Response of Prestressed Concrete Containment Model in Post-Cracking Region

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SUMMARY

Analytical procedures were developed to predict the deformations, crack patterns and failure modes of prestressed concrete containment structures due to increasing internal pressure. To evaluate the reliability of these procedures a test structure having an outer diameter of 10'-6" (3200 mm) and a height above the base of 12'-6" (3810 mm) was built. Construction details and materials that were patterned after the Canadian 600-CANDU-PHW reactor containment. This structure was instrumented and tested to failure. Agreement between predicted and observed behavior was generally good to excellent.

The analytical procedures were incapable of properly considering the localized stiffening effects of the buttresses used to anchor the circumferential tendons. This paper discusses the influence of the buttresses on the structural response to internal pressure. It was found that the buttresses do provide significant stiffening to the outward movement of the wall and influence the crack pattern particularly on the inside face. It was also observed that the crack pattern resulting from the outward bulging of the buttress may decrease the capacity of the tendon achorage by permitting a wedge of concrete to spall off the buttress face in this region.

Description of Test Structure

The secondary containment buildings for reactors of the 600 MW-CANDU-PHW type are constructed from post-tensioned reinforced concrete and are designed to have no tensile stresses at the inner surfaces under design accident conditions. No internal structural lining to prevent leakage is provided. To evaluate the accuracy of analytical procedures developed to predict overall behavior of such structures in the unlikely event that all safety systems fail simultaneously with a design accident thereby permitting the internal pressure to exceed the design pressure, a test structure was constructed and loaded to failure.

The test structure consisted of a base slab, cylindrical wall, ring beam and dome as shown in Fig. 1. The anchorages of the horizontal prestressing tendons in the wall were located in four buttresses spaced at 90 deg. intervals around the wall (Fig. 2). At each tendon elevation, there were two horizontal tendons each extending half way around the vessel. Alternate rows were anchored 90 deg. apart. Reinforcement and construction details are given in Ref. 2.

It is important to note that the test structure was not intended to model any particular nuclear containment structure. Rather, it was a unique structure built to determine the accuracy of analyses developed to predict the response of containments to high overpressures. The overall dimensions are roughly 1/14 those of the CANDU type of containment and the reinforcement patterns are also similar. The model differs from the prototype containment in the ratio of wall and dome thickness to overall size, there is no inner dome (for dousing system) in the model and penetrations were not included.

Summary of Testing

To preclude the possibility of an explosive failure and to be able to maintain a constant pressure with time when readings were being taken, water rather than air was chosen as the loading fluid. To prevent leakage of the loading fluid in the post-cracking region and to permit the build up of pressures sufficiently great to fail the model, a plastic liner was fitted to the inside of the test structure.

Extensive measurement of internal pressure, deflections, steel and concrete strains, crack locations and crack widths were made. These measurements included both manual and automatic electronic readings.

While several loadings to pressures below cracking were made to test the reliability of the gages and recording devices, there were two significant tests. The first test was to 80 psi which was sufficient to cause extensive cracking of both the wall and dome segments. The structure was then drained, the liner removed and the inside inspected. The second test was to failure and because of the time required was performed in two days.

Overall Behavior of the Test Structure

For internal pressures corresponding to those used for design of prototype structures for leakage criteria, the test structure responded linearly with no observed cracking and no observeable residual deformation after unloading. With increased pressures this response was linear until approximately 40 psi (0.28 MPa). At this load level the first cracks were observed near the midheights of the wall and were in the circumferential (horizontal) direction. Between pressures of 40 psi (0.28 MPa) and 50 psi (0.35 MPa) several cracks were observed in both the meridonal and circumferential directions throughout the wall and central portion of the dome. At 80 psi the crack pattern was well developed and reflected the tendon and reinforcement pattern in the wall and central portion of the dome. No cracks were observed in the outer portion of the dome surface.

After reaching 80 psi (0.56 MPa) the structure was unloaded. After unloading there was a residual outward deflection of the midheight of the wall of 0.022 in.(0.56 mm) and a residual upward deflection of the crown of the dome of 0.035 in. (0.90 mm).

Upon reloading the existing cracks reopened. With further loading beyond 80 psi (0.56 MPa) these cracks became wider and new cracks developed such that at approximately 110 psi (0.77 MPa) there was a crack at essentially each tendon and reinforcement location in the wall and central portion of the dome. This pressure corresponds to that required to cause widespread yielding of the reinforcement in the structure. At higher pressure loads few new cracks formed but the existing cracks continued to open.

The test structure showed a remarkable amount of ductility. This was evidenced by the noticeable outward bulging of the walls at pressures around 80 psi (0.56 MPa) which became much more pronounced after 110 psi (0.76 MPa). The outward bulging of the northwest buttress and the crack pattern at a pressure of 135 psi (0.93 MPa) can be seen in Fig. 2. The maximum pressure reached in the test was 159 psi (1.10 MPa) at which time the outward deflection of the midheight of the wall between buttresses was 2.5 in. (63 mm) and the upward deflection of the crown was 3.0 in. (76 mm).

