Darlington Generating Station Vacuum Building

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Abstract

The paper describes aspects of the design of the Darlington Vacuum Building, its structural configuration, analytical procedures, and reinforcing steel layout. Attention focuses on the ring girder where the juncture of dome and perimeter wall produces a complex post-tensioning layout, and attendant difficulties in design and construction. At the wall base, full fixity imposes large local stresses. Long-term shrinkage and creep, and temperature effects become significant. A research program and in-house analytical procedure established time-dependent concrete behaviour and corresponding wall-sectional stresses. The outcome is examined in terms of reinforcement, temperature controls, and wall liner requirements.

1.0 The Darlington Vacuum Building

The Darlington containment consists of a 48-metre inside diameter wall 1.2 metres thick, supporting a 1-metre thick dome having an inside radial configuration of 36 metres. At its periphery a massive ring girder serves to carry the dome thrust and anchor the prestressing tendons of both the dome and wall which oppose the tensile stresses generated by positive pressure, (Figure 1). Since the roof is self-supporting the containment envelope is a separate entity, independent of, and wholly enclosing the dousing system.

The structural behaviour of the perimeter wall - foundation slab joint was thoroughly examined under many loads and loading combinations. This included examination of long-term shrinkage and creep and thermal effects.

2.0 Shrinkage and Creep Laboratory Testing

To obtain reliable shrinkage and creep data for the design of the vacuum building containment shell, a concrete specimen testing program was initiated. The concrete mix was formulated for slipformability and low shrinkage and creep with low heat of hydration cement, giving a design strength of 35 MPa at 28 days. The properties examined in the tests included shrinkage, basic creep, drying creep, strength, coefficient of thermal expansion and modulus of elasticity. A total of seventy-two cylinders and nine prisms was used in the tests. The cylinders for testing basic creep were sealed with Bituthene sheets. Test data collected in a one year period indicate that the shrinkage and creep deformation of this mix is normal and comparable to past and other published data.

3.0 Design for Shrinkage and Creep Effects

The prediction of long-term shrinkage and creep behaviour of full-sized structural members is subject to large uncertainties. To obtain realistic results, it is necessary to use a comprehensive model capable of accounting for all important contributory factors. The BP Model[1] of Professor Bazant was chosen for its applicability to very thick concrete elements. Predictions can be made from the mix composition, 28-day strength, type of cement, relative humidity, curing period, temperature, age at loading, and the volume/surface ratio and shape of the concrete element. To evaluate stresses arising from differential shrinkage and creep, Bazant's "Age-adjusted Effective Modulus Method"[2] was used.

Actual test data were compared with BP Model predictions with good agreement. The differences are as follows: shrinkage 18 percent, basic creep 10 percent, and drying creep 17 percent.

4.0 Base Joint Design

The wall has been designed as a thin-walled, homogeneous, elastic cylinder rigidly fixed at its base, and undergoing primary (membrane) stresses generated by the external loads, and secondary stresses localized at the bottom as a direct consequence of the fixity. Three layers of 35M bars spaced horizontally at 125 mm and 180 mm vertically in each face, along with 20M ties at 500 mm intervals horizontally and 180 mm vertically are required largely to resist the moments and shears so induced. Ten metres above the base secondary stress is negligible and only about twelve percent of the base steel provision is needed to resist primary stresses and temperature/shrinkage effects.

Numerous combinations of pressure, temperature, shrinkage/creep, hydrostatic (dousing water), seismic, prestress, and self-weight loads were applied in accordance with the CAN3-N287.3-M82 code. The internal negative pressure, temperature decreases (To) and hoop prestressing (Fp) were most influential, and their cumulative effect imposed very large bending moments (tension on the outside face) and hoop tension across the full section exceeding the modulus of rupture. Hoop prestressing poses an interesting dilemma for the design engineer because, while it helps to precompress the wall over most of its height, at the base it has the opposite effect.

