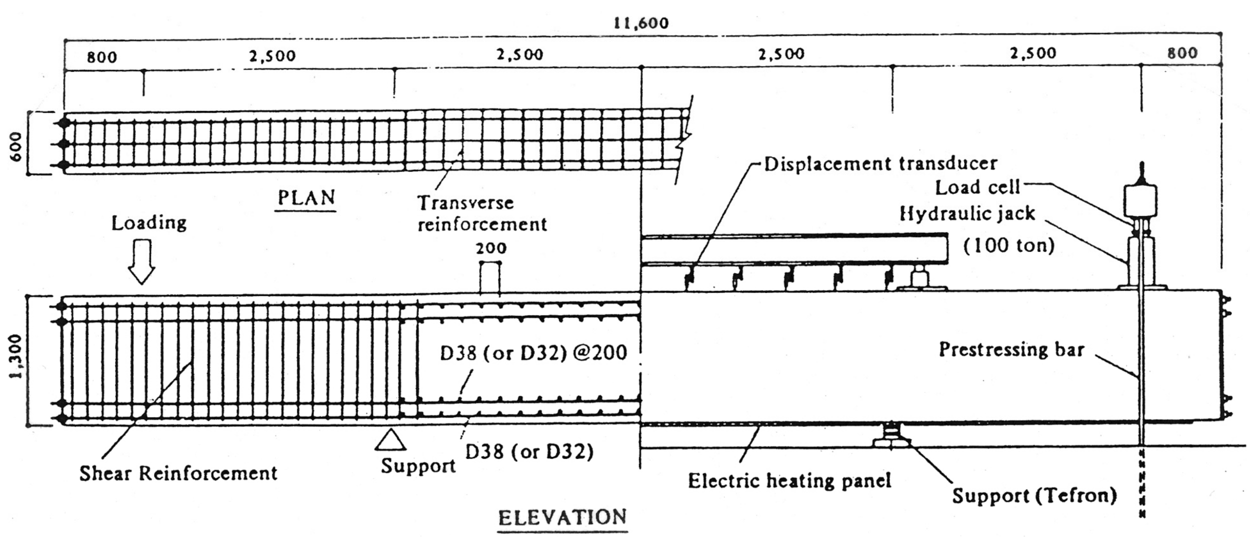


Figure 140 Reinforced concrete beam specimens tested to evaluate thermal cracking and thermal stress relaxation due to cracking. *Source:* N. Shibasaki et al., "Thermal Cracking and Thermal Stress Relaxation of Reinforced Concrete Member Tested by Full Sized Beam Specimens," Paper J4/2, Vol. J, *Trans. of 7th Intl. Conf. on St. Mech. in Reactor Technology*, p. 179–187, Chicago, Illinois, August 22–26, 1983.

cracking and thermal stress relaxation due to cracking.[178] Properties of the specimens are given in Table 11. The bottom surface of each specimen was heated over a 48-h period from room temperature (10°C) to 65°C using electric resistance panels. The temperature was then maintained at this level throughout the test duration. The upper surfaces of the beams were exposed to room air. After the temperature distributions in the beam cross-sections attained steady state, restraint moments were applied by hydraulic jacks at both ends of the specimens to return the free bending deformation to zero (i.e., thermal-stress-only condition was simulated). The restraining moments were then gradually increased until ultimate conditions were attained. Crack patterns in specimen D38 (Table 11) due to thermal stress only and thermal stress plus mechanical load are shown in Fig. 141. Cracking occurred along transverse reinforcing bars with maximum crack widths of 0.10 mm and 0.18 mm occurring in specimens D38 and D32, respectively, as a result of thermal stress only. Measured crack widths were compared with values calculated using several published crack-width formulas (i.e., CEP-FIP formula,[179]) and the calculated values were slightly larger.

Table 11 Properties of Reinforced Concrete Beam Specimens Tested to Investigate Thermal Cracking and Thermal Stress Relaxation

Specimen			D38		D32	
Reinforcement arrangement			D38 (#12) – 6 bars		D32 (#10) · 6 bars	
Reinforcement ratio (%)			0.877		0.611	
Concrete cover (mm)			90		90	
Transverse reinforcement arrangement			2 Layers – D38 @200mm		2 Layers – D32 @200mm	
Mechanical properties of materials	Rebar	Yield strength (kg/cm^2)	4000		3819	
		Modulus of elasticity ($\times 10^6$ kg/cm^2)	1.93		1.94	
	Concrete	Compressive strength (kg/cm^2)	10°C (51 days)	65°C (51 days)	10°C (67 days)	65°C (67 days)
			253.2	258.8	234.4	229.5
		Modulus of elasticity ($\times 10^5$ kg/cm^2)	2.88	2.20	2.49	2.25
		Tensile strength (kg/cm^2)	24.4	20.7	24.4	19.2
		Coefficient of thermal expansion ($\times 10^{-5}$/°C)	0.695	—	—	—
Temperature difference between top and bottom surface (°C)			45.5		48.0	

Source: N. Shibasaki et al., "Thermal Cracking and Thermal Stress Relaxation of Reinforced Concrete Member Tested by Full Sized Beam Specimens," Paper J4/2, Vol. J, *Trans. of 7th Ind. Conf. on St. Mech. in Reactor Technology,* p. 179–187, Chicago, Illinois, August 22–26, 1983.

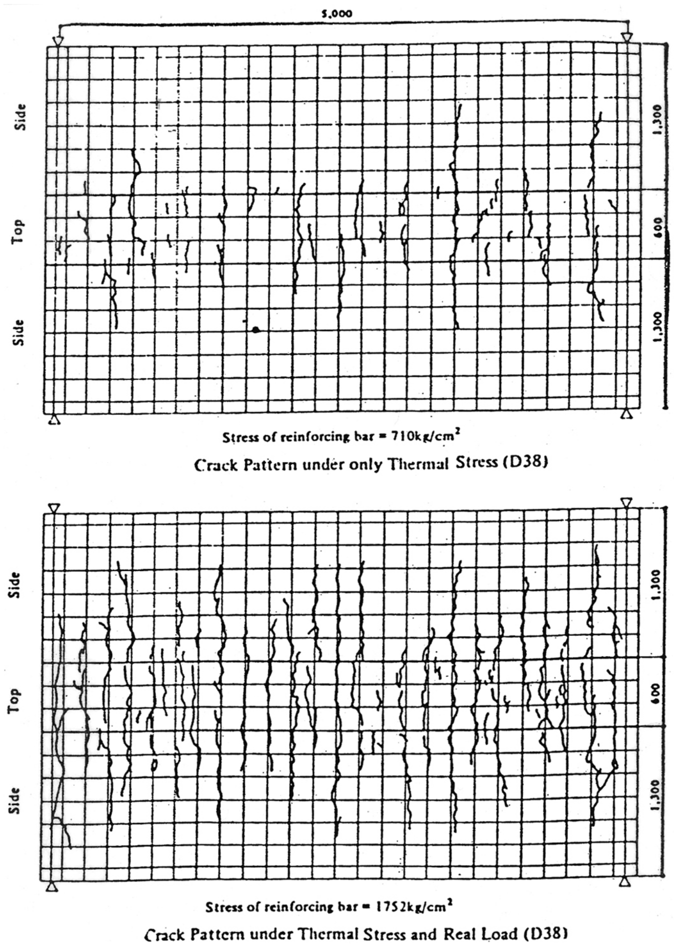


Figure 141 Crack patterns for specimen D38 (Table 11) due to thermal stress only and thermal stress with loading. *Source:* N. Shibasaki et al., "Thermal Cracking and Thermal Stress Relaxation of Reinforced Concrete Member Tested by Full Sized Beam Specimens," Paper J4/2, Vol. J, *Trans. of 7th Intl. Conf. on St. Mech. in Reactor Technology*, p. 179–187, Chicago, Illinois, August 22–26, 1983.

Nine reinforced concrete beams (Fig. 142) were tested to evaluate the thermal stress produced by restraining the deflections produced by a thermal gradient.[180] Table 12 summarizes material properties and test parameters for the study. Figure 143 presents the test apparatus. Axial force and moment acting on the specimens were produced using hydraulic jacks positioned as shown in the figure. Two primary types of specimens were tested: Type T and Type E. The loading procedure for the Type T tests included (1) specimen heated to 75°C at one face while cooled at 10°C on opposite face, specimen allowed to freely deflect; (2) after temperature distribution reached steady-state (~19 h), external axial force was applied as well as a pure moment at each end to restrain free thermal deflection; and (3) pure moment was increased until ultimate load was reached. The procedure for the Type E tests included (1) axial force and pure moment loads were applied to the specimen, (2) while maintaining the external loads, the specimen was allowed to deflect freely as it was heated to 50°C at one face and cooled to 10°C on opposite face; (3) after temperature distribution reached steady-state (~17 h), free thermal deflection was restrained by applying pure moment; (4) while holding above state, specimen was allowed to freely deflect while temperature at hot face was rapidly increased to 95°C; (5) while in an unsteady-state of nonlinear temperature distribution across the beam, pure moment was applied to restrain the free thermal deflection; and (6) pure moment was increased until ultimate load was reached. Test durations were kept short to reduce creep effects. When comparing the relationship between external thermal moment and external moment for all specimens, the following was observed: (1) thermal moments decrease with an increase in

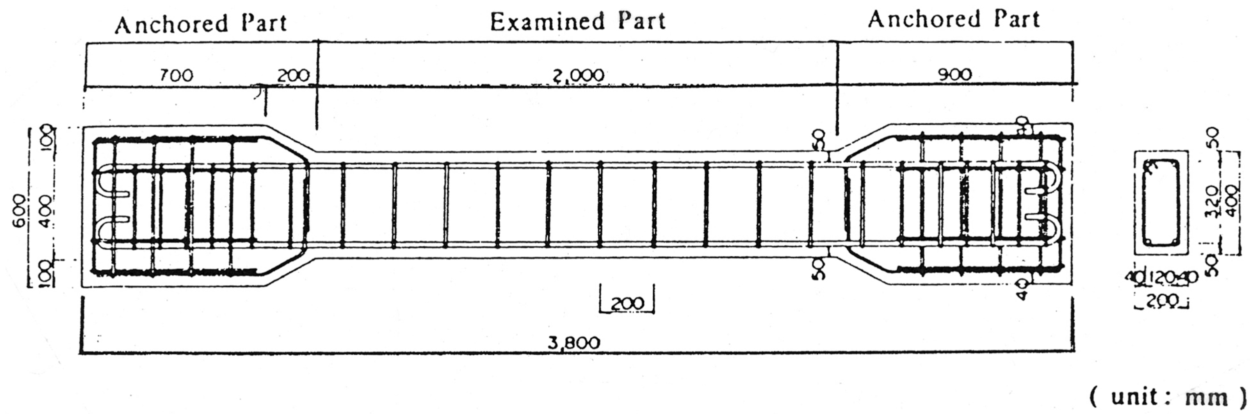


Figure 142 Test specimen utilized to evaluate thermal stress produced by restraining deflections produced by thermal gradients. *Source:* N. Shibasaki et al., "An Experimental Study on Thermal Stress of Reinforced Concrete Members Under Short-Term Loading," Paper J4/3, Vol. J, *Trans. of 7th Ind. Conf. on St. Mech. in Reactor Technology*, pp. 189–197, Chicago, Illinois, August 22–26, 1983.

Table 12 Material Properties and Test Parameters for Reinforced Concrete Thermal Gradient Experiments Conducted to Evaluate Stresses Produced by Restraining Deflections

Experimental Procedure		Type-T					Type-E			
Specimen		TC−0.71	TC−1.27	TO−0.71	TT−0.71	TT−1.27	EC−0.71	EC−1.27	ET−0.71	ET−1.27
External Axial Force (ton)		−10		0	15		−10		15	
External Moment (ton·m)		Optional					1.0		2.0	
Temperature Difference at Steady-state (°C)		60					35			
Hot Side Water Temperature at Unsteady-state (°C)							95			
Amount of Reinforcement (%)		0.71	1.27	0.71	0.71	1.27	0.71	1.27	0.71	1.27
Concrete	Compressive Strength (kg/cm^2)	301	326	313	333	269	302	299	290	289
	Tensile Strength (kg/cm^2)	30.1	28.5	27.6	26.4	29.5	26.4	25.4	28.6	26.5
	Young's Modulus ($\times 10^5$kg/cm^2)	3.21	3.08	2.94	3.33	3.12	3.14	3.59	3.28	2.70
Re-bar	Yield Strength (kg/cm^2)	3700	3800	3700	3700	3800	3700	3800	3700	3800
	Ultimate Strength (kg/cm^2)	5230	5430	5230	5230	5430	5230	5430	5230	5430
	Young's Modulus ($\times 10^6$kg/cm^2)	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98	1.98

TC−0.71

- 0.71 → Percentage of Tensile Reinforcement
- C → Identification of Axial Force C: Compression, O: Non-axial Force, T: Tension
- T → Type of Experimental Procedure

Source: N. Shibasaki et al., "An Experimental Study on Thermal Stress of Reinforced Concrete Members Under Short-Term Loading," Paper J4/3, Vol. J, *Trans. of 7th Intl. Conf. on St. Mech. in Reactor Technology*, pp. 189–197, Chicago, Illinois, August 22–26, 1983.

