A TEST OF A MODEL OF A THIN-WALLED PRESTRESSED CONCRETE SECONDARY CONTAINMENT STRUCTURE

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SUMMARY

The construction and testing to failure by internal overpressure of a 1:14 scale prestressed concrete secondary containment structure is described. The test structure consisted of a rigid base, cylindrical wall, ring beam and dome. The overall height of the test structure above the base was 12' - 6" (3810 mm) and the outer diameter was 10' - 6" (3200 mm). Internal pressure was obtained using water and leakage was prevented by using a flexible plastic liner.

Measurements made during the test include internal pressure, steel and concrete strains using electric resistance gages, concrete strains using manual gages, concrete crack widths and spacing, and curvatures at base of cylindrical wall. All electronic readings were taken using a 280 channel Nova 210/E digital computer and data acquisition system. Manual readings were typed into data files immediately following each test run so that all data could be reduced and plotted using computer routines.

The test structure began to exhibit cracking at an internal pressure of 40 psi (0.28 MPa) and yielding of the reinforcement at approximately 110 psi (0.76 MPa). The structure displayed considerable ductility before failing at an internal pressure of 159 psi (1.10 MPa) by rupture of three horizontal tendons at midheight of the wall.

Introduction

To evaluate the behaviour due to overpressure of thin-walled prestressed concrete secondary containment structures, a comprehensive analytical and experimental study was undertaken at The University of Alberta, Edmonton. The anlytical techniques developed to predict load-deformation, sequence of cracking of concrete and yielding of reinforcement and failure mode for such structures are described in a companion paper (1). These techniques incorporate the response of section and material properties obtained from testing large scale wall segments in biaxial tension (2) (3). To verify the predictive capability of the analytical procedures in the inelastic region and to examine the modes of failure, a prestressed concrete containment was tested to failure. The analysis of this test structure and a comparison with the results is given in (4). This paper contains a description of this test structure and a summary of the observed behaviour.

The prototype for the structure tested is the CANDU reactor secondary containment structure. In general, the dimensions and proportioning of sections in the test structure were chosen to represent the behaviour of this containment under high overpressures. However, to facilitate construction and comparisons of observed to predicted values, the test structure was simplified to consist only of a stiff base slab, a cylindrical wall section, a ring beam and a spherical dome. There were no penetrations in the shell sections but an access hatch was provided in the center of the base. Thus the test structure, while designed to parallel the behaviour of the prototype, should be considered as an independent prestressed concrete containment used for evaluating the reliability of methods developed to predict behaviour of such structures rather than as a model of the prototype.

Design of the Test Structure

The criterion used in proportioning the test structure was that it should have the same cracking sequence under increasing internal pressure as elastic analysis had predicted for the prototype.

An overall scale of 1:14 was selected as a convenient size to be tested in the laboratory. This resulted in an overall height above the base of 12' - 6" (3810 mm) and an outer diameter of 10' - 6" (3200 mm). A section taken vertically through the test structure is shown in Fig. 1.

The thickness of the base was selected as 3' - 6" (1067 mm) to provide a relatively rigid bottom to the model and still provide for a 24 x 30 in.(610 x 762 mm) tunnel along a diameter to provide passage to the access hatch located in the middle of the base.

Using the same scaling ratio for member thicknesses as was used for the overall dimensions would have resulted in thickness values for the dome and wall of 1.71 in.(43 mm) and 3.0 in.(76 mm) respectively. These thicknesses were deemed too thin to realistically model the construction details.

The minimum thickness required to accommodate two 1.0 in.(25 mm) ducts for post-tensioning in orthogonal directions, two layers of reinforcing in each face and maintain 1/2 in. (13 mm) cover was considered to be 4 in.(102 mm). This thickness was used for the dome. Using the ratio of wall to dome thickness as exists for the prototype would give a wall thickness of 7 in.(178 mm). Preliminary calculations using this thickness and full prestress indicated that the internal pressure required to initiate cracking in the wall was approximately four times that for the dome. To obtain the desired cracking sequence the wall thickness was reduced to 5 in. (127 mm) and the level of both horizontal and vertical prestressing decreased. The thickness of the ring beam was taken to be twice that of the wall.

The steel reinforcement was selected so that the ultimate strength of each section was greater than the cracking strength of the concrete. In the prototype the spacing in each direction was between 1/4 and 1/2 of the member thickness. A spacing of 3 in. (76 mm) was chosen for the model.