Even small variations in temperature from that at which the initial concrete set occurred induce substantial secondary stresses. An accident induced temperature rise will combine with the attendant positive pressure to develop flexural stresses placing the inner wall surface in tension. Happily concrete is slow to absorb or release heat; hence the most severe accident causes a temperature excursion whose duration is too short to influence concrete stresses. The wall is designed for the long-term temperature rise of the dousing water - just 10 centigrade degrees above ambient. Consequently, the most severe loading case actually derived from the winter temperature drop (in combination with prestressing, vacuum, and temperature forces). Here too there was a moderating factor; heaters placed in the manifold will prevent temperatures from declining below +2°C. Also, the temperature gradient creates flexural stresses tending to cause compression on the inside face.

The foregoing discussion emphasizes that base fixity will inevitably lead to large secondary stresses with the likelihood of vertical cracks forming. Therefore, the following precautions were taken to mitigate the consequences: (a) the provision of a liner over the first 6.5 metres of inner wall surface; (b) a judicious choice of hoop prestress inducing a small compression in the wall above the liner for all loading conditions, while contributing as little as practical to base moments; (c) the specification of a low placing temperature, between 4.5°C and 12°C for the conventionally formed part of the perimeter wall; (d) maintaining accessibility for proper concrete vibration during construction; and (e) minimizing crack widths by introducing construction joints to maintain a 1:1 height-to-length ratio, choosing a pour sequence avoiding numerous "filler" pours, and placing a wire mesh on the outside face.

The lower 6.5 metres of the wall were thickened in order to accommodate the steel liner and its channel anchors without displacing the very substantial base reinforcement. Thus, while the upper portion was slipformed, the bottom was conventionally placed with the liner serving as formwork for the inside face.

The spacing of construction joints between pours was somewhat irregular because they had to be placed between the bearing plates of the vertical tendons. These anchorages within the gallery roof slab were in turn dictated by the layout of the dome tendons, since numerous points of interference exist within the ring girder between dome and vertical wall tendons. In fact, the final arrangement of pours and reinforcement of the wall base and gallery slab had to await completion of the dome tendon layout model.

The base reinforcement is extremely dense. Because there are three rows of 35M vertical bars on both faces which dowel into the gallery slab, and because these vertical rows alternate with three layers of horizontal steel, it became necessary to place virtually all the lower reinforcement before the gallery roof slab was poured (Figures 2 & 3). It would have been impossible to "thread" the curved horizontals between vertical bars already anchored in concrete.

To avoid further compounding the congestion, no splicing of the verticals was allowed in the conventional portion. For Construction, this meant stabilizing a cage of reinforcement rising as high as eleven metres before pouring the gallery roof slab. While the use of 45M or 55M bars would have allowed a greater spacing between verticals, there was insufficient depth in the gallery roof to develop adequate anchorage. Moreover, concrete containing such large bars is more prone to shrinkage cracking.

The wall design includes a steel percentage to resist tangential shear. Diagonal steel was not needed; it would have required bending into a spiral curve and caused innumerable construction difficulties.

5.0 Temporary Heating Requirement of Perimeter Wall

During the first two winters after slipforming the perimeter wall but before pouring the ring girder and dome, thermal contraction of the wall could cause cracks above the steel liner on the inside face. Temporary heating of the inside face is therefore specified during this period for the lower 7.5 m of the wall.

This problem will not occur after closing of the vacuum building (pouring the ring girder and dome) and erection of the manifold since a heat transfer study indicates above-freezing temperatures inside the building at all times.

6.0 Perimeter Wall Liner

The bottom 6.5 metres of the perimeter wall are lined with 7 mm thick carbon steel plate (Figure 4). The analysis indicates that through cracks might occur in the region due to high discontinuity forces arising from shrinkage, temperature effects, prestressing forces, etc., so a liner is needed to achieve air-tightness. The remainder of the perimeter wall and the dome will not experience such conditions and will not be lined.

The main design considerations were temperature change, negative pressure, shrinkage, creep, and prestress forces. The liner plate spans horizontally between vertical C100x8 anchor channels embedded in the wall. For convenience, 2.4 m wide panels consisting of vertical channels, horizontal channels at top and bottom, and the 7 mm thick liner welded on were fabricated in the supplier's shop and then erected on site and used as part of the concrete forming. The panels were joined together by field welds in their final positions.