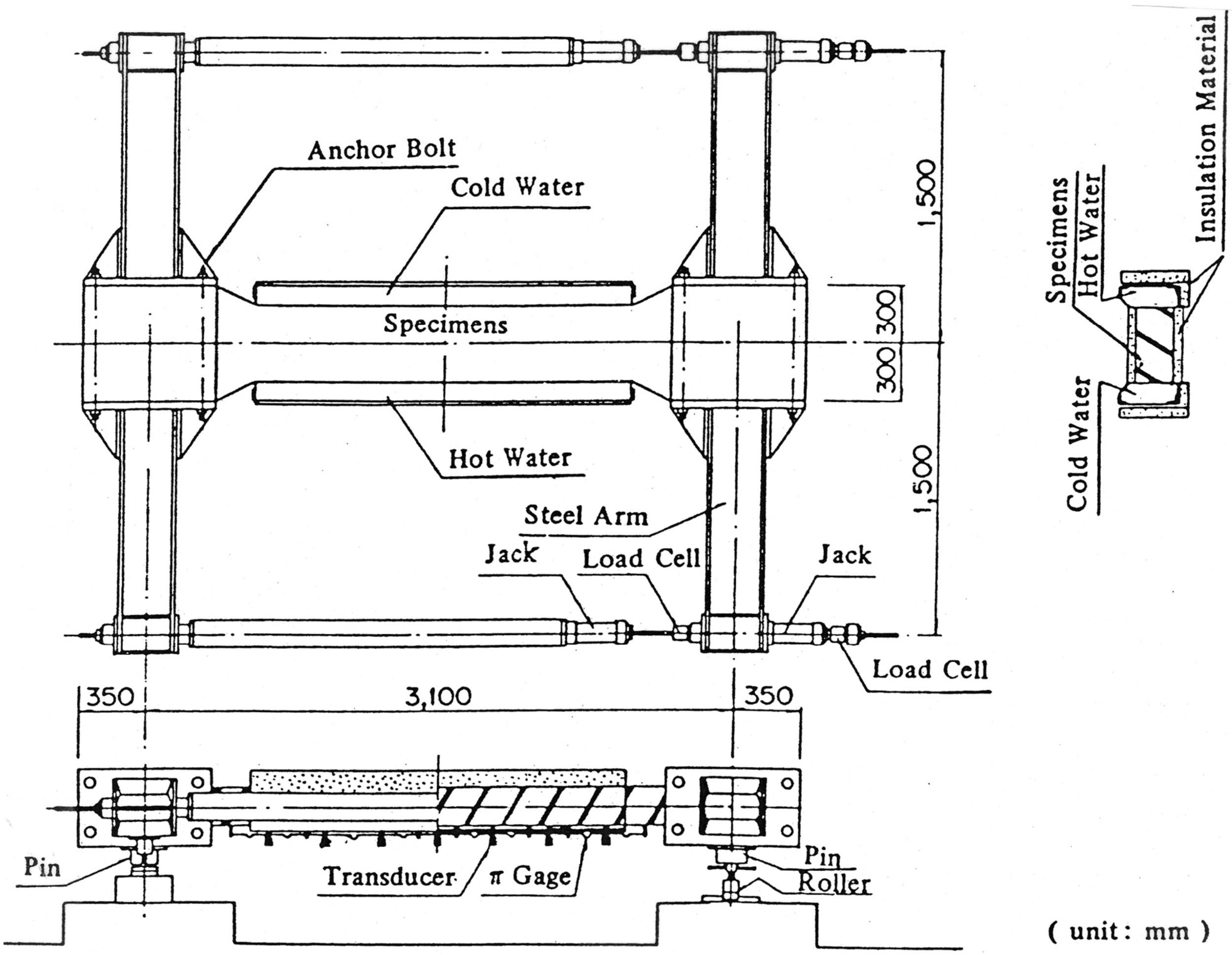


Figure 143 Apparatus used to test specimen shown in Fig. 142. *Source:* N. Shibasaki et al., "An Experimental Study on Thermal Stress of Reinforced Concrete Members Under Short-Term Loading," Paper J4/3, Vol. J, *Trans. of 7th Ind. Conf. on St. Mech. in Reactor Technology,* pp. 189–197, Chicago, Illinois, August 22–26, 1983.

external moments; (2) when specimens are subjected to the same axial forces, thermal moments increase as the amount of steel reinforcement increases; and (3) when the specimens have the same amount of steel reinforcement and different axial forces are applied, thermal moments decrease with increasing compressive load, nonaxial loading, and tensile loading.

Four beam specimens (Fig. 144) having identical dimensions and steel reinforcement were tested to investigate the time-dependent thermal effects either with or without application of external forces.[181] Test parameters are summarized in Table 13. All surfaces of the specimens, except those exposed to ambient conditions, were sealed with neoprene rubber sheets to prevent moisture migration. The test apparatus used to apply thermal moment and the sustained external moment is shown in Fig. 145. Thermal gradients of 40°C and 70°C at the heating surface were selected to simulate operating conditions in a nuclear power plant. A "thermal" moment was applied by mechanical jacks at both ends of a specimen to cancel out the deflection induced by the thermal gradient. The sustained external moment was applied and kept constant during the testing period by spring elements. Any changes in curvature during the ~4-month test period was adjusted by controlling the moment so that the thermal curvature was kept constant at zero (i.e., reduction in thermal moment was observed by measuring the change of the

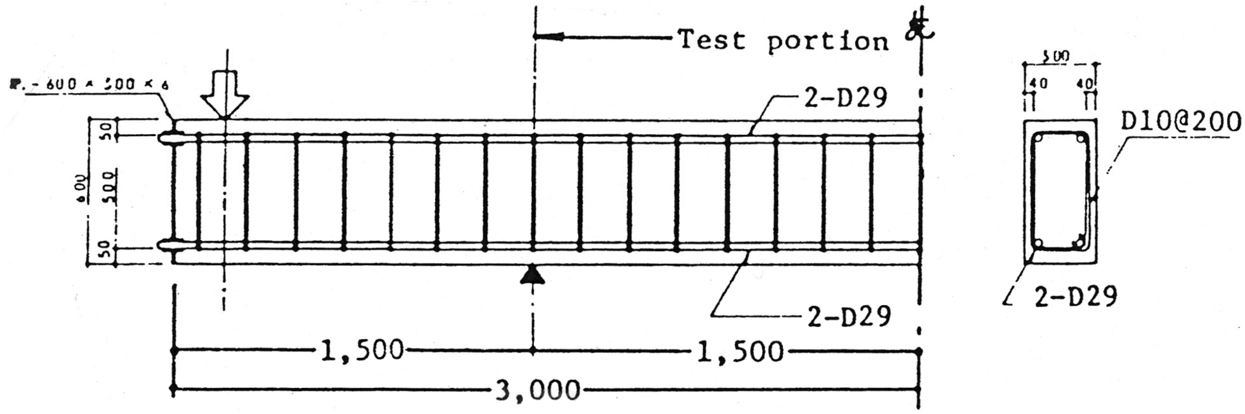


Figure 144 Test specimen utilized to investigate the time-dependent thermal effects either with or without application of external forces. *Source:* N. Shibasaki et al., "Thermal Stress Relaxation and Creep Tests of Reinforced Concrete Beams Under Long Term Thermal Effects and Loadings," Paper J4/4, *Trans. of 7th Int. Conf. on St. Mech. in Reactor Technology,* pp. 199–207, Chicago, Illinois, August 22–26, 1983.

Table 13 Parameters of Reinforced Concrete Beam Specimens Tested to Investigate Time-Dependent Thermal Effects Either With or Without External Forces

Test specimen	Test type	ΔT (deg. C)	M_F (t.m)	T_i (°C)
RH-1	Relaxation	40	0	70
RH-2	Relaxation	40	6.9	70
MH-1	Creep	40	6.9	70
MC-1	Creep	0	6.9	-

where ΔT : thermal gradient across the depth of beams
M_F : external moment
T_i : temperature at heated surface

Source: N. Shibasaki et al., "Thermal Stress Relaxation and Creep Tests of Reinforced Concrete Beams Under Long Term Thermal Effects and Loadings," Paper J4/4, *Trans. of 7th Int. Conf. on St. Mech. in Reactor Technology,* pp. 199–207, Chicago, Illinois, August 22–26, 1983.

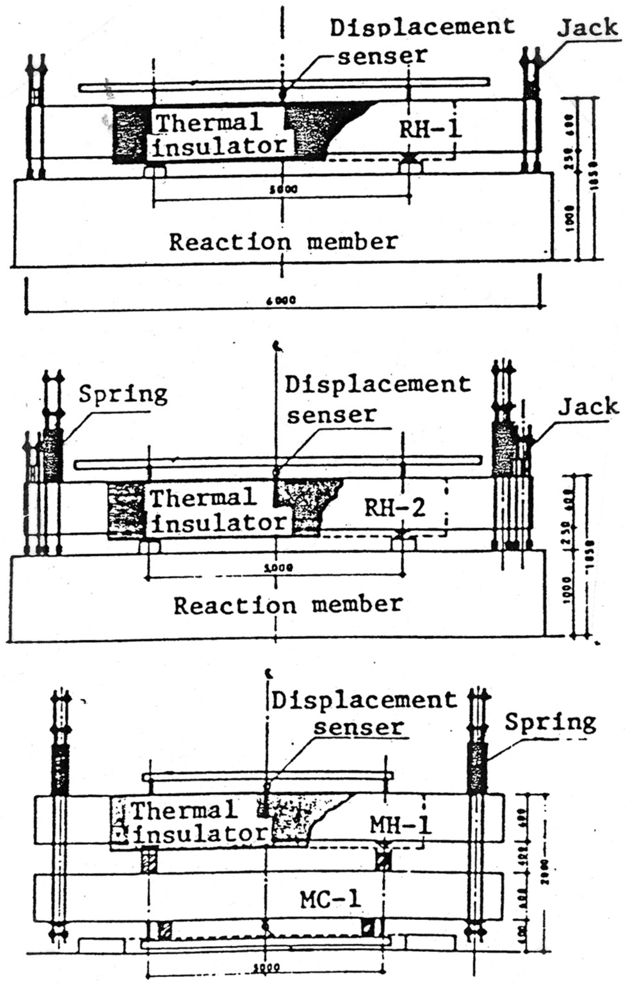


Figure 145 Apparatus used to apply thermal moment and sustained external moment test specimen shown in Fig. 144. *Source:* N. Shibasaki et al., "Thermal Stress Relaxation and Creep Tests of Reinforced Concrete Beams Under Long Term Thermal Effects and Loadings," Paper J4/4, *Trans. of 7th Int. Conf. on St. Mech. in Reactor Technology*, pp. 199–207, Chicago, Illinois, August 22–26, 1983.

moment). When subjected to a constant thermal gradient only, results showed that due to development of cracks the thermal moment decreased rapidly early in the loading stage. Crack widths estimated using a model such as proposed in Ref. 179 were considerably smaller than the test results at 4 months loading, probably due to the thermal effects such as concrete creep at elevated temperature.

A reinforced concrete box structure (Fig. 146) was subjected to thermal and mechanical loads to determine the general behavior of reinforced concrete at elevated temperatures and to develop a data base for verification and/or calibration of analytical procedures.[182] The test was conducted in two phases. The purpose of Phase I (concrete age 4–5 months) was to evaluate the response of the structure to a simulated sodium spill. Cracking patterns, temperatures, strains, displacements, and changes in stiffness of the structure were evaluated while the cell was heated to 205°C at ~6°C/h, maintained at temperature for

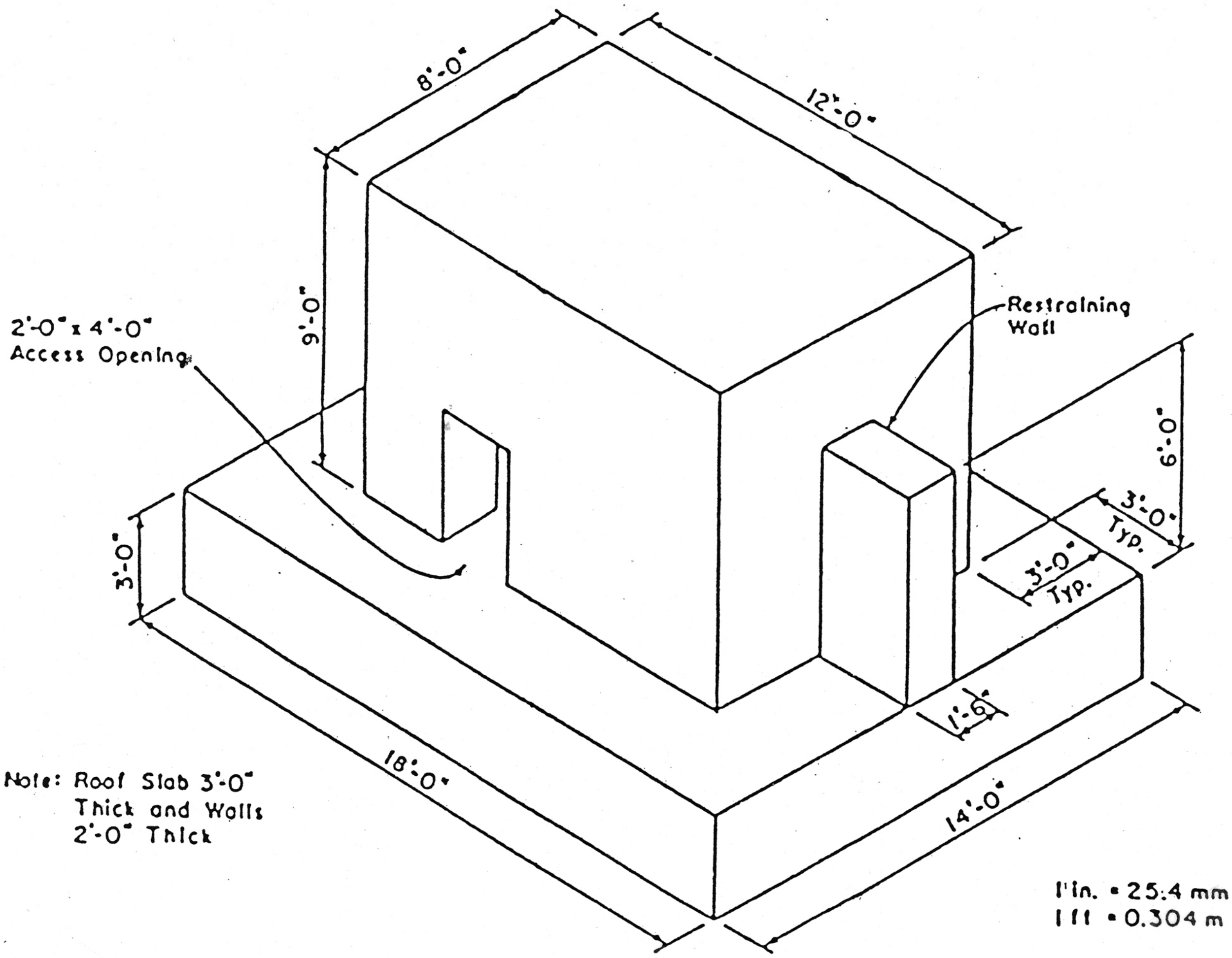


Figure 146 Reinforced concrete box structure subject to thermal and mechanical loads to determine the general behavior of reinforced concrete at elevated temperature. *Source:* G. N. Freskakis, "High Temperature Concrete Testing," *8th CRIEPI/EPRI Workshop*, Agenda Item 7.2, Palo Alto, California, September 23–25, 1987.

100 h, and cooled down. Mechanical load tests were conducted before heatup and after cooldown. Results of Phase I were that (1) stresses in the reinforcing steel increased during heatup, stabilized during the constant temperature period (stresses highest in top slab and near top of walls), and decreased during the cooldown period (stresses, however, were higher than expected); (2) bending moments were large during initial stages of heatup but dropped sharply as cracking developed (high moments occurred near top of walls and in top slab); (3) small, almost negligible, axial forces occurred; (4) cracks that occurred in exterior walls were extensive but uniform having small crack widths and closed after cooldown; (5) substantial water release and seepage through the cracks occurred; (6) stiffness was reduced 60% after heating; and (7) approximate analysis methods produced good agreement with experimental results at the center of cell except at restraints and discontinuities that were not accounted for in the analysis (further analytical verification is required to eliminate the factor required to account for two-dimensional effects and associated conservatism). The overall physical condition of the test structure at conclusion of the investigation was judged to be very good. The purpose of Phase II (concrete age 20 months) was to simulate temperature conditions of a second sodium spill in a plant in order to determine if the plant could be reused after an initial spill. Items measured and test procedures utilized were the same as for Phase I.