Originally it was planned to use welded wire mesh with appropriate wire size in each direction at 3 in. (76 mm) spacing. In order to obtain a well defined yield point for the material, a roll of mesh was heated in an annealling oven to $1000^{o}F$ ($538^{o}C$). Subsequent testing indicated that there was considerable variation in the yield point (30 - 60 ksi) at different locations in the roll and so this mesh was discarded. Tests were then made on a new unheated roll but this material was found to have only an elongation at failure of from between 3/4 to 3%. Since this ductility is less than that of the prestressing tendons there would be the possibility of a brittle failure of the test structure. Further use of welded wire mesh was abandoned.

Attention was then given to hot rolled reinforcement. The hoop steel in the wall could be provided using readily available #3 bars (10 mm dia) spaced at 3 in. (76 mm). The yield stress for this material was 50.2 ksi (346 MPa). After some searching a nominal 6 mm bar conforming to Swedish specifications was located. This bar was found to have an area of 0.051 in^2 (33 mm^2) and a yield point of 70 ksi (482 MPa) and was used for the vertical wall reinforcing and dome reinforcing. Details of the reinforcing and prestressing tendons are given in Fig. 2.

Concrete for the model was specified as normal sand and gravel concrete (maximum aggregate size of 3/8 in. (10 mm) with a 28 day compressive strength of 4500 psi (30 MPa). To improve workability a liquid plasticizer was specified for the concrete when casting the thin doubly formed wall sections. However due to delays in placing the concrete in the wall section the life of the plasticizer was exceeded before the bottom lift was completed. This required chipping out the upper layer of poorly compacted concrete. The remainder of the wall section was placed by shotcreting. While the final properties of the shotcrete were different from the concrete used in the lower portion of the wall, ring beam and dome, these differences could be accounted for in the analysis of the test structure and did not significantly influence behavior or the mode of failure.

The connection between the wall and base was designed as a hinge. To preclude the possibility of a premature failure of the hinge, sufficient reinforcement was placed through the center of the hinge to ensure that under the upward tension and outward force expected at an internal pressure of 120 psi (0.83 MPa) that the reinforcement would remain elastic. Single leg stirrups were also placed to resist shear stresses in this region.

Post-tensioning

Each tendon was fitted with a load cell at one end which was monitored throughout the tensioning period. These cells were not recovered but were grouted into the model. All tendons were fully grouted using a pressure grouting system. Examination subsequent to testing indicated that full tendon bond had been achieved.

The vertical tendons in the wall were anchored at the base and pulled from the top only. Four jacks were used and four tendons located at 90° intervals around the perimeter were pulled simultaneously to full design tension of 14 kips (62 kN). There was little relaxation observed due to pull on adjacent tendons and no tendons were repulled.

To reduce friction each horizontal wall tendon extended only over 1/2 of the circumference. Adjacent rows were staggered by 90° resulting in four buttresses located symmetrically around the wall to accommodate the anchorages. The four jacks were used to pull the two tendons at the same level from each end simultaneously. Each row of tendons was initially stressed to half design value in a sequence to maintain as close to uniform stress distribution as possible. The tendons were then pulled to final prestress values. It was found that some tendons had to be repulled to adjust final tensions to within acceptable tolerances.

When prestressing the dome, two parallel tendons were pulled at a time from each end to half design tension values. After all tendons had been pulled to this stress they were then tensioned to final values. Again some tendons had to be repulled to achieve desired tolerances.

Instrumentation

Measured quantities included internal pressure, deflections, steel and concrete strains, crack widths and meridian rotations at the lower portion of the cylindrical wall.

A total of 24 deflection readings were taken electronically along the meridian extending from the base of the cylinder to top of dome and located on the south face midway between buttresses, referred to as line 1, and along a circumferential line 2.25 ft. (686 mm) from the base on the south quadrant.

Strains were measured using electric resistance gages along and across line 1 located as above, on line 2 located on the meridian diametrically opposite line 1 and on line 3 located along the southwest buttress. In addition, average strains were measured manually along and across line 2 using Demec mechanical extensometers. A total of 207 strain gages located on steel reinforcing, 38 gages on concrete faces and 74 Demec gages were read. In addition, 9 pairs of dial gages were used to measure curvatures at various locations.

Crack widths were measured across and along two lines on both the north and west faces using a 40 power microscope. Readings were recorded to the nearest 0.001 in. (0.025 mm).

In general, strain and deflection readings were taken electronically at intervals of internal pressure of 5 psi and manual readings of crack widths and strain at 10 psi although in later stages readings were taken at smaller intervals, the interval being controlled by strain measurements. Above 140 psi (0.97 MPa) only electronic readings were taken.