The 480 mm maximum spacing of the vertical channels was set by the span capacity, in flexure, of the liner plate when acted on by full atmospheric pressure from behind with design gauge negative pressure of 96 kPa in the vacuum building. Welding of the plate to the channels was mostly determined by the effects of temperature change, shrinkage, creep, and prestress forces. All of these can induce compression in the liner. The welds will be stressed most severely when a non-buckled span is adjacent to a buckled span since the post-buckling strength can not balance the full force in the non-buckled span. The welds must then transfer force to the anchoring channels. Also it was assumed that the liner will heat up to the maximum design accident temperature of 55°C with no increase in the concrete temperature. Continuous 5 mm fillet welds between the liner plate and the channel flange at both the toe and heel were required. Air-tight welds were specified at panel perimeters.

The liner will be coated with coal-tar epoxy for protection against corrosion. High humidity will exist in the vacuum building. Also, based on inspection of the Pickering vacuum building, corrosion could be induced by sulphate-reducing anaerobic bacteria. Coal-tar epoxy has been proven effective over many years of use.

7.0 Analytical Approach to the Containment Shell

Since the containment shell is unlined except for the bottom 6.5 meters of the wall, the design approach aims at preventing tension on the inside concrete face. Post-tensioning is used to achieve this goal.

The structural analysis of the containment shell was carried out using the NASTRAN finite element program. An axisymmetric model was used to evaluate axisymmetric loads such as design pressure and shrinkage and creep; a nonaxisymmetric model was assembled to handle earthquake, explosion, and dome tendon anchorage reactions; a lumped-mass dynamic model provided the seismic response.

CAN3-N287.3 Code Design Requirements for Concrete Containment Structures for CANDU Nuclear Power Plants imposes both strength and stress requirements on a prestressed concrete section. The concrete section must demonstrate both sufficient strength and compliance with maximum allowable stresses in concrete and reinforcing steel.

Concrete section design was carried out using a section analysis program developed in-house. This program evaluates the strength and working stress conditions of a prestressed concrete section. In the strength design mode, the force-moment interaction diagram of the section is plotted. In the stress analysis mode, the stress-strain conditions of concrete, reinforcement and prestressed steel are evaluated, including thermal gradient effects and the internal force redistribution caused by long-term shrinkage and creep.

8.0 Perimeter Wall Slipforming

The perimeter wall between elevation 97.4 m and 148.1 m was poured in one single concreting operation which lasted 220 continuous hours (Figure 5). The slipforming technique was used for this very major concreting operation which called for detailed planning and close cooperation and interface between many Construction disciplines and Engineering. All concrete, reinforcing steel, embedded parts, post-tensioning ducts and trumplate assemblies were placed from two upper platforms. The lower platform was used mainly for finishing concrete.

The specified concrete mix provided a compressive strength of 35 MPa at 28 days. The strength of concrete, as it passed the form's lower edge was sufficient to resist any permanent deformations from loads present during the slipforming period.

For the first three metres of slipformed wall, concrete pouring temperature was specified to be a maximum of 18°C; for the rest of the wall, it could vary between 12 and 21°C.

The main reinforcing steel was kept as simple as possible, 30M at 250 mm each face, horizontally and vertically. Radial "U" shaped 20M ties are placed at 1 m apart, horizontally and vertically to resist the delaminating effects of the curved cables.

9.0 Ring Girder

The basic purpose of the ring girder is to provide sufficient space for anchoring the dome tendons and to restrain the dome thrust. After examining several alternatives, the present configuration was adopted since it gives more favourable stresses. Hoop post-tensioning was introduced at the top of the ring girder to better control the dome thrust under dead load and negative pressure.

Bursting, spalling and shear stresses at the dome cable anchorages are rather complex and triaxial in nature. It is generally agreed that Guyon & Zielinskis' method provides conservative results for the design of reinforcement. From Guyon's assumption, tensile stresses occur on the axes of the applied loads and between the loads in addition to the tensile stresses around the surface of box-outs. Therefore, the reinforcement in the ring girder is classified into four types: cages, ties, surface reinforcing, and main structural reinforcing.

Reinforcement in the ring girder is extremely dense and complex, therefore, the sequence of reinforcement erection and provisions to allow proper concrete placement and vibration are most carefully considered in the detailed design.