Although detailed results were not available for inclusion in the reference, some general comments were made relative to the Phase II test: (1) exterior cracking was the same as occurred in Phase I (delayed in opening), and the cracks closed on cooldown; (2) no additional water was released, and (3) results of the mechanical load tests indicated that the section stiffness had increased 25% since Phase I. The overall physical condition of the test structure at conclusion of Phase II was also judged to be very good.

6.2 Model Tests in Support of PCRV Development

In the *ASME Code for Concrete Reactor Vessels and Containments*[1] the use of models in support of the development of PCRVs is required where accurate analytical procedures for the ultimate strength and behavior in the range approaching failure have not been established or when models of a prototype with similar characteristics to those of the current design have not been constructed and tested. The models are required to maintain similitude, including that of materials, to the prototype design and be geometrically similar with respect to the principal dimensions of the prototype in a scale ratio consistent with test purposes as listed in Section CB-3340 of Section III, Division 2 of Ref. 1.

Model testing requirements also are noted in the French and British codes. The French Code[146] requires that each vessel design be subjected to the construction and testing of at least one representative prestressed concrete model geometrically similar to the structure with principal dimensions in a ratio at least equal to 1:6. The British Code[147] provides that the validity and accuracy of any method or computer program shall be demonstrated using known solutions, and, if necessary, they shall be checked against measurements made on models or previously completed vessels in order to verify the analysis method or computer program. Table 14[183] provides a summary listing of most of the PCRV-related model tests that have been conducted. Also included in the table are the type of test, scale, and investigating agency. Summarized below are results of several investigations that have involved the testing of models that included elevated-temperature conditions.

6.2.1 Single-Cavity PCRV Model Tests

Electricité de France.[184] Three 1:6-scale models of EDF3 (Chinon III), such as shown in Fig. 147, were tested. The first two models were identical except the first did not have a gas-tight liner. The third model was used for thermal experiments to determine the influence of relatively high temperatures, the interaction of the concrete and liner following an insulation fault, and the effects of elevated temperature on the loads of the most exposed tendons. Conclusions derived from these tests were that (1) loss of prestressing force under temperature effects was due to steel relaxation and differential expansion between the steel and concrete, (2) drying shrinkage resulted at relatively low hot-wall temperatures (80°C) and was irreversible on cooling, (3) the liner can become highly compressed locally due to the presence of a liner defect (or constraint), (4) application of a second thermal cycle did not result in increased shrinkage beyond that experienced from the first thermal cycle, and (5) tests at temperatures up to 143°C indicated an increase in the coefficient of thermal expansion.

Tests at ambient temperature (to determine the effects of prestressing and of pressure) and under unusual thermal conditions were undertaken on the 1:5-scale model of EDF4 (St. Laurent I) shown in Fig. 148. During an increase in temperature, cracks occurred in the outer walls of the model. Measurements showed that the concrete coefficient of thermal expansion was considerably greater than that assumed in the design calculations and was related to the moisture condition of the concrete.

Table 14 Summary of PCRV Model Tests

Organization	Test Item	Scale	Project	Number of Models	Test for*
1. French AEC	Head, PCRV	Not Known	G-2, G-3	2	A, B, C
	Cylindrical PCRV	1/10	G-2, G-3	3	A, B, C
	Cylindrical Vessels	I.D. 0.76 m I.H. 2.29 m	Safety Studies	25	C, D
	Cylindrical Vessel	Unavailable	G-2, G-3	2	A, B, C
2. Societe d'Etudes et d'Equipments d'Enterprises (SEEE), France	Cylindrical PCRV	1/6	EDF-3	3	A, B, C, D
	Cylindrical PCRV	1/10	EDF-3	1	T
	Cylindrical PCRV	1/5	EDF-4	2	A, B, C, T
	"Hot-Liner" Vessel	Not Known	General	1	A, B, C, T
3. Electricite de France (EDF), France	Cylindrical PCRV	1/5	Bugey I	2	A, B, C, T
	2 Layer Cylinder	1/3	General	1	
4. Central Electric Research Laboratory, England	Cylindrical PCRV	1/8	Oldbury	1	A, B, C, T
	Cylindrical PCRV	1/8	Pre-Oldbury	1	B, C

Table 14 (continued)

Organization	Test Item	Scale	Project	Number of Models	Test for*
5. Sir Robert McAlpine & Sons, England	Cylindrical PCRV	1/7	Prelim, PCRV	1	A, B, C, D
	Heads, PCRV	1/50	General	11	B, C
			Oldbury AGR	2	B, C
	Cylindrical PCRV	1/8	Oldbury	1	A, B, C, T
	Heads, PCRV	1/50	Hinkley Point B	5	B, C
	Cylindrical PCRV	1/10	Hinkley Pont B	1	A, B, C
	Heads PCRV	1/40	HTR	15	B, C
	Multicavity PCRV	1/40	HTR	3	B, C
	Multicavity PCRV	1/14	HTR Oldbury B	1	A, B, C, D
	Heads, PCRV	1/40	Modified AGR	2	B, C
	Heads, PCRV	1/22	Modified AGR	1	B, C
	Heads, PCRV	1/50, 1/30	LMFBR	11	B, C
6. Taylor Woodrow Constr. Ltd. (TWC), England	Spherical PCRV	1/12, 1/40	Wylfa	2	A, B, C
	Cylindrical PCRV	Not Known	Wylfa	3	A, B, C
	Cylindrical PCRV	1/10	Hunteston B	1	A, B
	Heads, PCRV	1/24	Several	12	A, B, C
	Multicavity PCRV	1/10	Hartlepool	1	A, B, C
	Head, PCRV	1/13	Ft. St. Vrain	2	A, B, C, D
	Multicavity PCRV	1/30	GT-HTGR	2	A, B, C
	Cylindrical PCRV	1/40	Future HTGR	4	A, B, C
	Heads, PCRV	1/26, 1/8	Future HTGR	2	A, B, C
	Heads, PCRV	1/8	Future HTGR	2	A, B, C, T
	Boiler Closures	1/10	Hartlepool/Heysham	60	A, B, C
	Boiler Closures	1/3	Hartelpool/Heysham	4	A, B, C, T
	Restrained Concrete Elements	Not to Scale	General	500	A, C
	Closure Plug	1/18, 1/5	Pressure Vessel	45	A, B, C
	Head	1/30, 1/5	Pressure Vessel	77	A, B, C

Table 14 (continued)

Organization	Test Item	Scale	Project	Number of Models	Test for*
7. UKAEA/Imperial College, UK	Spherical PCRV	1/12	General	1	A, B, C, T
	Cylindrical PCRV (domical heads)	1/6	General		A, B, C
8. Kier Ltd., England	Spherical PCRV	1/12	Wylfa	1	A, B, C, T
9. Atomic Power Const., England	Cylindrical PCRV	1/10	Dungeness B	1	A, B, C
	Cylindrical PCRV	1/26	Dungeness B	1	B, C
	Heads, PCRV	1/72	Dungeness B	1	B, C
	Heads, PCRV	1/24	Dungeness B	3	B, C
	Heads, PCRV	1/26	Dungeness B	2	B, C
10. UKAEA, Foulness, England	Cylindrical PCRV	1/20	Safety Studies	10	C, D
11. Building Research Station, England	Cylindrical PCRV	1/10	Hinkley Point B	1	T
	Cylindrical PCRV	1/20	Hinkley Point B	4	T
12. General Atomic	Cylindrical PCRV	1/4	General	1	A, B, C
	Cylindrical PCRV	1/4	Ft. St. Vrain	1	A, B, C, D
	Multicavity PCRV	1/20	HTGR	1	A, B, C

Table 14 (continued)

Organization	Test Item	Scale	Project	Number of Models	Test for*
13. Oak Ridge National Lab.	Cylindrical PCRV	<1/5	General	4	A, B, C
	Wall, PCRV	1/6	General	1	A, T
	Closure, Steam Generator Cavity	1/15	GCFR	2	A, B, C
	Closure, Core Cavity	1/20	GCFR	1	A, B, C
	Head, PCRV	~1/30	General	2	C, D
14. University of Illinois	Head, PCRV	Not to scale	General	35	C, D
15. University of Sydney, Australia	Head, PCRV	1/20, 1/40	General	23	C, D
16. Siemens, Germany	Cylindrical PCRV (Prefabricated Blocks)	1/3		1	A, B, C
17. Krupp, Germany	Cylindrical PCRV	1/5	Gas-Cooled	1	A, B, C, T
	Head, PCRV	1/20	Reactor	1	A, B, C
18. ENEL/ISMES, Italy	Cylindrical PCRV	1/20	HTGR	4	A, B, C
	Head, PCRV	1/20	HTGR	6	A, B, C
	Cylindrical PCRV	1/10	BWR	2	A, B, C

Table 14 (continued)

	Organization	Test Item	Scale	Project	Number of Models	Test for*
19.	Ohbayashi-Gumi, Japan	Cylindrical PCRV	1/20	HTGR	1	A, B, C
		Multicavity PCRV	1/20		1	A, B, C
20.	Cement and Concrete Inst. Trondheim, Norway	Cylindrical PCRV	1/3.6	Scandinavian PCRV (LWR)	4	A, B, C
21.	A. B. Atomenergi, Studsvik, Sweden	Cylindrical PCRV	1/3.5	Scandinavian PCRV (LWR)	1	A, B, T
22.	Shimizu Const. Ltd.,	Cylindrical PCRV	1/10	Hinkley Point B	1	A, B, C
			1/40	AGR	6	A, B, C
			1/40	General	1	T
		Multicavity PCRV	1/30	HTGR	3	A, B, C
		Head, PCRV		General	4	T
23.	Nuclear Power Development Lab & Kashmi Kenetsu, K.K.	Cylindrical PCRV	1/20		3	A, B, T
24.	PCPV Research & Development Group Kajima Corp.	Cylindrical PCRV		ORNL Model	3	A, B, C, T
		Multicavity PCRV	1/20	GA 1100 MW(e)	2	A, B, C

Table 14 (continued)

	Organization	Test Item	Scale	Project	Number of Models	Test for*
25.	Takenaka, Technical Research Laboratory	Head, PCRV	1/20	General	14	A, B, C
26.	Central Research Institute of Electric Power Industry, Chiba, Japan	Cylindrical PCRV	1/10	HTGR	1	A, B, C, T
27.	Danish Atomic Energy Commission, Riso	Closure, PCRV	1/11	Scandinavian PCRV	8	A, B, C
28.	Austrian Research Center, Seibersdorf	Cylindrical PCRV		HTR	1	A, T
29.	Swiss Institute of Technology (CEBAP), Lausanne	Multiactivity PCRV	1/20	HHT	1	A, B, C
30.	KFA	PCRV Wall Segment	Full-section	HTR-500 THTR-300	3	A, D, T

*A - elastic response, B - design overpressure, C - failure, D - abnormal conditions, T - long-term creep and temperature.

Source: D. J. Naus, *A Review of Prestressed Concrete Reactor Vessel Related Structural Model Tests*, ORNL/GCR-80/10, Oak Ridge National Laboratory, Oak Ridge, Tennessee, 1980.

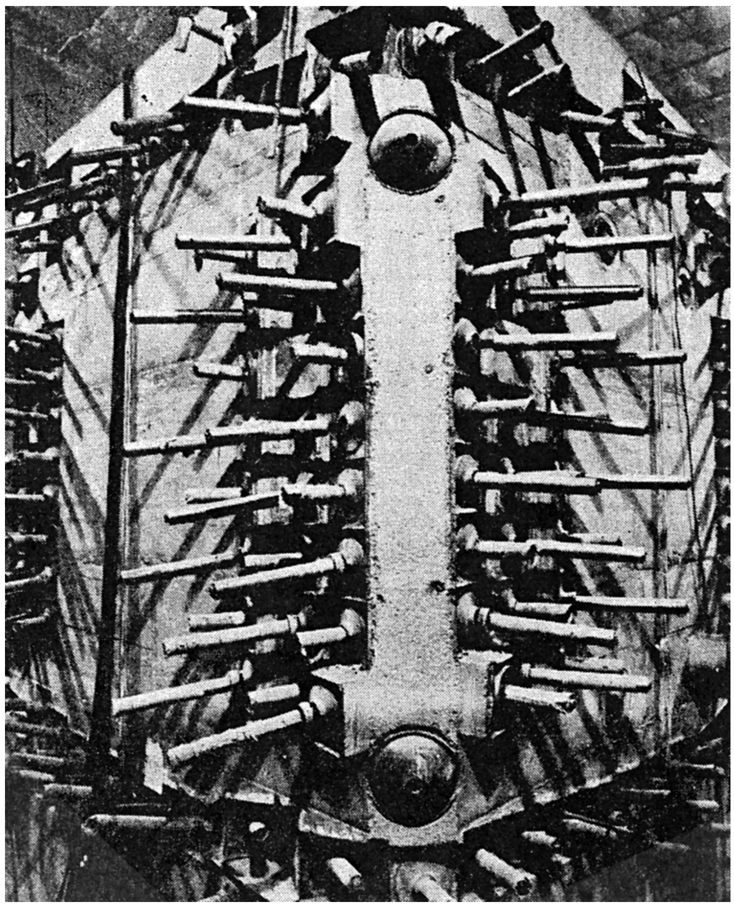


Figure 147 **EDF3 1:6-scale PCRV model.** *Source:* M. Lida and R. Ausangee, “Scale Models for Strength Testing Nuclear Pressure Vessels,” Group G, Paper 44, *Prestressed Concrete Pressure Vessels*, Institution of Civil Engineers, London, pp. 497–505, 1968.

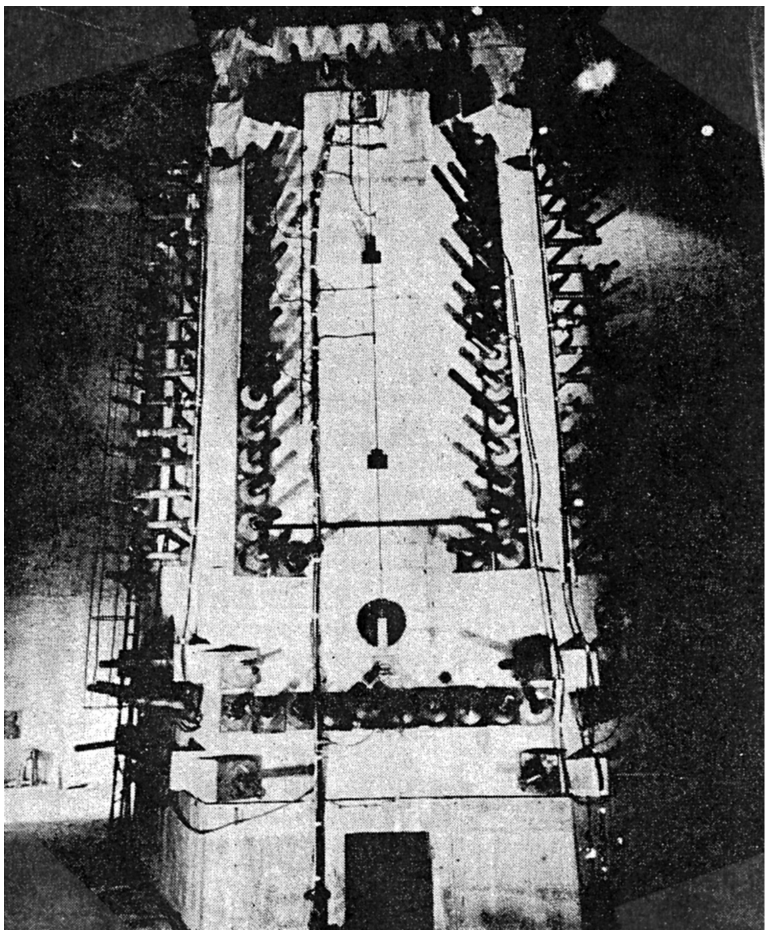


Figure 148 **EDF4 1:5-scale PCRV model.** *Source:* N. M. Lida and R. Ausangee, “Scale Models for Strength Testing Nuclear Pressure Vessels,” Group G, Paper 44, *Prestressed Concrete Pressure Vessels*, Institution of Civil Engineers, London, pp. 497–505, 1968.