Failure of the test structure resulted when a horizontal and vertical tendon ruptured at a pressure of 159 psi (1.10 MPa).

Effect of the Buttresses

The presence of the buttresses used to anchor the prestressing tendons will have a localized stiffening effect on the wall of the structure. Since this effect could not be handled adequately using the analytical tests developed, the influence of the buttresses in the prototype must be inferred from the test structure.

The influence of the buttresses in restricting outward movement of the wall along a line located 26 in. (660 mm) above the base is shown in Fig. 3. This Fig. also indicates the large observed deformations that occurred with pressures greater than 110 psi. From deformation readings it was concluded that the outward movements of the buttresses varied from 40 to 70% of the maximum wall movement, the lower value being near midheight of the wall.

At the same time that cracks were observed in the wall, horizontal cracks were observed across the face of the buttresses. These were initially located on either side of the prestressing anchorage plates. As the internal pressure was increased intermediate cracks were formed. These cracks can be seen in the photograph of the northwest abutment in Fig. 2.

By the time the pressure reached 135 psi (0.93 MPa) there was a noticeable outward movement of the concrete in the face of the buttress in the region between the anchor plates. This can be seen in Fig. 4. In some instances this outward wedging of concrete was accompanied by an inward movement of the anchorage plates. At several locations the inward movement of the anchorage plates into the abutment increased as the load increased.

Major vertical cracks were observed on the inside faces of three of the abutments indicating very high horizontal tensile strains across the inside face of the buttresses. This is to be expected from the deformed shape in Fig. 3. In the test structure all circumferential reinforcement bars were fabricated from a single rod and made continuous by a single lap welded splice. These splices were all located in the northwest buttress and overlapped the full width of the buttress. No major vertical cracking was observed on the inside face of this buttress which indicates that additional reinforcement in this region may have a beneficial effect on behavior.

Failure Mode of the Test Structure

At pressures above 110 psi (0.76 MPa) the outward deformations of the structure became visibly apparent and the crack widths increased. At a pressure of 142 psi (0.98 MPa) a loud bang was heard and inspection indicated the upper anchorage of one vertical tendon had slipped. It was noted that a small drop in pressure was measured momentarily in the electric pressure transducer but that the load recovered quickly. This slippage also occurred in another vertical tendon at 144 psi (0.99 MPa) and in a horizontal tendon in the ring beam at 145 psi (1.0 MPa). It is noted that none of these anchorage slips were in the vicinity of the final failure and it appeared that after the anchorages slipped approximately 1/4 in. (6 mm) the tendons continued to carry load.

At a pressure of 153 psi (1.06 MPa) slippage occurred at the upper end of three more vertical tendons. One of these was in the southeast buttress which permitted the horizontal cracks in the buttress face to open wider. A movie was taken of the final stages of loading. From viewing the film of the failure and inspection of the failure zone after the test the following failure sequence could be determined.

The 6th and 8th tendons from the base are anchored in the southeast buttress at approximately the midheight of the wall. As the pressure approached 159 psi there appeared to be a slippage at one or both of these anchorages which permitted a sudden opening of the cracks in the adjacent portions of the wall. This relaxation of the ends of these two tendons caused an increase in force to be carried by the 7th tendon from the base which is continuous through the southeast buttress. This tendon fractured very shortly after at a point about 5 inches (125 mm) to the right of the buttress.

Almost instantly after the fracture of this horizontal tendon the vertical tendon at this location also fractured which permitted the liner to burst.

Closure

While there is no doubt that the ultimate pressure capacity of the test structure would have occurred by fracture of a horizontal prestressing tendon at approximately midheight of the wall the actual pressure and location were probably influenced to some extent by the partial relaxation at that point of the two adjacent tendons. It would appear that the relaxation at this point was due to anchorage slippage at a load that was very near the fracture load of the tendons. However, there could also have been some relaxation of the tendon anchorages by the shearing action permitted by the outward wedging of the concrete between the anchorage plates. In any event the capacity of the tendon anchorages in the test structure were not appreciably greater than the tendon capacity.

The capacity of the tendon anchorage zone is reduced by the formation of the horizontal cracks across the face of the buttress and the outward wedging of the concrete on the face of the buttress between the anchorage plates. The ratio of cracked moment of inertia of the buttress section to the cracked moment of inertia of the wall section for the test structure was approximately twice that for the prototype. Thus the stiffening effect of the buttresses in the test structure at high internal pressures would be expected to be greater than in the prototype. For this reason it is concluded that in the design of the tendon anchorages and buttress reinforcement some consideration must be given to ensure that under the cracking conditions in the buttress caused by high internal overpressure the ultimate capacity of the tendon anchorage will continue to exceed the capacity of the anchored tendons.