10.0 Ring Girder Mock-Up

A full size mock-up was built for that part of the ring girder containing nine pockets at the steepest angle (Figure 6). Here congestion of steel is likely to cause the biggest design and erection problems. The main benefits of the model include, but are not limited to the following: (a) layout of reinforcing steel around the box-outs; (b) establishing logical erection and reinforcing steel sequence; and (c) organizing survey layout and control.

11.0 Post-Tensioning System

A post-tensioning system has been introduced to negate the tensile stresses developed on the inside face of the containment shell under positive pressure. The perimeter wall is post-tensioned biaxially with 152 vertical and 99 horizontal tendons: the dome by means of 102 cables divided equally between three layers oriented at 60 degrees to one another; and the ring girder by four circumferential tendons.

All vertical wall and all dome cables are anchored in the ring girder producing a complexity of anchorage boxouts and difficulties in steel placement. This intricate interaction of boxouts and tendons prompted the decision to produce a scale model of the dome and ring girder tendons. By portraying the third dimension, the model of the post-tensioning system provided a valuable design tool and enhanced constructability (Figure 7). The main benefits were as follows. (a) The establishment of locations of vertical cables so that there would not be interference with dome cables and their anchorage box-outs. This activity took place very early during the detailing of reinforcing steel for the gallery slab, since the position of all bearing plates for the vertical cables had to be known. (b) The layout of ring girder construction joints. Dome cables are placed in three layers with 34 cables in each layer. The dome comprises a spherical segment and each dome cable is located on a great circle of that sphere. Each layer of cables passes through a set of poles. The three sets of poles are located 60° apart on the horizontal great circle of the dome sphere.

The unbonded (greased) tendon system was selected for the vacuum building post-tensioning system. The chief reason is that a grease filler permits periodic lift-off readings during the operation to determine the actual tendon stress as well as the retensioning of the tendon to a higher stress level if it is found to be below the design level.

12.0 Loads and Loading Combinations

The structure is designed to withstand fourteen load combinations categorized as Construction, Test, Normal, Abnormal, Environmental, and Abnormal/Environmental. All include a core of loads: dead "D", shrinkage and creep "S_c", prestress "F_p", plus either the ambient temperature effect "T_o", or an accident temperature LOCA "T_A" or maximum long-term design temperature "T_{AI}" (35°C). Twelve combinations involve the operating negative pressure (-96 kPa gauge), test (1.15 P_{AI}) and accident conditions: LOCA "P_A", or maximum design positive pressure "P_{AI}" (+96 kPa gauge). As well, eight cases impose an external load: the site design earthquake "F_{SDE}" (0.03 g ground acceleration, one-hundred year recurrence); the design basis earthquake "F_{DBE}" (0.08 g ground acceleration); a pressure wave "X" generated by the explosion of a railway car containing 61.5 mg of TNT; or a tornado "W_T". A hydrostatic pressure "L_W" representing the dousing water to a height of six metres is imposed in six cases. The effects of wind, snow, and rain are comparatively insignificant.

13.0 Dynamic Wind Effects on Vacuum Building

Dynamic wind effects arise from periodic variations in the pressure distribution around the circular shell of the vacuum building. They generate three types of oscillation, namely in-wind, ovalling and cross-wind oscillation.

For the vacuum building, with aspect ratio H/D of 1.4, a wall thickness of 1.2 m and natural frequency equal to 8.5 Hz, the first two types of oscillation are negligible. Static analysis shows that to bring the structure in resonance with the vortex shedding, the approaching wind velocity must be in the range of 6,500 km/h which is beyond the realm of possibility.

14.0 Accidental Explosion

The postulated explosion of a railway car containing 61.5 Mg of TNT on the CNR track 580 m from the perimeter wall was taken into account during design. The blast wave is considered as a surface burst type and plane wave with peak overpressure of 18.6 kPa at the front and 5.6 kPa at the rear face.

Complete static and dynamic analyses of the containment shell for both explosion and tornado loads by the non-axisymmetric finite element model were conducted.

15.0 Tornado

The basic characteristics of the design tornado for the Darlington site are as follows: translational wind speed of 96 km/h; rotational wind speed of 320 km/h; and radius of maximum rotational wind of 46 m. The analysis indicates that the containment shell can withstand tornado missile impacts within the elastic range. With a recommended ductility ratio of 10 for such loadings, the effects of tornado missiles are therefore minimal.