Sir Robert McAlpine and Sons Ltd. (United Kingdom).[185,186] The 1:8-scale model of a cylindrical PCRV shown in Fig. 149 was tested under prestress and various combinations of internal pressure and thermal loading. The test program covered 4 years and involved 5 series of tests: (1) hydraulic—up to 4.42 MPa at ambient temperature, up to 2.76 MPa at 50°C, and up to 1.79 MPa at 90°C; (2) five tests at elevated temperature under zero pressure with liner and gas ducts heated to 172°C and various fault conditions simulated by heating selected areas of the liner; (3) approximately two-thirds of top slab tendons were detensioned and 60 pressure cycles to 1.93 MPa applied at ambient and 94.5°C; (4) four 162–mm-diameter holes were placed in upper slab to simulate boiler loading holes and five tests up to 3.45 MPa were conducted at ambient temperature with half the top slab tendons tensioned; and (5) all tendons were removed from top slab and the model hydraulically pressurized at ambient temperature until failure (test was terminated at 3 times design pressure when top slab had lifted at inside edge of helical anchorage thus preventing further pressurization). It was concluded that the analysis methods were sufficiently conservative to enable them to be adopted as a design tool, the method of ultimate analysis used made a good assessment of the ultimate pressure and accurately predicted the mode of failure, cycling the load at ambient and elevated temperatures did not adversely affect elastic behavior, fault condition temperatures did not adversely affect the elastic behavior, neither the standpipe systems designed on a modular replacement basis nor the large carbon dioxide ducts caused any excessive or unexpected deflections or stresses to be set up in the concrete, and it was shown to be entirely satisfactory to stress the end slabs of a cylindrical PCPV using only a helical cable system.

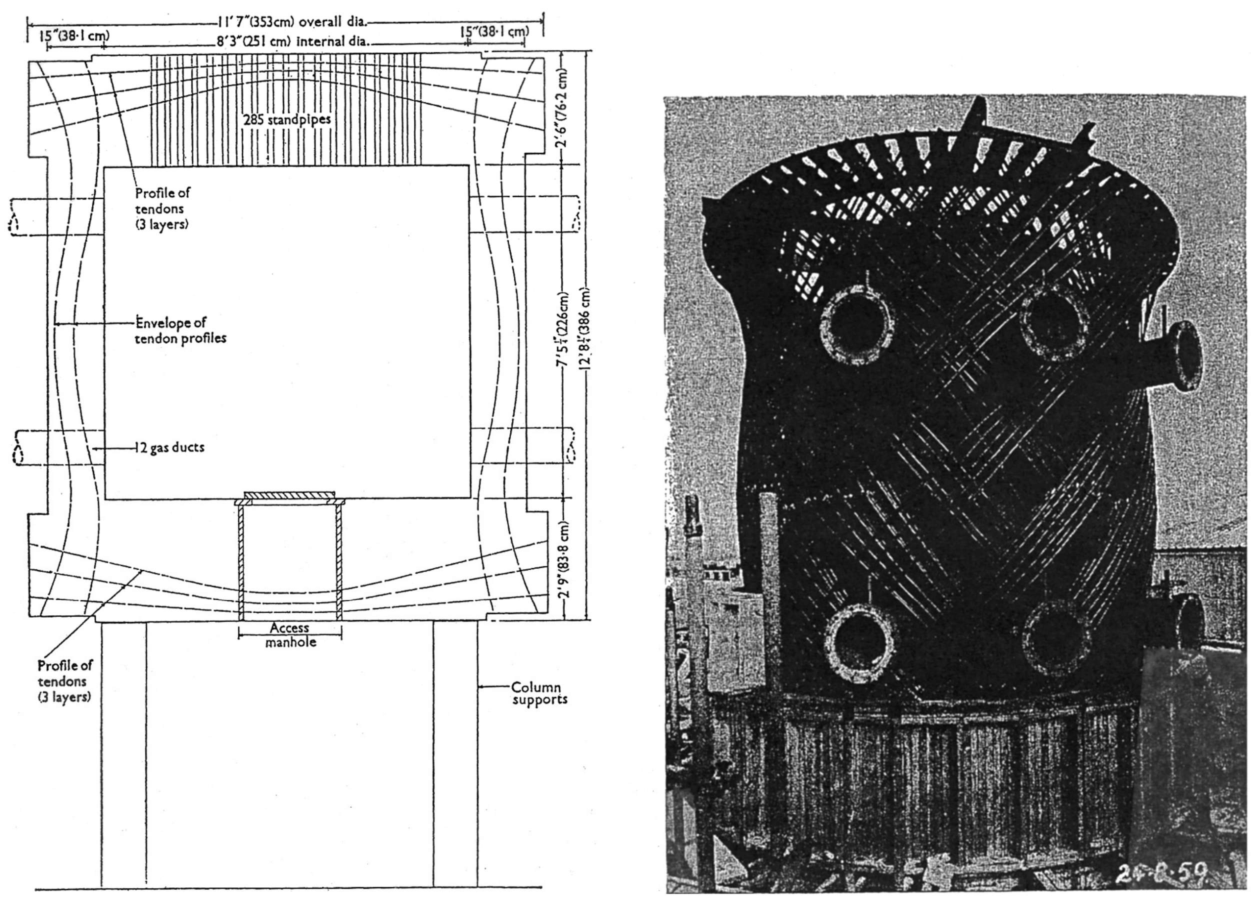


Figure 149 **1:8-scale cylindrical PCRV model.** *Source*: D. C. Price and M. S. Hinley, "Testing a l/8th Scale Cylindrical Vessel," Group G, Paper 43, *Prestressed Concrete Pressure Vessels,* Institution of Civil Engineers, London, pp. 489–496, 1968.

Kier Ltd. (United Kingdom).[187] The 1:12-scale ribbed spherical vessel shown in Fig. 150 was subjected to pressure and temperature loadings to investigate (1) elastic response to temperature and pressure loading prior to cracking. (2) cracking in a vessel that was largely unaffected by differential creep or shrinkage, and (3) the effect of aging on vessel performance. Under pressure testing, the model behavior was elastic to 1.72 MPa, and at pressures above 2.76 MPa, deflections increased rapidly with pressure. At 3.79 MPa the vessel liner failed, and the test was stopped so that the liner could be repaired. Upon repressurization, the liner again failed at 4.34 MPa. Vessel ultimate strength was then calculated to be 4.48 MPa. Temperature tests were conducted with an initial gradient of 24°C in order to avoid cracking and provide data for analysis comparisons. Long-term temperature tests were then conducted for a period of approximately 9 weeks in which the crossfall was increased in three equal stages to 36°C. During this test sequence, an internal pressure of 2.14 MPa was applied from day 40 to day 47. A temperature crossfall of 84°C was then imposed on the vessel while under a 2.14-MPa internal pressure to simulate a severe overload temperature. No extensive new cracking occurred as a result of this test, and the ability of the vessel to withstand severe temperature loading without great distress was demonstrated.

General Atomic Company (USA).[188] A 1:4-scale model of the PCRV for the Fort St. Vrain plant was fabricated and tested to meet the following objectives: (1) determine construction problems associated with use of preplaced aggregate, job-mixed concrete, liner installation, penetrations, and prestressing

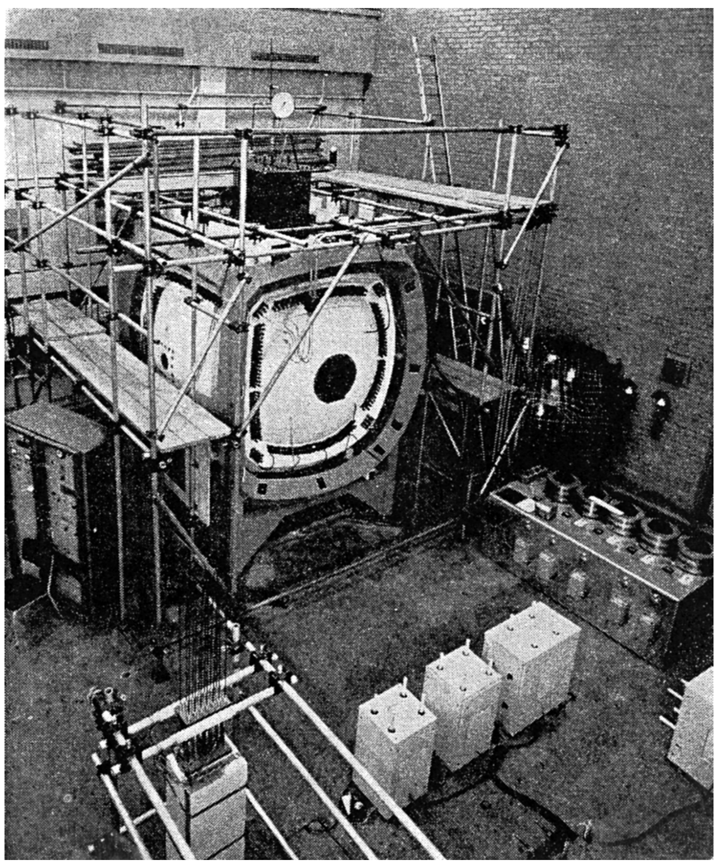


Figure 150 1:12-scale ribbed spherical pressure vessel model. *Source*: M. L. A. Moncrieff, "Comparison of Theoretical and Experimental Results for a Ribbed Spherical Vessel," Group G, Paper 42, pp. 469–479, *Prestressed Concrete Pressure Vessels*, Institution of Civil Engineers, London, 1968.

system; (2) evaluate strain and deformation response resulting from pressure, temperature, prestressing, and the combination of these forces; (3) qualitative determination of moisture loss rate; (4) observe effects of pressure cycles; (5) investigate time-temperature dependent concrete behavior; (6) evaluate gross gas leakage from a faulted liner; and (7) determine vessel response under overpressure loads. The model shown in Fig. 151 was subjected to a series of tests extending over a period greater than 2 years. Included in the test history were tests to demonstrate that the structural response of the vessel to short-term loadings up to reference pressure (4.86 MPa) was elastic, evaluate vessel performance with a constant temperature gradient of 27.8°C across the walls, and demonstrate the ability of the vessel to withstand overpressures up to 2.13 times the reference pressure without structural failure. An additional series of tests was conducted to demonstrate vessel behavior under abnormal and accident conditions (pneumatic overpressure to 1.6 times reference pressure, gas permeation tests, gas release tests, and tendon detensioning tests). Results obtained from the tests showed that the vessel response was linear up to 1.5 times reference pressure, response of vessel pressurization at temperature was not significantly different from the response at ambient when shrinkage cracks alone were present, creep rate during conditions of residual prestress and elevated temperature was lower than or equal to the measured rate of creep under prestress and ambient temperature and the creep rate in the model was less than that for reference cylinder specimens, and during overpressure tests up to 2.13 times reference pressure (2.61 times normal working pressure) no structural distress was noted although some surface cracking was

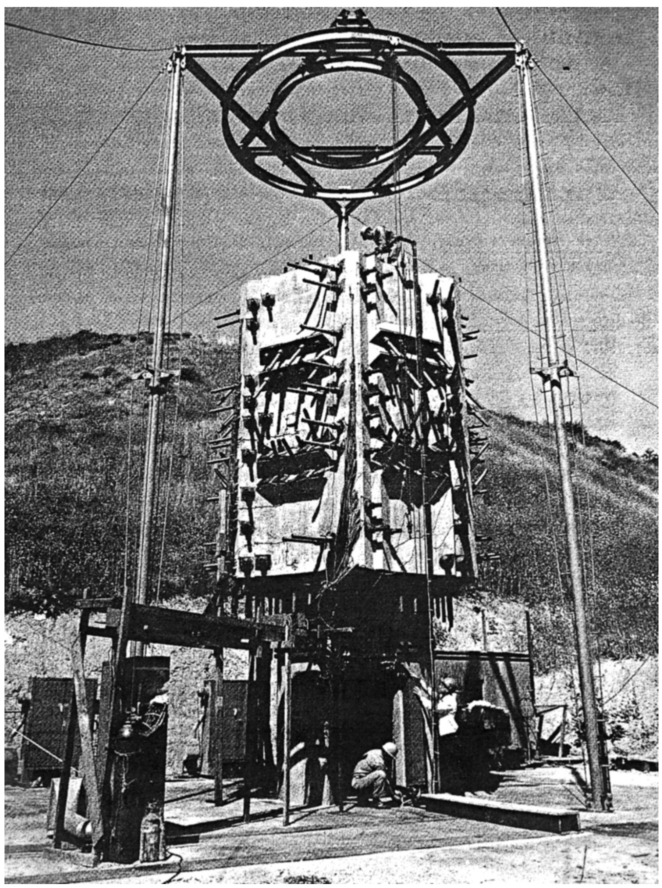


Figure 151 **1:4 scale Fort St. Vrain PCRV model.** *Source*: T. E. Northup, "Pressure and Temperature Tests and Evaluation of a Model Prestressed Concrete Pressure Vessel," GA-9673, General Atomic Co., September 15, 1969.

noted in the middle third portion of the barrel. Vessel response during the sustained prestress, transient and steady-state temperature distributions, short-time and sustained pressures, and pressure overload was calculated using a method of analysis that accounted for concrete creep, cable relaxation, cracking, and steel yielding. Results indicated that the analysis, which was based on a nonlinear superposition principle and a two-dimensional solution, agreed well with experimental results.