References

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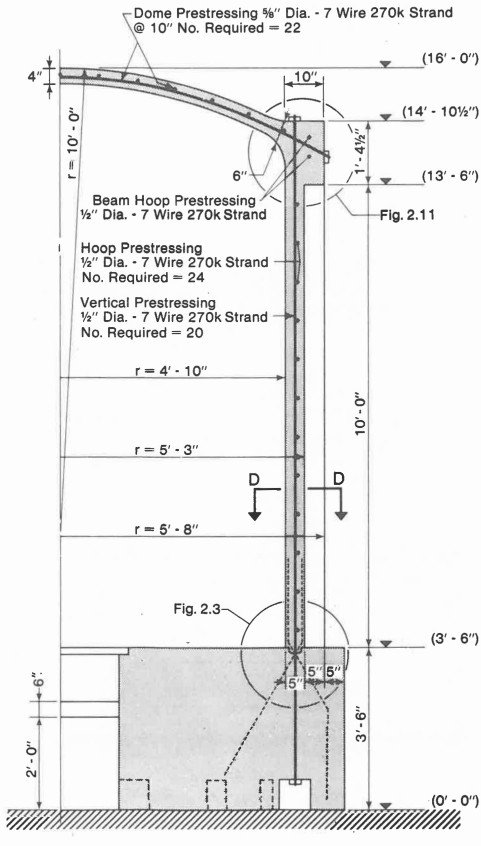

Fig. 1 Vertical Section through Test Structure.

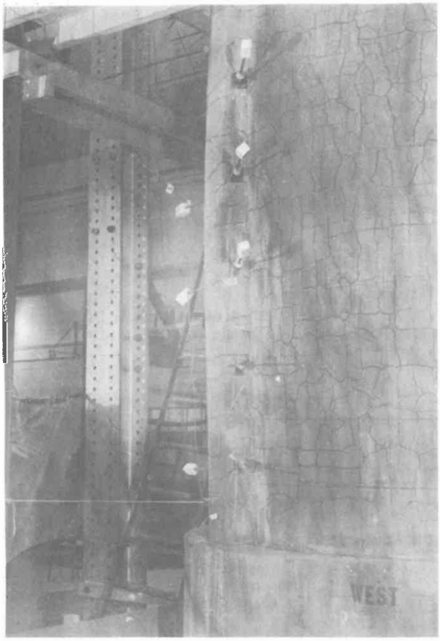

Fig. 2 Outward Bulging of Northwest Buttress at 135 psi.

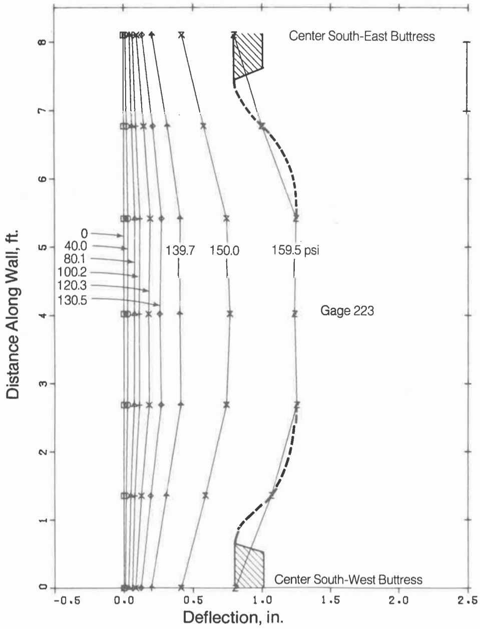

Fig. 3 Outward Displacement of Wall at 26 inches (660 mm) Above Base.

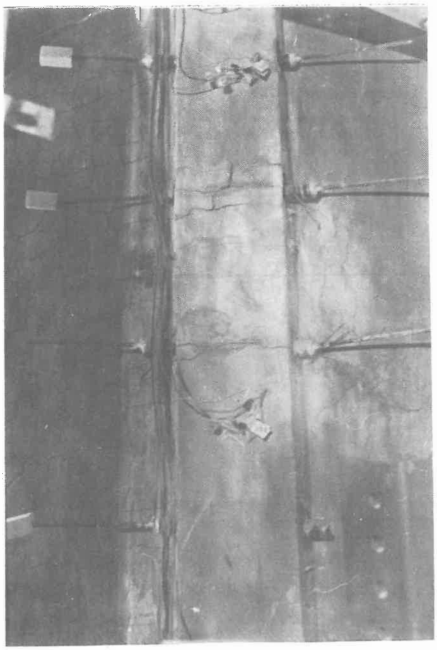

Fig. 4 Cracking at Tendon Anchorages in Southwest Buttress at 135 psi.