Design & Construction of the Circular Cofferdam for Ventilation Building No. 6 at the Ted Williams Tunnel

The adoption of a circular cofferdam design suited the site's unusual loading requirements, made in-process design alteration easy and reduced costs.

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The first major construction contract awarded for the Massachusetts Highway Department's (MHD) Central Artery/Tunnel (CA/T) Project was for the Third Harbor Tunnel (which was subsequently named the Ted Williams Tunnel). An extension of Interstate 90, the tunnel now connects Logan International Airport in East Boston across Boston Harbor to South Boston. The contract called for an immersed tube tunnel (ITT) consisting of twelve sections, the last terminating in South Boston at Ventilation Building No. 6, approximately 600 feet inland from the shoreline.

Individual sections of the tunnel were lowered into a pre-excavated trench in the harbor and then backfilled (see Figure 1). The shoreline was excavated and the final section of the tunnel on the South Boston side was placed in a trench against the excavation support system for the ventilation building. In order to complete the connection between the buildings and the tunnel, a watertight seal was created. The building was constructed as part of a separate contract. However, excavation for the building was completed as part of the ITT Project. A unique circular cofferdam was designed and constructed to serve these functions.

Preliminary Design

The contractor was responsible for selecting an appropriate cofferdam design that was subject to criteria set forth in the project contract documents. The contract established the clear di-