Austrian Research Center (Seibersdorf).[189,190] A large-scale model PCRV having a hot liner and adjustable wall temperature was constructed for use as a pressure vessel of the high temperature helium rig for the testing of high temperature reactor components. The 12-m-high by 2.6-m-diameter (1.5-m inner diameter) cylindrical vessel, as shown in Fig. 152, was designed to operate at a pressure of 10.0 MPa, a liner temperature of 300°C, and a concrete temperature of 120°C. The vessel wall section consisted of four functional parts (Fig. 153): the liner (5-mm-thick with anchor bolts), the insulating concrete, the structural concrete, and the prestressing system. Tubes that circulate nitrogen were used to control the temperature distribution in the wall. Thermal stabilization and pressure tests have been conducted on the vessel. During the first thermal cycle the vessel was carefully heated to 120°C and kept at this temperature for 100 d for thermal stabilization. During this period there initially was an increase and acceleration of viscoelastic strains and loss of prestress, but as the test period neared completion these changes had stopped and the values stabilized. Assumptions with respect to behavior and that large-scale concrete structures could operate for a prolonged period at temperatures above 100°C were verified. Prestress loss caused by creep and shrinkage of concrete was compensated for by retensioning. A pressure test to 1.15 times the operating pressure was conducted with measurements made to determine the liner and insulating concrete strains as well as the overall vessel geometric stability and tendon prestress. These measurements were noted to be in full agreement with the structural analysis that had been conducted previously. The next step was drying and stabilizing the insulating concrete at 140°C. The first test cycle with 150°C liner temperature and 80°C concrete temperature with a 50-bar internal pressure followed. Two 150°C cycles were executed followed by an increase in the liner temperature up to 200°C. In the fifth cycle, full load was applied with 300°C liner temperature, 120°C concrete temperature, and 95-bar internal pressure. Results obtained indicate that it is possible to operate a hot vessel in a stable state after a stabilization treatment is applied.

6.2.2 End Slab Model Tests

Imperial College (United Kingdom).[191] A study was conducted to investigate the behavior of unperforated and perforated circular plates with reinforced holes when subjected to radial in-plane loading and sustained uniform temperature. Two series of five specimens each, as shown in Fig. 154(a), were tested in the test rig shown in Fig. 154(b) at a test temperature of 80°C. Strains, temperatures, and loads were obtained during the tests so that creep, elastic, thermal, and shrinkage strains as well as internal stress and strain redistributions could be determined. Conclusions reached were that stresses around the perforated zones decrease as a result of differences in the rate of creep between the perforated and unperforated zone concretes, and the thermal stresses due to restrained thermal expansion on application of heat are reduced gradually as a result of thermal creep causing a redistribution of applied load stresses.

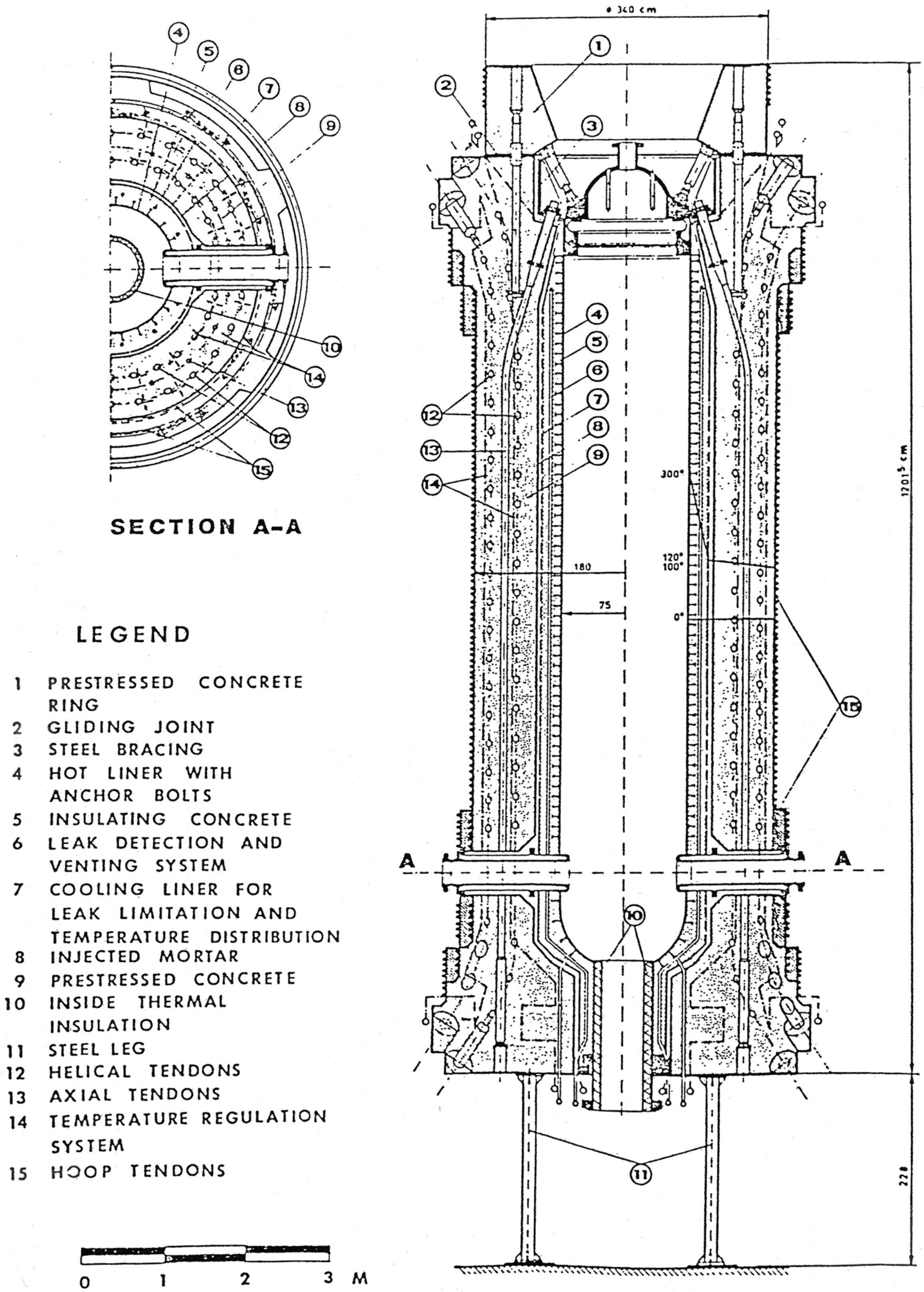


Figure 152 **Austrian large PCRV model with hot liner.** *Source*: J. Nemet et al., "Testing of a Prestressed Concrete Pressure Vessel with Hot Liner," Report SBB/He-3E, Reaktorbau Forschungs-und Baugesellschalft, Seibersdorf, Austria, November 1977.

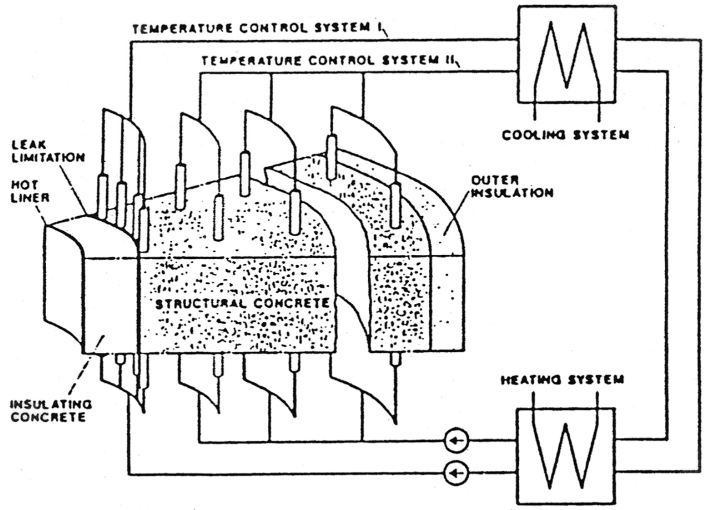
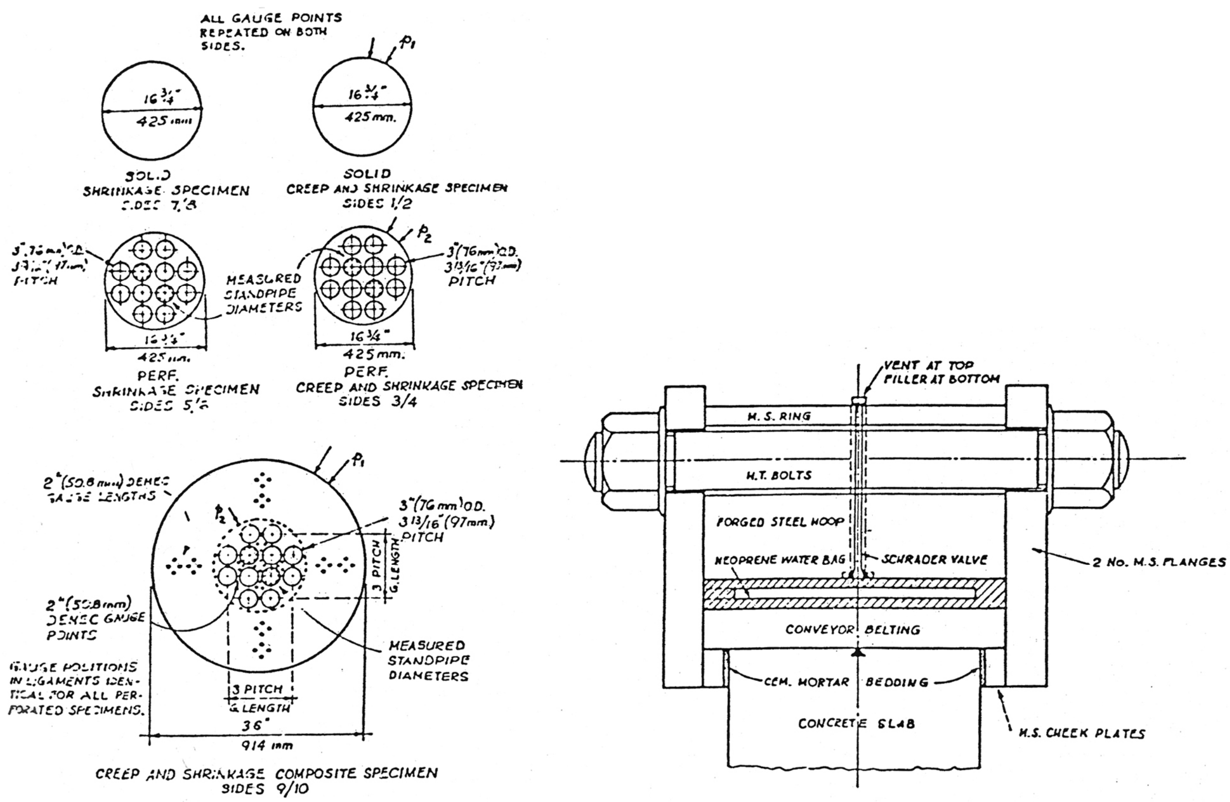


Figure 153 Section through vessel wall of Austrian PCRV model showing hot liner and adjustable wall temperature system. *Source*: J. Nemet et al., "Testing of a Prestressed Concrete Pressure Vessel with Hot Liner," Report SBB/He-3E, Reaktorbau Forschungs -und Baugesellschalft, Seibersdorf, Austria, November 1977.



a. End slab models. b. End slab model test setup.

Figure 154 Unperforated and perforated plate tests. *Source*: G. D. Stefanou et al., "An Experimental Investigation Into the Behaviour of Perforated End Slabs for Concrete Pressure Vessels Under Temperature and External Load," Paper 8, *Model Techniques for Prestressed Concrete Pressure Vessels*, The British Nuclear Energy Society, London, 1968.

6.2.3 Thermal and Moisture Migration Model Tests

Building Research Station (United Kingdom).[192] An investigation was conducted to provide information on model techniques applied to temperature loading on massive concrete structures. The primary concrete shields at Hinkley Point A nuclear power station were used as the prototype. Repeated tests were made to compare, during alternations between uniform temperature and the required temperature distribution, the behaviors of models of two different geometric scales with each other and with analysis results. Four 1:20-scale and one 1:10-scale (Fig. 155) models were tested. In addition to thermal loadings, one of the

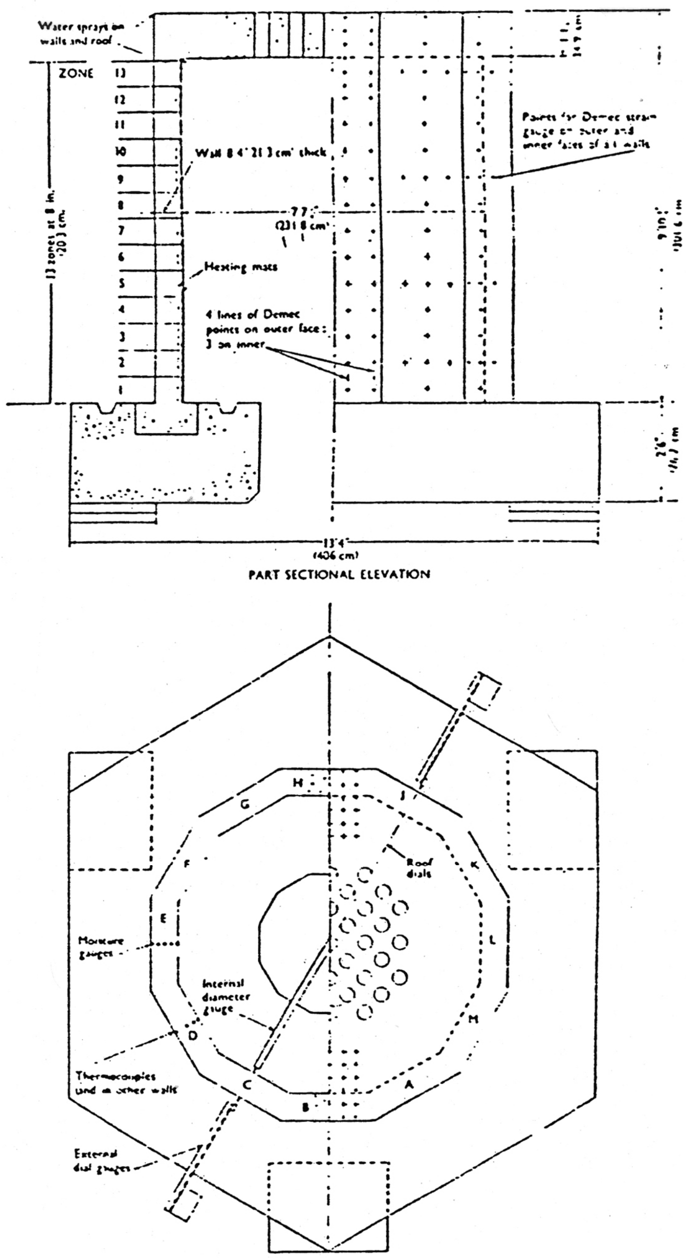


Figure 155 1:10-scale Hinkley Pt. A primary shield model. *Source*: C. R. Lee et al., "Behaviour of Model Concrete Structures Under Temperature Loading," Group G, Paper 46, *Prestressed Concrete Pressure Vessels*, Institution of Civil Engineers, London, pp. 517–525, 1968.