16.0 Buckling Analysis

The stability of the containment shell was evaluated by two approaches. In the first, the effect of the nonlinear concrete stress-strain relationship was accounted for by employing the tangent modulus at the appropriate stress level. The influence of creep was addressed by Bazant's "Age-adjusted Effective Modulus Method", and the benefit of reinforcement was included by using the transformed section properties for analysis. A second approach following the recommendations of the International Association for Shell and Spatial Structures was taken as a check. The results indicate that crushing failure would occur before buckling.

17.0 Dome

The possibility of delamination under prestressing will receive attention in the design of the dome. Delamination would constitute a major failure and has occurred in similar prestressed concrete domes. The effects of such contributing factors as radial tension due to prestress, biaxial stress failure, and the sequence of prestressing will be examined closely. To design against delamination, the ASME Code Section III Division 2 requires the placement of radial steel in a prestressed concrete dome.

Acknowledgements

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References

[1] BAZANT, Z.P., and PANULA, L., Practical Prediction of Time-Dependent Deformations of Concrete, Materiaux et Constructions, Vol. 11 No. 65 (1978) 307-328, Vol. 11 No. 66 (1978) 415-434, Vol. 12 No. 69 (1979) 169-183.

[2] BAZANT, Z.P., Prediction of Concrete Creep Effects Using Age-adjusted Effective Modulus Method, J. Am. Conc. Inst. 69 (1972) 212-217.

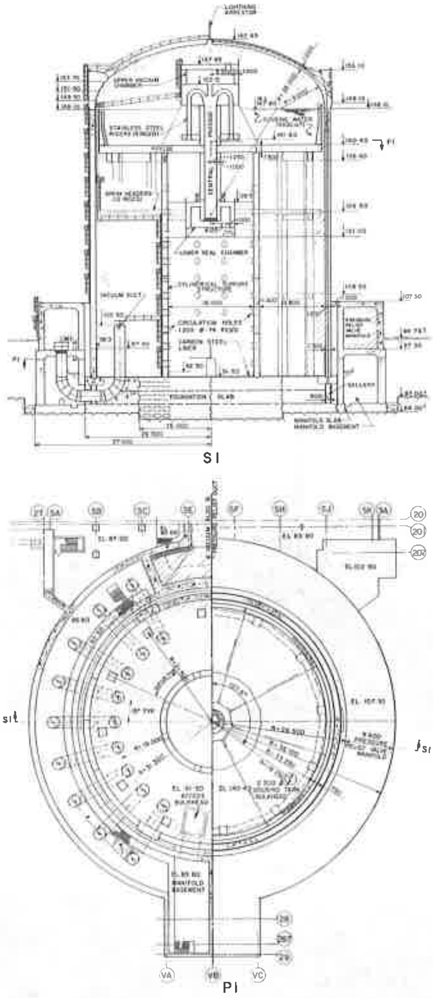

Figure 1: Darlington GS Vacuum Building - domed roof, ring girder and perimeter wall are monolithic prestressed concrete independent of internal structure. The floor and the bottom 6.5 metres of wall are lined.

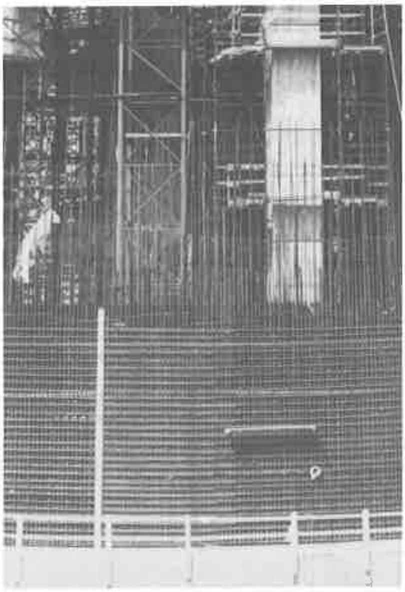

Figure 2: General View of Perimeter Wall Reinforcing. Because of heavy congestion of reinforcement, bottom 6 metres of the wall were conventionally formed.