The internal pressure, deflections and electric resistance strain gages were read and recorded into data files automatically using the Nova 210/E digital computer and data acquisition system. Manual readings were typed into computer files immediately after each test so that data reduction and plotting of results could be completed quickly using computer routines.

Testing

Water was the agent used to develop the internal pressure in the test structure. The pressure was obtained using a hand operated pump to pressures of 140 psi (0.97 MPa) above which a truck mounted high capacity, high pressure pump was used. To prevent the water from leaking through the containment wall after cracking and still provide the opportunity of viewing the interior concrete face after loading sequences, a removable liner made from a heavy plastic material was fitted to the inside surface. A series of five test runs to below the initial cracking pressure were made to test the sealing of the liner at the hatch and to evaluate the response of the instrumentation and data reduction routines. As a result of these runs the seal around the hatch was replaced and certain gages were replaced or rewired.

The first major loading sequence, referred to as load F, was terminated at an internal pressure of 80 psi (0.55 MPa) due to the difficulty of maintaining pressure because water was leaking through the cracks in the walls. After draining the tank and removing the liner it was found that several seams in the liner had failed. A new liner was fabricated that incorporated several modifications learned from experiences with the first liner. Using this liner the final load test, referred to as load G, was taken to rupture of the structure at a pressure of 159 psi (1.10 MPa). Until failure of the test structure no leakage of the second liner was observed.

Observed Behaviour and Mode of Failure

The first visible signs of meridional and circumferential cracking in the cylindrical wall occurred at 40 psi (0.28 MPa) At 80 psi (0.56 MPa) this cracking was extensive and with further loading these cracks became wider and new cracks developed. Fig. 3 shows the crack pattern on the west face of the wall at a pressure just over 130 psi (0.90 MPa). Cracks also appeared at the same time in the outer surface of the dome in directions essentially parallel and perpendicular to the meridians except in the region extending approximately 4 ft. (1220 mm) from the outer edge of the ring beam in which there are no visible cracks. The cracks at the top of the dome increased rapidly near the end of loading being about 1/10 in. (2.5 mm) at 130 psi (0.90 MPa) and 3/8 in. (10 mm) at 140 psi (0.97 MPa) with accompanying spalling of adjacent concrete. The crack pattern for the dome at 130 psi (0.90 MPa) is shown in Fig. 4.

Outward bulging of the cylindrical walls was noticeable at pressures above 80 psi (0.56 MPa) and increased markedly at approximately 110 psi (0.76 MPa). It was observed that the buttresses appeared to bulge almost as much as the other parts of the cylinder implying that the effects of their flexural stiffnesses were small. The degree of bulging of the northwest buttress at a pressure just over 130 psi (0.90 MPa) can be seen in Fig. 3.

This marked increase in deformation at approximately 110 psi (0.76 MPa) is associated with the beginning of widespread yielding of the reinforcement. The degree of ductility inherent in the structure is evident from Fig. 6 in which the outward deflection of the wall at midheight (channel 225) and the upward deflection near the crown of the dome (channel 237) are shown. The measured deflections at these points immediately prior to the apparent drop in load associated with failure are 2.5 in. (64 mm) and 3.0 in. (76 mm) respectively. The break in the load-deflection curves corresponding to a drop in load from 148 psi (1.02 MPa) to 135 psi (0.93 MPa) and subsequent reloading indicates the amount of deflection occurring after the first day of load test G. The pressure was intentionally reduced to 137 psi when leaving the structure overnight. When loading, using a high capacity pump, was resumed after an interval of 14 hours, the pressure had dropped only 2 psi (0.01 MPa). The gap between the two points after reloading to the previous pressure is an indication of the creep that occurred during that period under a pressure of approximately 135 psi (0.93 MPa). Thus it can be seen that although the test structure was cracked extensively and the internal pressure was over 85% of failure pressure it was still able to maintain this pressure over a significant time period with very little additional deformation. This was not altogether unexpected as the structure was essentially being held together by the prestressing tendons.

Between 142 and 145 psi (0.98-1.00 MPa) one of the 7 wires in two vertical tendons and one horizontal ring beam tendon fractured near an anchorage. These were accompanied by a loud bang and an unravelling of the strand extending beyond the anchorage. In each instance a small drop in pressure was observed momentarily in the electronic pressure transducer but the load appeared to redistribute quickly and the fracture of these wires did not appear to influence behaviour. It was noted that none of these wire fractures occured in the vicinity of final failure.

Failure of the test structure occurred when three horizontal tendons and one vertical tendon fractured at midheight near the south-east buttress permitting a rupture of the liner. Immediately prior to failure the concrete cover over one of the horizontal tendon anchorages at this buttress was observed to pop outwards followed by rapid opening of the cracks in the immediate vicinity and spalling of the concrete. The failure region showing this spalling and the ruptured tendons is shown in Fig. 5.