1:20-scale models was tested while under external mechanical loadings (Fig. 156). Models 1 and 2 were used to develop test techniques while the remaining three models were used for the main investigation. The majority of tests were conducted with the models subjected to superficial water sprays to maintain a saturated condition to give better stability and a better simulation of practical conditions than would have been obtained by permitting the concrete to dry. Measurements obtained during testing included temperatures, internal and external wall deflections, and vertical and horizontal strains of the inner and outer surfaces of the walls and roof. Observations from the tests were that the rate of drying was potentially much greater in the model than prototype, short-term temperature tests were insensitive to changes in the rate of heating, reasonably good agreement was noted between experiment and theory, and model techniques can be satisfactorily applied to short-term temperature loadings of massive concrete structures within normal operating conditions.

<u>Central Electricity Research Laboratories (United Kingdom)</u>.[193,194] A 1:8-scale model of the PCRV for Oldbury was investigated (Fig. 157). The thermal testing was conducted in four phases: (1) preliminary thermal cycle of 17 d (2 d required to obtain desired inner and outer temperatures of 55°C and 29°C,

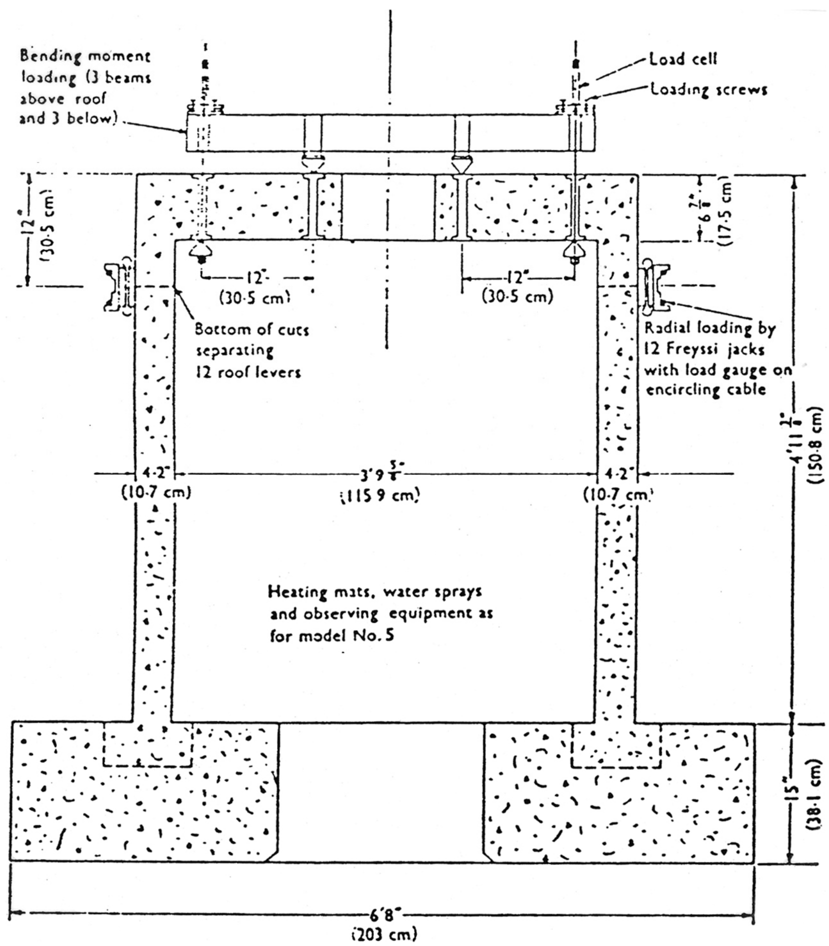


Figure 156 1:20-scale Hinkley Pt. model mechanical load system setup.
Source: C. R. Lee et al., "Behaviour of Model Concrete Structures Under Temperature Loading," Group G, Paper 46, *Prestressed Concrete Pressure Vessels*, Institution of Civil Engineers, London, pp. 517–525, 1968.

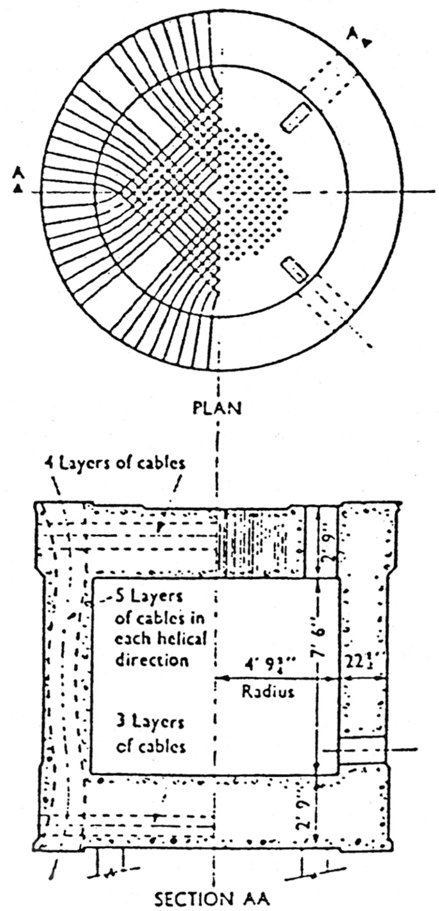


Figure 157 **1:8-scale Oldbury PCRV Model.** *Source*: I. W. Hornby, "The Behaviour of the Oldbury Model Vessel with Time Under Thermal and Pressure Loadings," Paper No. 11, *Model Techniques for Prestressed Concrete Pressure Vessels,* The British Nuclear Energy Society, London, 1968.

respectively, followed by 15 d of cooling); (2) main thermal cycle of 2 d heatup followed by 5 d of cooling; (3) superposition of pressure (2.65 MPa) onto the thermal loading (vessel pressurized, heated for 2 d, allowed to cool for 13 d, and then depressurized); and (4) same sequence as the third phase except the temperature and pressure remained 61 d followed by 16 d of cooling prior to depressurization. It was noted in the tests that several factors were to be considered in determining total strain changes: the coefficient of thermal expansion for the second and subsequent loading cycles was approximately 20% less than the initial value, so the residual thermal expansion from the first thermal cycle must be considered in subsequent loading cycles; creep strain due to a temperature increase was not understood sufficiently; creep-produced stress redistributions were neglected for long periods of loading; the variation of creep recovery with stress decrease was nonlinear; and shrinkage was neglected in the tests, but large shrinkage strains did not occur prior to the test. It was concluded that the response of the model could be predicted during the thermal and creep tests, but basic information relative to creep of concrete subjected to variable stress, temperature, and moisture content was required for estimating (modeling) long-term performance.

During commissioning tests of Oldbury, there were a small number of localized breakdowns of the liner insulation permitting the temperature to reach 180°C in the head penetration region and 90°C in the haunch region at the upper boiler instrument penetration, which could have induced cracking in the concrete. To provide input on concrete cracking, a full-scale model (3.66 m by 1.52 m thick) of the region of the vessel local to the upper boiler instrumentation where the highest liner temperatures were recorded was fabricated and tested (Fig. 158). The test procedure included heating of the model over a 24-h period to the steady-state condition achieved in the hot spot region of the prototype, allowing the model to attain thermal equilibrium, and maintaining this condition for 3 months with the prestressing force reduced as the test progressed, permitting the model to cool to ambient, and injection of a dye between the liner and concrete to denote cracking. Core samples that were taken to determine concrete strength and to locate internal cracking revealed cracking parallel to the liner at the level of the cooling tubes. This indicated that cracking probably had occurred in the prototype vessel near penetrations at hot spots over 100°C, but the cracking was limited to the immediate vicinity of the hot spot, and the effectiveness of the liner

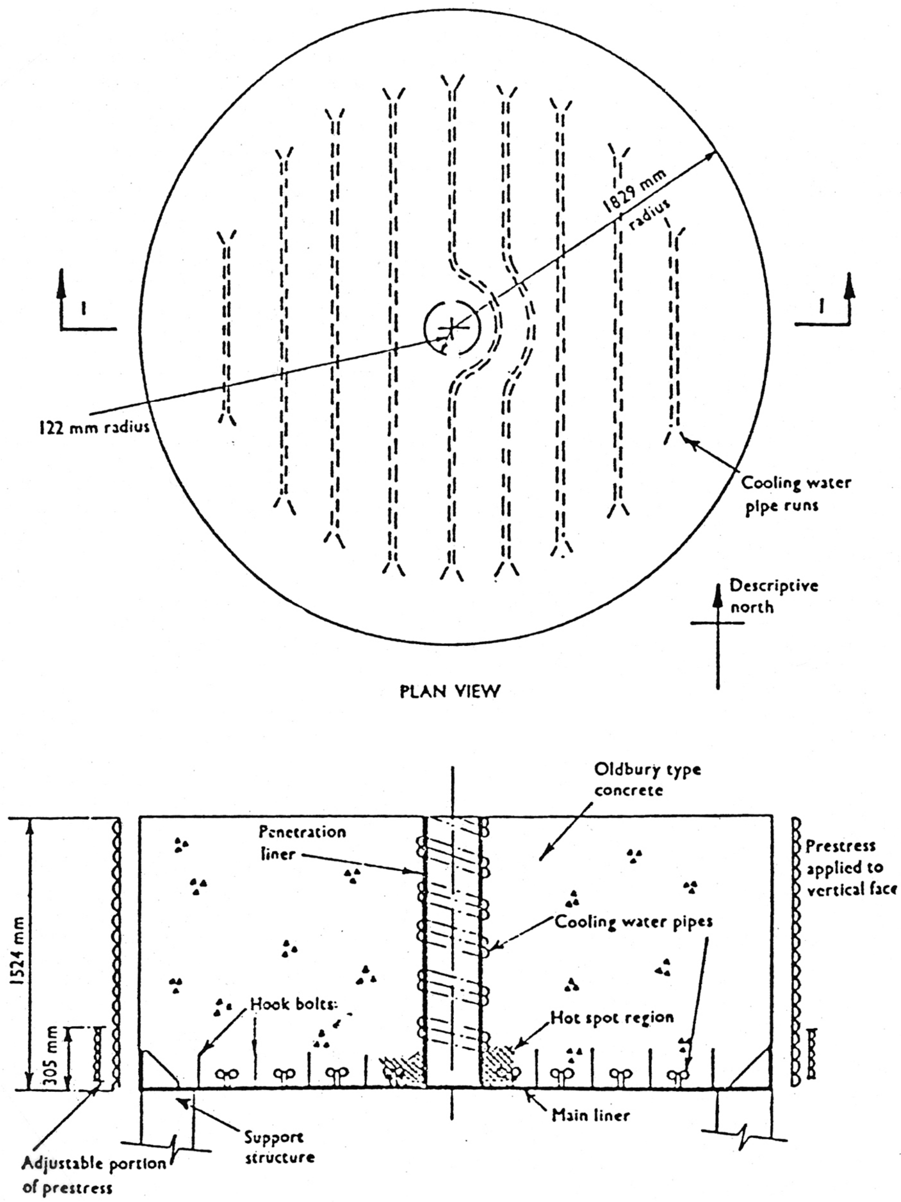


Figure 158 **Full-scale Oldbury hot-spot model.** *Source*: J. Irving et al., "A Full Scale Model Test of Hot Spots in the Prestressed Vessels of Oldbury Nuclear Power Station," Paper 7699, *Proc. Instn. Civil Engineers* **57**, June 1974.

anchorages was not jeopardized. Results also showed that there was not significant loss of strength in uncracked regions of the model where cooling tubes provided heat removal functions, and that the cracks were restricted to localized hot spots around penetrations.

Compagnie Industrialle le Travaux (Paris).[195] Two 1:5-scale models of the Bugey PCRV were constructed (Fig. 159). The first model was to determine rupture strength, and the second was for more detailed tests such as thermal tests. The model had an outside diameter of 5.5 m, a wall thickness of 1.1 m, a height of 10.7 m, and an end slab thickness of 1.4 m. Thermal tests were conducted on the second model according to the test history presented in Fig. 160(a). These tests were followed by special tests as noted in Fig. 160(b), which included a series of tests in which one, five, or all of the standpipes in the head region (Fig. 161) were heated to temperatures of 80, 100, and 120°C. During these tests buckling of the liner occurred between liner anchorages due to the large compressive strains caused by the thermal gradient. In general, the tests confirmed earlier computer analyses, and only slight modifications in the design of the anchors were required near some of the penetrations.

Centre Experimental de Researches et d'Etudes du Batiment et des Travaus Publics (CEBTP) France.[196] Following satisfactory operation of G2 and G3 in Marcoule, and with difficulties encountered in the construction of steel containments of the type EDF 1 and EDF 2, the French Atomic Energy Commission (CEA) decided to test a simplified model of the EDF 3 type and subject it to thermal cycling tests.

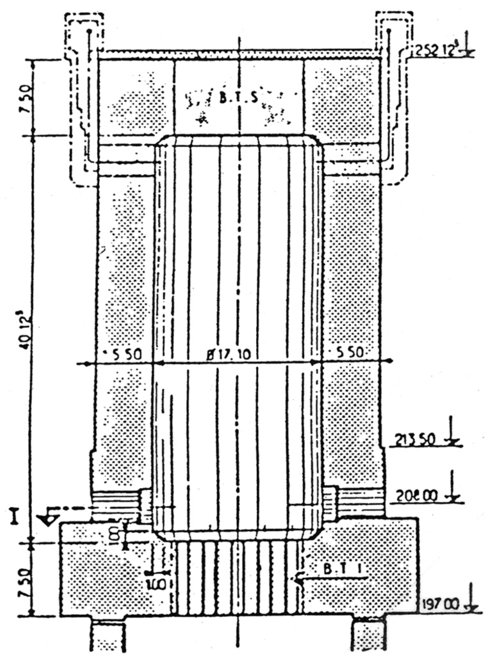
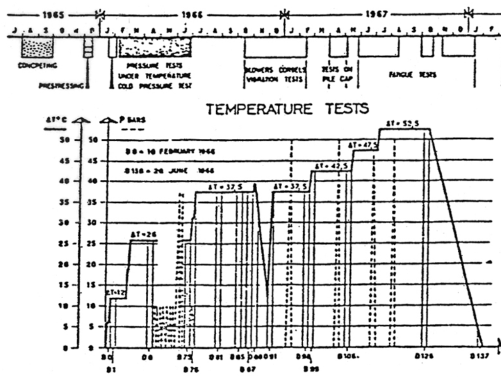
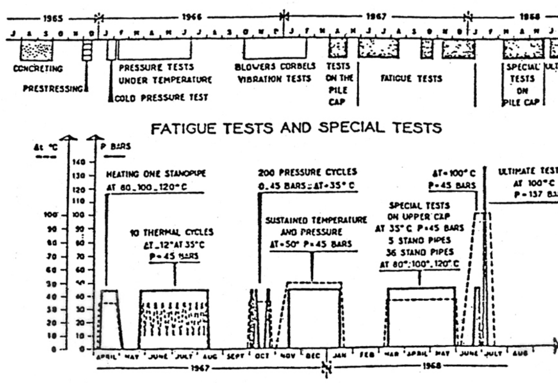


Figure 159 1:5-scale model Bugey PCRV. *Source*: P. Launay, "Apparatus, Instrumentation, and Concrete Models of Bugey I Prestressed Concrete Pressure Vessel," Paper SP-34-69, Session 17, *Concrete for Nuclear Reactors*, Vol. III, American Concrete Institute Special Publication SP-34, Farmington Hills, Michigan, pp. 1529–1566, 1979.



a. Temperature-time history.



b. Special test loading history.