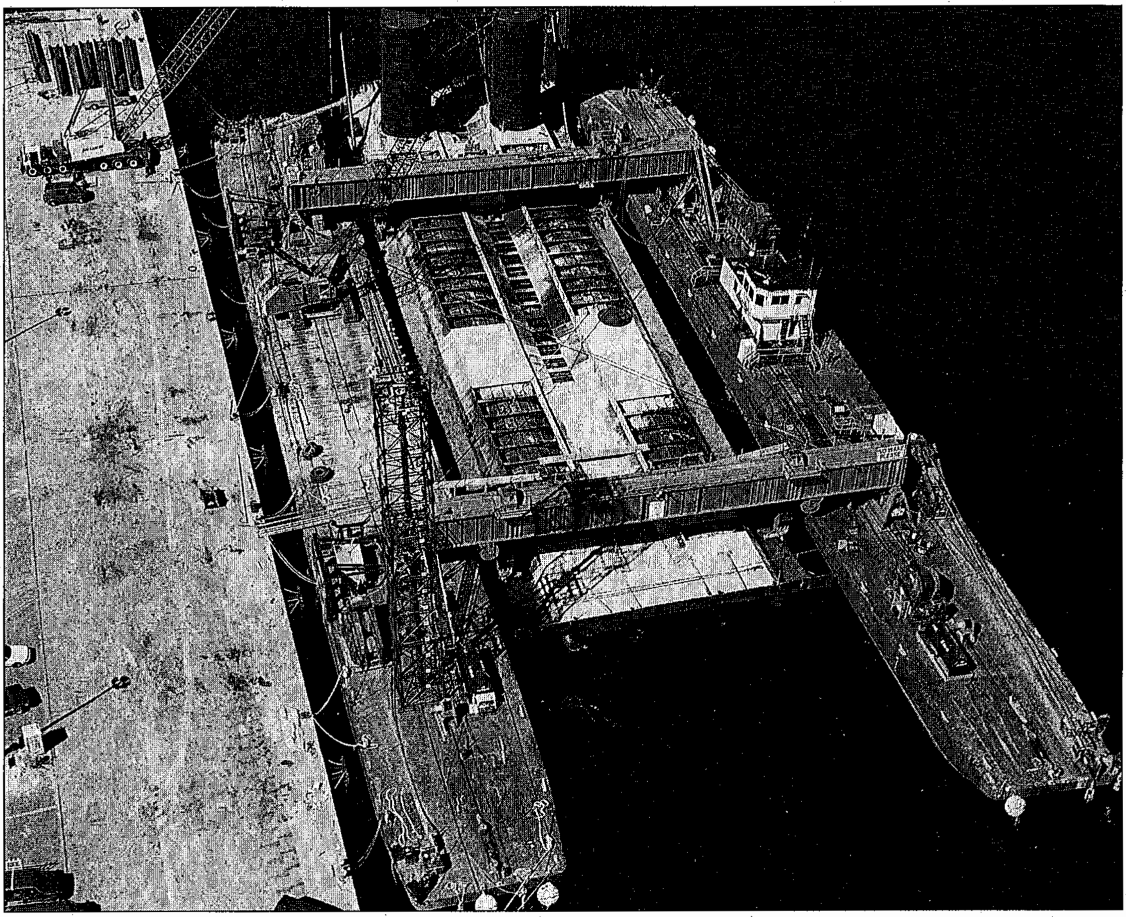

FIGURE 1. ITT Section No. 1, which was placed against the cofferdam, is supported by lay barges in South Boston. The secondary cofferdam is being erected in the background.

mensions within the cofferdam as 150 by 220 feet, and the required depth to subgrade at approximately 80 feet.

The contract documents suggested two possible support systems. The first option involved an initial dredging operation followed by the installation of eight cellular cofferdams. A sheet pile braced bulkhead was recommended to allow the tube connection to the ventilation building (see Figure 2). The second option was a cofferdam constructed of concrete-filled pipe anchor piles supporting sheet piling. This system also required an initial dredging operation and was to have been braced using internal struts for the upper levels and tiebacks in rock for the lower levels (see Figure 3).

The contractor considered several design- and construction-related issues affecting the selection of an excavation support system:

Site/Soil Conditions. The location selected for the ventilation building presented several difficulties. The site — entirely seaward for the original bulkhead line — was filled with extraneous materials. Piers erected during World War II had since been demolished and their remains left on site. For the last 20 years or so, the area had been used as a dump for building debris and foundation excavations from throughout Boston. These materials would constitute an obstruction to any system used as the support for excavation. The soil profile consisted of approximately 50 feet of these fill materials, 25 feet of Boston blue clay and 10 feet of glacial till. The till overlaid weathered argillite, which was expected to be encountered within the depth of excavation at the northern end of the building.