Sample Results

A comparison of measured and predicted strains along the meridian (line 1) and the circumferential strains across this meridian are given in Fig. 7 and 8, respectively, for an internal pressure of 120 psi (0.83 MPa). In general the agreement is very close. The apparent large differences in circumferential strains at midheight of the wall can be better interpreted by examining the load-strain curve for the gage at midheight given in Fig. 9. The horizontal difference in Fig. 8 is the same distance between measured and computed values in Fig. 9 for a pressure of 120 psi (0.83 MPa). At lower pressures (i.e. below yielding at 110 psi) the differences would have been very small.

Fig. 9 also shows clearly the effects of cracking on the stiffness of the structure. The analytical curve is a monotonic loading and the effects of concrete cracking and yielding of the reinforcement can be readily seen by the breaks in the curve. The solid points represent load F which was taken from zero load and uncracked structure to 80 psi (0.56 MPa) Measurements from this test closely follow the predicted values. The open points represent measured values from test G which went from zero load in a cracked structure to failure.

It is seen that the strains for the cracked structure are much greater than for the uncracked in the initial loading range.

A similar behaviour is shown in Fig. 10 for a point near the crown of the dome. Again the analysis clearly shows the point of concrete cracking and yielding of the reinforcement with the observed values of test F closely following the predicted values and test G being less stiff in the preloaded range.

Conclusions

The prestressed concrete containment structure tested incorporated many of the construction details found in secondary containments. The response to loading from internal pressure and the mode of failure are representative of the behaviour of such structures. Agreement between measured and predicted values was good.

Acknowledgements

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Figure 3.
Crack Pattern and Bulging of Wall at 130 psi (0.90 MPa)

Figure 4.
Crack Pattern in Dome at 130 psi (0.90 MPa)

Figure 5.
Failure Zone, Note Anchorage Distress, Broken Reinforcement and Tendons

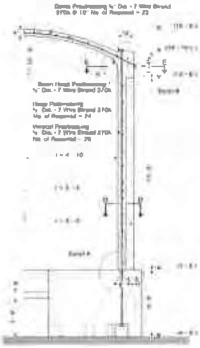

Figure 1.
Vertical Section Through Test Structure

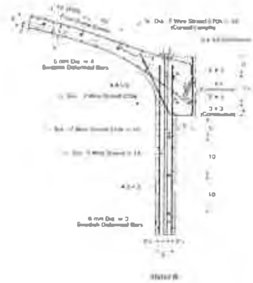

Figure 2.
Reinforcement Details

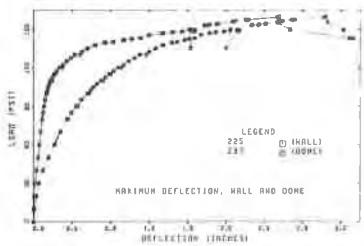

Figure 6.
Load-Deformation Relationships for Two Critical Points

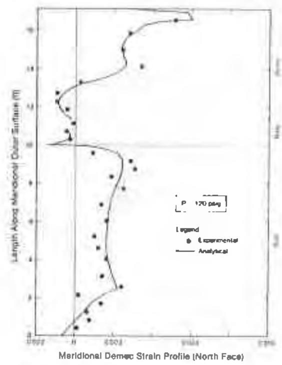

Figure 7.
Measured and Computed Meridonal Strains, 120 psi (0.83 MPa)

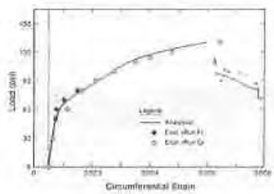

Figure 9.
Measured and Computed Circumferential Strains in Wall at 65 inches (1650 mm) above base

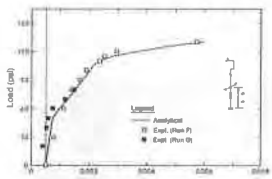

Figure 10.
Measured and Computed Meridonal Strains near Crown of Dome

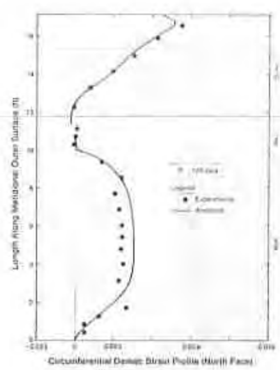

Figure 8.
Measured and Computed Circumferential Strains along Meridian, 120 psi (0.83 MPa)