Figure 160 1:5-scale Bugey PCRV model test history.
Source: P. Launay, "Apparatus, Instrumentation, and Concrete Models of Bugey I Prestressed Concrete Pressure Vessel," Paper SP-34-69, Session 17, *Concrete for Nuclear Reactors*, Vol. III, American Concrete Institute Special Publication SP-34, Farmington Hills, Michigan, pp. 1529–1566, 1979.

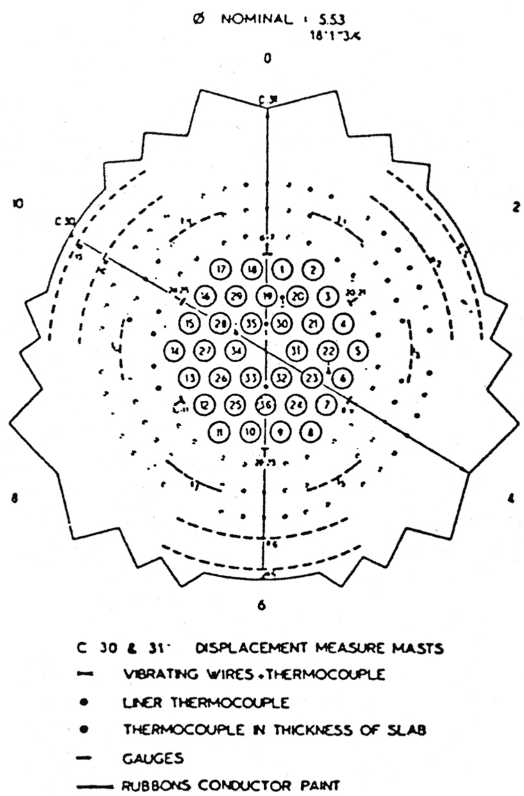


Figure 161 1:5-scale Bugey standpipe region model.
Source: P. Launay, "Apparatus, Instrumentation, and Concrete Models of Bugey I Prestressed Concrete Pressure Vessel," Paper SP-34-69, Session 17, *Concrete for Nuclear Reactors*, Vol. III, American Concrete Institute Special Publication SP-34, Farmington Hills, Michigan, pp. 1529–1566, 1979.

Figure 162 presents a cross section of the model that was prestressed vertically and horizontally. The model was a 1:10-scale version of the prototype except for the height, which was doubled so that the central region could be considered as approximating an infinitely long cylinder. The interior of the model was heated by electrical-resistance heaters, and the exterior was cooled by circulated air. Eight heating cycles were applied to the model over a period of approximately 27 months with maximum temperatures at the inside face of the model being 200°C for cycles 1–7 and 260°C for cycle 8. Temperature distributions were found to be relatively uniform along the 5-m height for a distance of approximately 0.5 m from the ends. Temperatures, strains, and overall deformations were measured during the test. Results obtained showed that calculated temperature distributions were valid for both the steady-state and transient conditions; primary cracking was vertical, forming in the center of the free section between lugs for prestressing anchorage and running the complete model length; secondary horizontal cracks also formed (it was noted that these cracks which were 8-cm long at the end of the second thermal cycle were 12–13 cm at the end of the last cycle and that their width had increased from approximately 0.33 mm to 2.25 mm); water content measurements indicated that the 7 cm of concrete next to the inside surface had dried when the temperature reached 150°C, at a temperature of 175°C the region of drying had reached

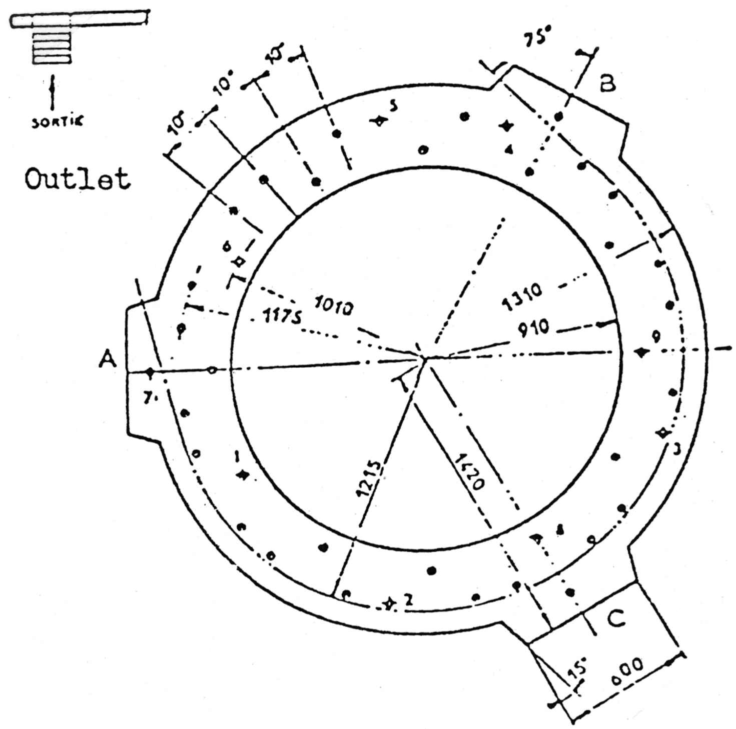


Figure 162 Simplified 1:10-scale EDF3-type model used in thermal cycling tests. *Source*: F. Dubois et al., "Study of a Reduced Scale Model of a Prestressed Concrete Vessel Subjected to a Large Thermal Gradient," Annales de 1'Institut Technique du Batimen't et des Travaux Publics. No. 214, October 1965.

20 cm, and at the end of the second heating cycle, drying had reached 30 cm; and prestressing losses averaged 30% and 20% at the end of all heating cycles for the horizontal and vertical tendons, respectively. Companion test specimens were also tested to determine concrete mechanical properties under the influence of temperature, and in general it was found that there was no significant compressive strength variation for specimens subjected to 150°C for periods of 7–180 d, tensile strengths decreased approximately 12% for 180 d exposure at 150°C, and the weight loss for specimens exposed to 150°C was approximately 4.6% regardless of curing period. It was concluded in the investigation that the safety factor for temperature for these vessels is high and that accidental temperature increases of the vessel can be considered without too much fear for vessel integrity.

Oak Ridge National Laboratory (USA).[197,198] A thermal cylinder experiment was designed both to provide information for evaluating the capability of analytical methods to predict the time-dependent stress-strain behavior of a 1:6-scale model of the barrel section of a single-cavity PCRV and to demonstrate the structural behavior under design and simulated thermal conditions such as could result from an accident. The model shown in Fig. 163 was a thick-walled cylinder having a height of 1.22 m, a thickness of 0.46 m, and an outer diameter of 2.06 m. It was prestressed both axially and circumferentially and subjected to a 4.83-MPa internal pressure together with a thermal crossfall imposed by heating the inner surface to 65.7°C and cooling the outer surface to 24°C. Because the model was designed to study the behavior of the barrel section of a massive concrete structure, all exposed surfaces were sealed to prevent loss of moisture, and the ends of the cylinder were insulated to prevent heat flow in the axial direction. The experiment utilized information developed from previous studies of concrete materials properties, triaxial creep, instrumentation, analyses methods, and structural models. The initial

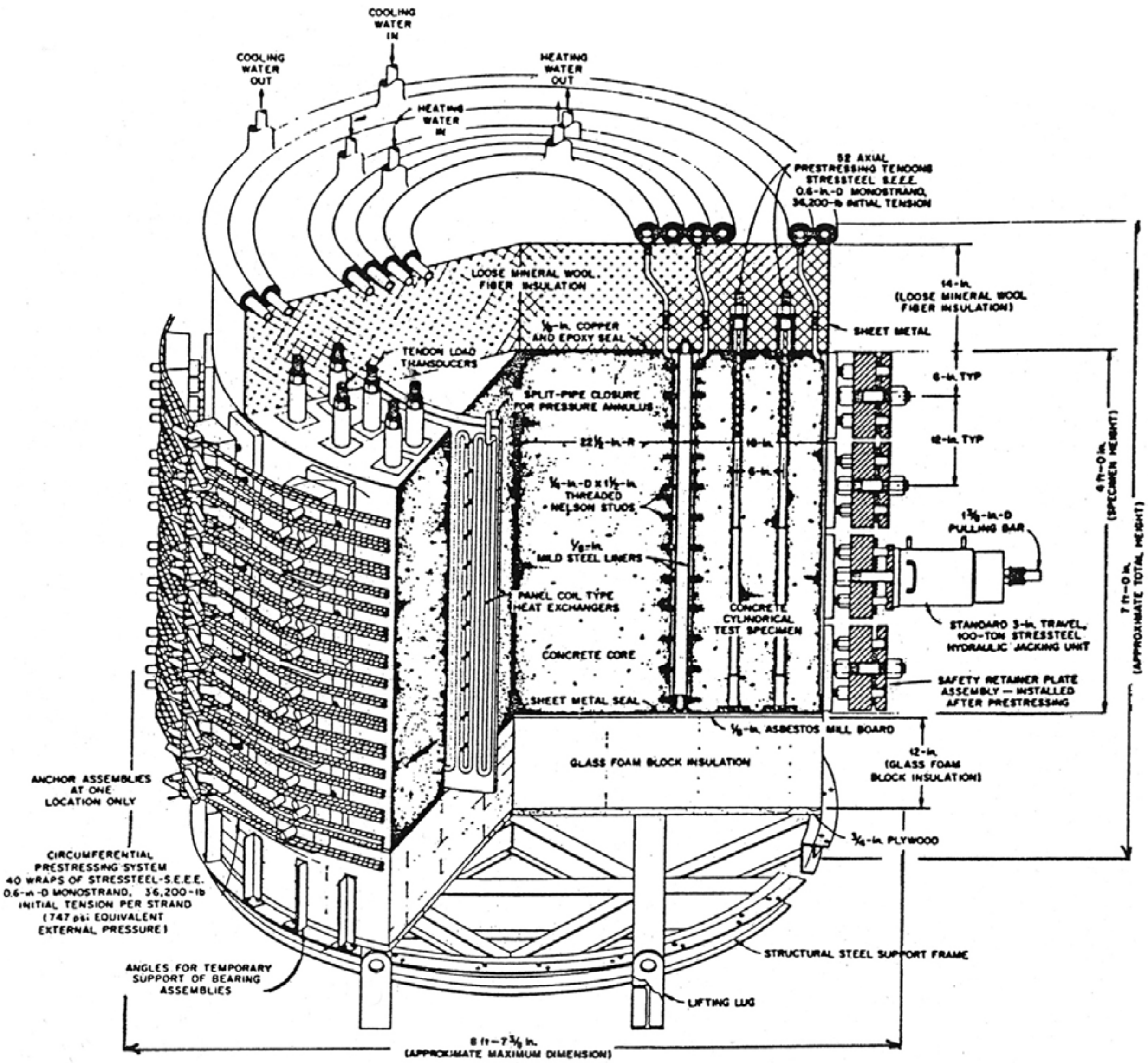


Figure 163 **Isometric of ORNL thermal cylinder test structure.** *Source*: J. J. P. Callahan et al., *Prestressed Concrete Reactor Vessel Thermal Cylinder Model Study*, ORNL/TM-5613, Oak Ridge National Laboratory, June 1977.

460 d of testing were divided into time periods that simulated prestressing, heatup, reactor operation, and shutdown. At the conclusion of the simulated operating period, the model was repressurized and subjected to localized heating at 232°C for 84 d to produce an off-design hot-spot condition. Comparisons of experimental data with calculated values obtained using the SAFE-CRACK finite-element computer program showed that the program was capable of predicting time-dependent behavior in a vessel subjected to normal operating conditions, but that it was unable to accurately predict the behavior during off-design hot-spot heating. Readings made using a neutron and gamma-ray backscattering moisture probe showed little, if any, moisture migration in the concrete cross-section. Destructive examination indicated that the model maintained its basic structural integrity during localized hotspot heating.

In an effort to obtain information regarding the nature of moisture movement and rate of moisture loss in a PCRV, an experimental study of moisture migration in a pie-shaped specimen representing the flow path through a cylindrical wall of a PCRV was conducted. The model was 2.74 m in length with cross-sectional dimensions of 0.61 by 0.61 m on one end and 0.61 by 0.81 m on the other end. It was sealed against moisture loss on the small end (interior) and along lateral surfaces and exposed to the atmosphere at the other end (exterior). A series of heating lamps such as shown in Fig. 164 were used to maintain the required temperature on the simulated interior surface. Temperature distributions, shrinkage, and moisture distribution were monitored for approximately 17 months prior to application of a 44°C temperature gradient that was maintained for one year. At the end of the test, with the exception of zones nearest the ends of the specimen, moisture contents were fairly constant. Concrete strains corrected for thermal effects were small with only 1 m (that nearest open end) indicating shrinkage strain in excess of 20 millionths, implying that drying shrinkage was minimal. It was concluded that moisture migration in thick sections of concrete, such as a PCRV, is a slow process and is not likely to be a significant factor with temperature differences of 44°C or less.

Central Research Institute of Electric Power Industry (Japan).[199] An investigation was conducted to determine the effects of differential thermal creep on the behavior of a PCRV model that was subjected to a long-term temperature gradient across the wall for a duration of 4 months and to investigate the applicability of analytical methods for estimating the time-dependent behavior. The model shown in Fig. 165 was approximately 1:10-scale and was prestressed axially and circumferentially. A lead plate liner was used to seal the inner surface. During the first stage of tests the elastic behavior of the model at prestressing and during an internal design pressure test was investigated. The temperature at the inside surface of the model was then incremented in 10°C steps with the thermal crossfall maintained for 3 weeks at each increment except for the last increment which represented a $\Delta T = 40°C$ where it was maintained for 8 weeks. After thermal creep tests, the model was pressurized to failure which occurred at 3.2 times design pressure. It was concluded that the creep characteristics of the model could be predicted using a strain hardening method as well as the rate of creep if measured values of the concrete creep and thermal properties were incorporated into the analysis, and that significant stress relaxation occurs indicating the necessity of evaluating the thermal stresses in the design with due consideration given to

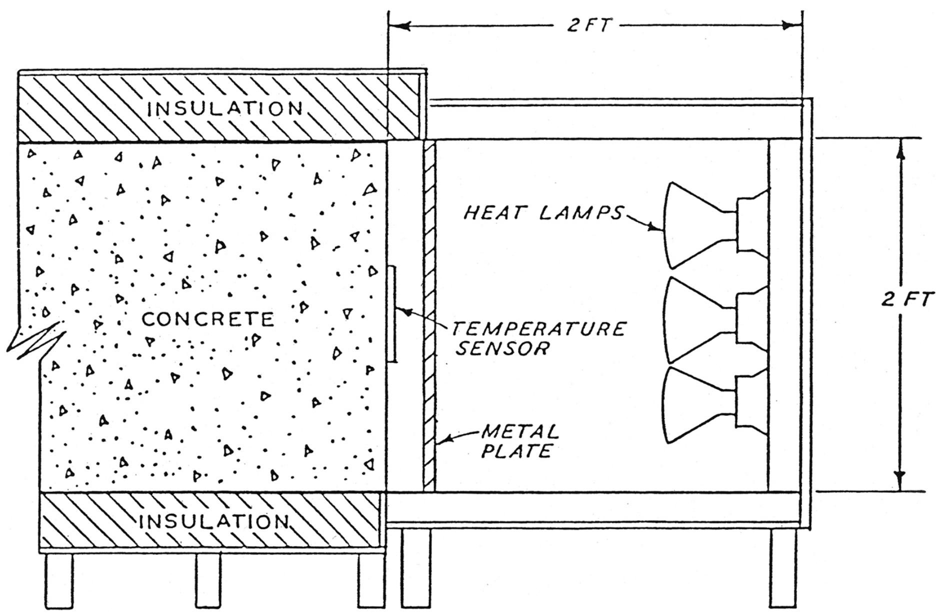


Figure 164 Heating arrangement for moisture migration test conducted at Waterways Experiment Station. *Source*: J. E. McDonald, *Moisture Migration in Concrete*, Technical Report C-75-1, U.S. Army Waterways Experiment Station, Vicksburg, Mississippi, May 1975.