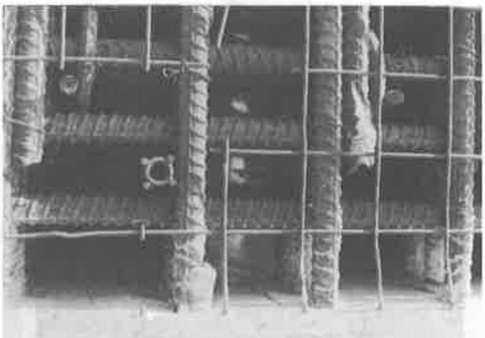

Figure 3: Detail of Wall Reinforcing at Outside Face: three layers each way. Main bars are all 35M. Horizontal and vertical post-tensioning tendons are at the middle of the wall. Wire mesh and through wall ties are also shown.

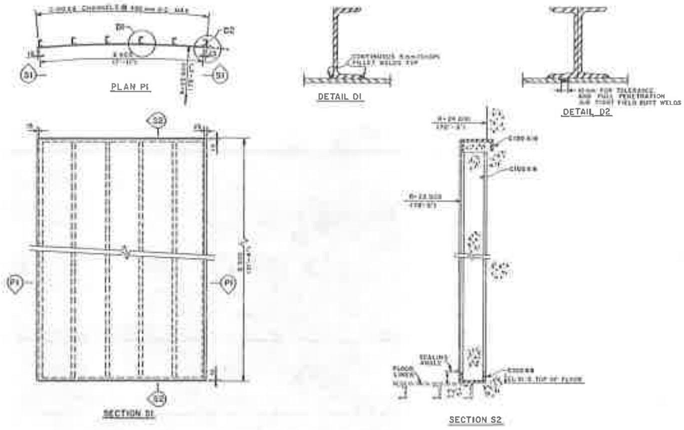

Figure 4: Carbon Steel Wall Liner: 62 shop fabricated panels, as shown, plus a narrow plate closure strip were positioned and welded together and served as formwork. The 7 mm thick plate and channels are CSA G40.21M 300 W material.

Figure 5: Slipforming of Perimeter Wall - Wall was slipformed from elevation 97.40 m to underside of ring girder at elevation 148.10 m. Internal structure columns and structural steel framing erected on top of them were used for lateral support and guidance.

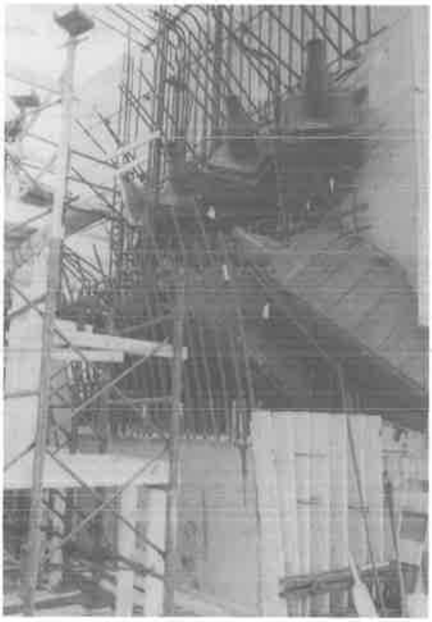

Figure 6: Full Size Ring Girder Mock Up: A "practice run" on a complex part of the ring girder was made to establish construction sequence.

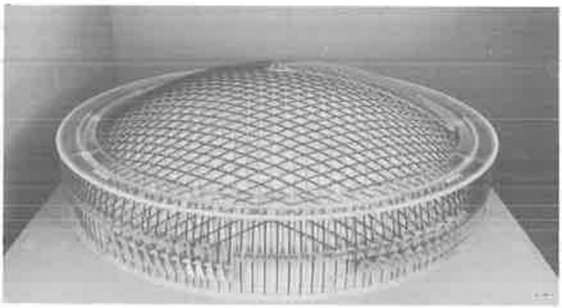

Figure 7: 1:25 Scale Model of Ring Girder and Dome provided a three dimensional check on calculations and drawings. Its use was especially helpful in positioning vertical tendons to clear dome tendons and boxouts.