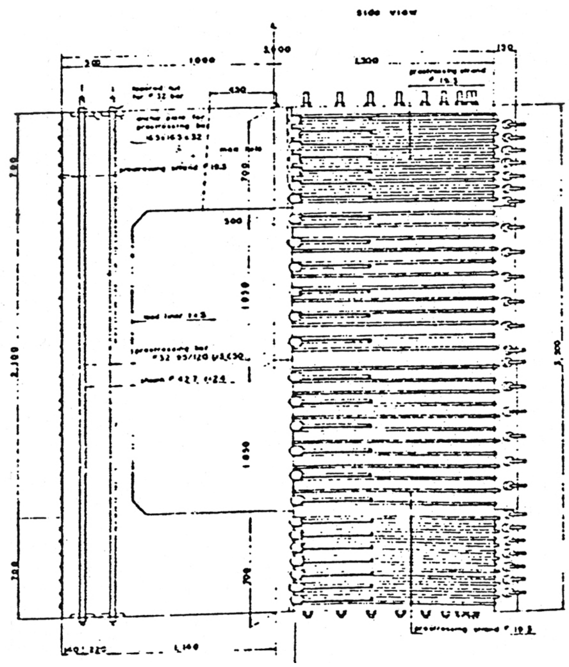


Figure 165 1:10-scale PCRV thermal creep model.
Source: H. Ohnuma et al., "Creep Behavior Under Sustained Temperature Gradient in a Model of Prestressed Concrete Reactor Vessels," Paper H 4/9, Vol. H, *Proc. of 5th Ind. Conf. on St. Mech. in Reactor Technology*, Berlin, 1979.

creep behavior of concrete at high temperature and of selecting suitable prestressing procedures to cope with the stress redistributions caused by thermal creep.

Institut für Nukleare Sicherheitsforschung der Kernforschungsanlage (Germany).[200] At KFA, the effects of a hypothetical accident that can lead to an unrestricted heatup of the reactor core in a high-temperature reactor (HTR) was investigated. For such an accident, it was assumed that all active cooling systems had failed, and during the course of the accident it takes an extended period of time before the temperature reaches a level sufficient to fail the insulation, liner, and concrete. In the experiments, sections (reinforced concrete, liner plate anchored with bolts and cooling pipes) of the PCRV 1 m by 1.5 m by 0.5 m were heated while suspended over an electric chamber furnace (Fig. 166). The facility can heat specimens up to 1500°C using a preset accident temperature-time curve. Two types of high-strength (55–60 MPa) concrete were investigated: a limestone aggregate concrete used in the THTR-300 and a basalt aggregate concrete for the HTR-500. Three tests were completed: two tests utilizing the limestone aggregate concrete without insulation and one test using the basalt aggregate concrete with 10 cm of Kaowool insulation. During a test, which may last up to 6 months, measurements were made of the temperature distribution in the concrete and insulation, pressure buildup, and water released. Calculations of water and gas release were made using a modified version of the Sandia USINT code.[201] Results of the first two tests in which the specimens were heated on the liner side to 1410°C and 1470°C showed that release of CO_2 from the calcitic concrete began at about 600°C with a maximum near 900°C; above 900°C the concrete was granular and powdered, and possessed little, if any strength; at 600°C the concrete retained 45% of its room temperature strength; above 1000°C the liner had lowered perceptibly due to creep; at a liner temperature of 1270°C, a ~3-mm-thick iron oxide layer (scale) began to peel off the liner (when a helium temperature was present, the scale did not form); at 1350–1400°C, a hole formed in the liner

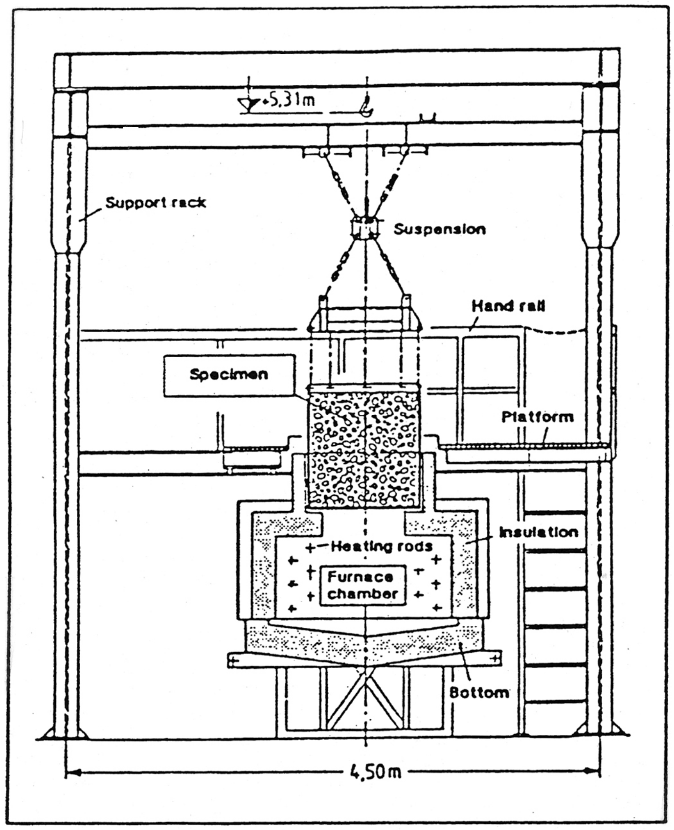


Figure 166 Experimental setup for subjecting PCRV wall sections to elevated temperature. *Source*: J. Altes et al., "Experimental Study of the Behaviour of Prestressed Concrete Pressure Vessels of High Temperature Reactors at Accident Temperatures," *Trans. of the 9th Intl. Conf. on St. Mech. in Reactor Technology*, Vol. H, Lausanne, Switzerland, August 17–21, 1987.

through which molten steel material leaked; and the side of the liner facing the concrete displayed considerable corrosion. In one test, the ability to refeed water into the cooling tubes, after a simulated failure of both trains of the liner cooling system, was investigated. Experimental results showed that for temperatures up to 450°C it was possible to refeed water into the cooling tubes to cool the concrete down to normal operating conditions (liner temperature of 50–55°C).

7 SUMMARY AND CONCLUSIONS

7.1 Summary

Under normal conditions most concrete structures are subjected to a range of temperature no more severe than that imposed by ambient environmental conditions. However, there are important cases where these structures may be exposed to much higher temperatures (e.g., jet aircraft engine blasts, building fires, chemical and metallurgical industrial applications in which the concrete is in close proximity to furnaces, and some nuclear power-related postulated accident conditions). Of primary interest in the present study is the behavior of reinforced concrete elements in designs of new generation reactor concepts in which the concrete may be exposed to long-term steady-state temperatures in excess of the present ASME Code limit of 65°C. Secondary interests include performance of concrete associated with radioactive waste storage and disposal facilities, and postulated design-basis accident conditions involving unscheduled thermal excursions. Under such applications the effect of elevated temperature on certain mechanical and physical properties may determine whether the concrete will maintain its structural integrity.

Concrete's properties are more complex than for most materials because not only is the concrete a composite material whose constituents have different properties, but its properties also depend on moisture and porosity. Exposure of concrete to elevated temperature affects its mechanical and physical properties. Elements could distort and displace, and, under certain conditions, the concrete surfaces could spall due to the buildup of steam pressure. Because thermally induced dimensional changes, loss of structural integrity, and release of moisture and gases resulting from the migration of free water could adversely affect plant operations and safety, a complete understanding of the behavior of concrete under long-term elevated-temperature exposure as well as both during and after a thermal excursion resulting from a postulated design-basis accident condition is essential for reliable design evaluations and assessments. Because the properties of concrete change with respect to time and the environment to which it is exposed, an assessment of the effects of concrete aging is also important in performing safety evaluations.

The objective of this limited study was to provide an overview of the effects of elevated temperature on the behavior of concrete materials. In meeting this objective the effects of elevated temperatures on the properties of ordinary Portland cement concrete constituent materials and concretes are summarized. The effects of elevated temperature on HSC materials are noted and the performance compared to NSCs. A review of concrete materials for elevated-temperature service is presented. Nuclear power plant and general civil engineering design codes are described. Design considerations and analytical techniques for evaluating the response of reinforced concrete structures to elevated-temperature conditions are presented. Pertinent studies in which reinforced concrete structural elements were subjected to elevated temperatures are described.

7.2 Conclusions

A substantial body of knowledge on the material properties of ordinary Portland cement concretes at elevated temperature is available. The use of these data for a quantitative interpretation of the response of reinforced concrete structural elements in nuclear power plants to long-term moderate elevated-temperature exposure (≥65°C) or design basis and hypothetical severe accident conditions needs to be carefully evaluated. In many of these elevated-temperature tests, neither representative materials nor representative environmental conditions were modeled: (1) samples were tested hot or cold, (2) moisture migration was either free or totally restricted, (3) concrete was either loaded or unloaded while heated,

(4) concrete constituents and proportions varied from mix to mix, (5) test specimen size was not consistent, (6) specimens were tested at different degrees of hydration and moisture contents, and (7) heatup rates and thermal stabilization periods varied.

Concrete in the temperature range of 20°C to 200°C can show a small strength loss. Between 22 and 120°C any strength loss that occurs is attributed to the thermal swelling of the physically bound water, which causes disjoint pressures. A regain of strength is often observed between 120°C and 300°C and is attributed to greater van der Waals forces as a result of the cement gel layers moving closer to each other during heating. Between 200°C and 250°C the residual compressive strength is nearly constant. Beyond 350°C there can be a rapid decrease in strength. The following observations can be made relative to the behavior of Portland cement concretes at elevated temperature:

1. Specimens lose more strength if moisture is not permitted to escape while heated than do those where the moisture escapes.
2. Specimens heated and then permitted to cool before testing lose more strength than those tested while hot.
3. Concrete specimens loaded during heating lose less strength than unloaded specimens.
4. The longer the duration of heating before testing, the larger the loss in strength; however, the loss in strength stabilizes after a period of isothermal exposure.
5. The decrease in modulus of elasticity caused by elevated-temperature exposure is more pronounced than the decrease in compressive strength.
6. Relative to the effect of mix proportions, low cement-aggregate mixes lose less strength as a result of heating than richer mixes, and concretes made with limestone aggregate degrade less due to heating than concrete made with siliceous aggregate.
7. The water-cement ratio has a limited effect on strength degradation of heated concrete.
8. Small test specimens generally incur greater strength losses than larger ones.
9. Specimens subjected to several cycles of heating and cooling lose more strength than those not subjected to thermal cycling.
10. The strength of concrete before testing has little effect on percentage of strength retained at elevated temperature.

In general, for structural applications involving service temperatures in the range of ambient to 300°C or 400°C, provided many temperature cycles of large magnitude are not present, Portland cement concretes are the best materials if heat-resistant aggregates (basalt, limestone, or serpentine) are used; and for limited periods of time, temperatures to 600°C could probably be tolerated by the Portland cement concretes.[3] At higher temperatures or for prolonged exposure to temperatures around 600°C, special procedures would have to be considered such as removal of the evaporable water by moderate heating.

Codes and standards for concrete technology recognize that concrete strength tends to decrease with increasing temperature. Consequently, current design procedures specify concrete temperature limits to ensure predictable concrete behavior. Analytical models for accurately predicting the response of a structure to thermal loadings for practical design considerations, where thermal environments exceed the limits contained in the code, are very complex. As a result, most existing methods utilize various types and degrees of simplification that affects the accuracy of results. Current designs for nuclear structures cover these shortcomings by appropriate conservatism in designs. When design conditions exceed established temperature limits, experimental investigations for characteristic mechanical and physical properties data and for design verification may be required to avoid undue and impractical conservatism in design.

Several research projects have been conducted to investigate the behavior of reinforced concrete structures at elevated temperature; however, the overall level of effort has not been sufficient for establishment of widely accepted elevated-temperature concrete design procedures. A review of the literature in which representative concrete structures were subjected to moderate elevated-temperature service indicates that many of these structures have performed adequately; however, some losses in strength and other properties have occurred. Results of these structural tests, together with the material properties data determined in conjunction with these tests, can serve as the basis for numerical modeling of the response of a reinforced concrete structure to a thermal excursion. Analysis methods requiring development are related to the more realistic representation of embedded reinforcing elements, modules for improved representation of time-dependent behavior, better constitutive relationships for input into computer modules, models for cracking analysis, and modeling of concrete behavior under long-term steady-state elevated temperature, or accident conditions resulting in increased thermal exposures and loadings. The end result of improved analysis methods would be the development of significantly improved rules for the analysis and design of reinforced concrete structures for temperatures that exceed those currently permitted by the Code.

If a reinforced concrete structural element in one of the new generation nuclear power plants is required to maintain its functional and performance requirements at temperatures in excess of 400°C, or at moderately elevated temperatures for extended periods of time, techniques for optimizing the design of structural elements to resist these exposures should be investigated (i.e., material selection and design). With respect to material selection, the performance of the concrete materials can be improved by (1) minimizing the moisture content through aggregate gradation, placement techniques, or use of extended-range water-reducing agents; (2) utilizing aggregates having good thermal stability and low thermal expansion characteristics such as lightweight or refractory materials; and (3) incorporating fibrous reinforcing materials such as short, randomly oriented steel fibers to provide increased ductility, dynamic strength, toughness, tensile strength, and improved resistance to spalling. Another possible approach is to design the concrete mix for higher strength so that any losses in properties resulting from long-term thermal exposure will still provide adequate design (safety) margins.

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