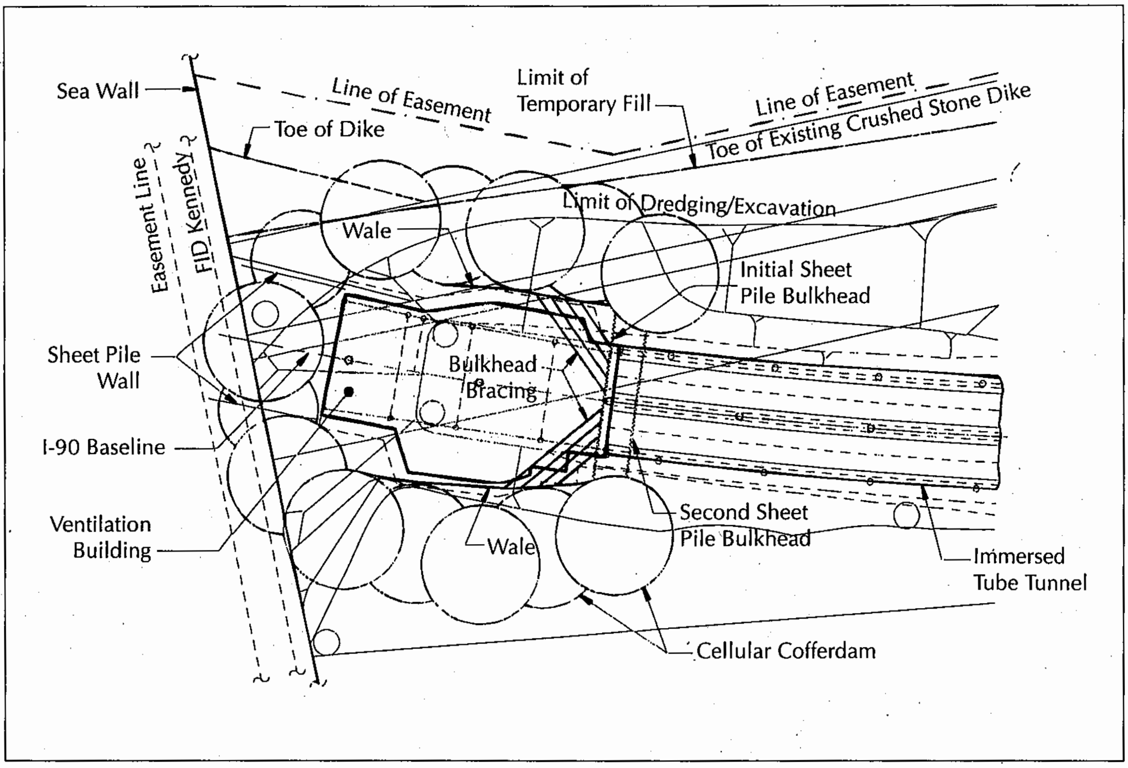

FIGURE 2. One option using cellular cofferdams with a braced sheet pile bulkhead.

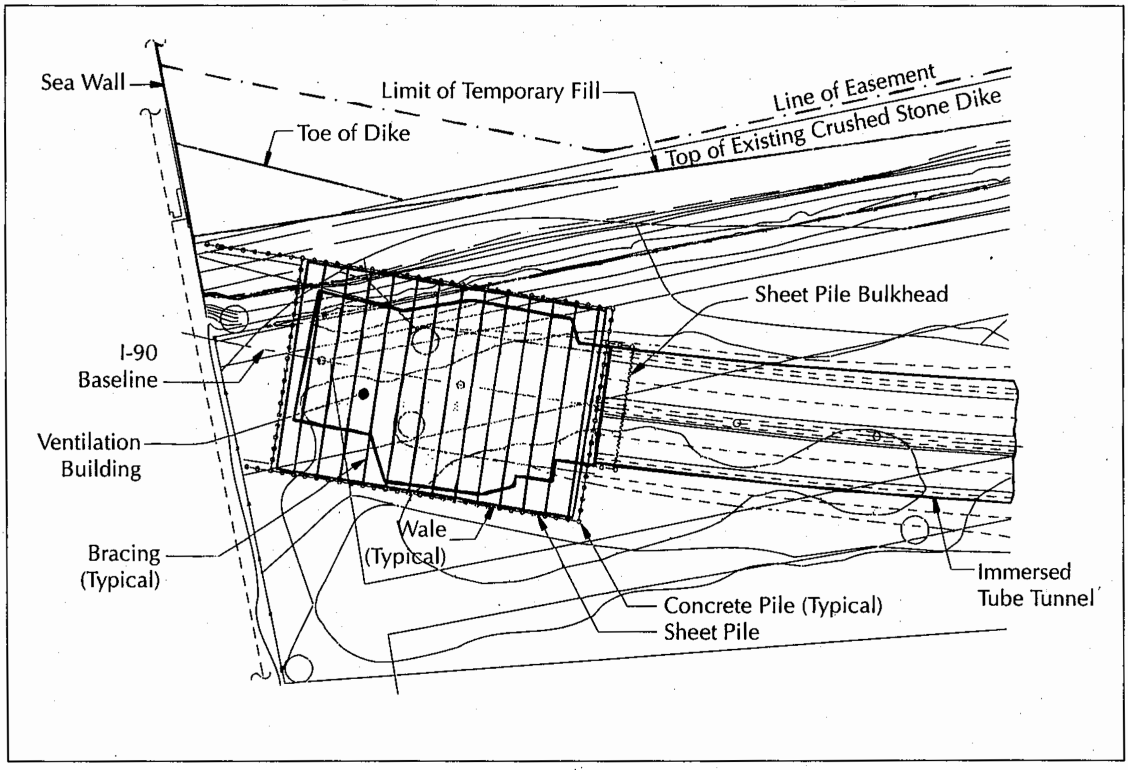

FIGURE 3. Another option using concrete-filled pipe anchor piles with sheet piling.

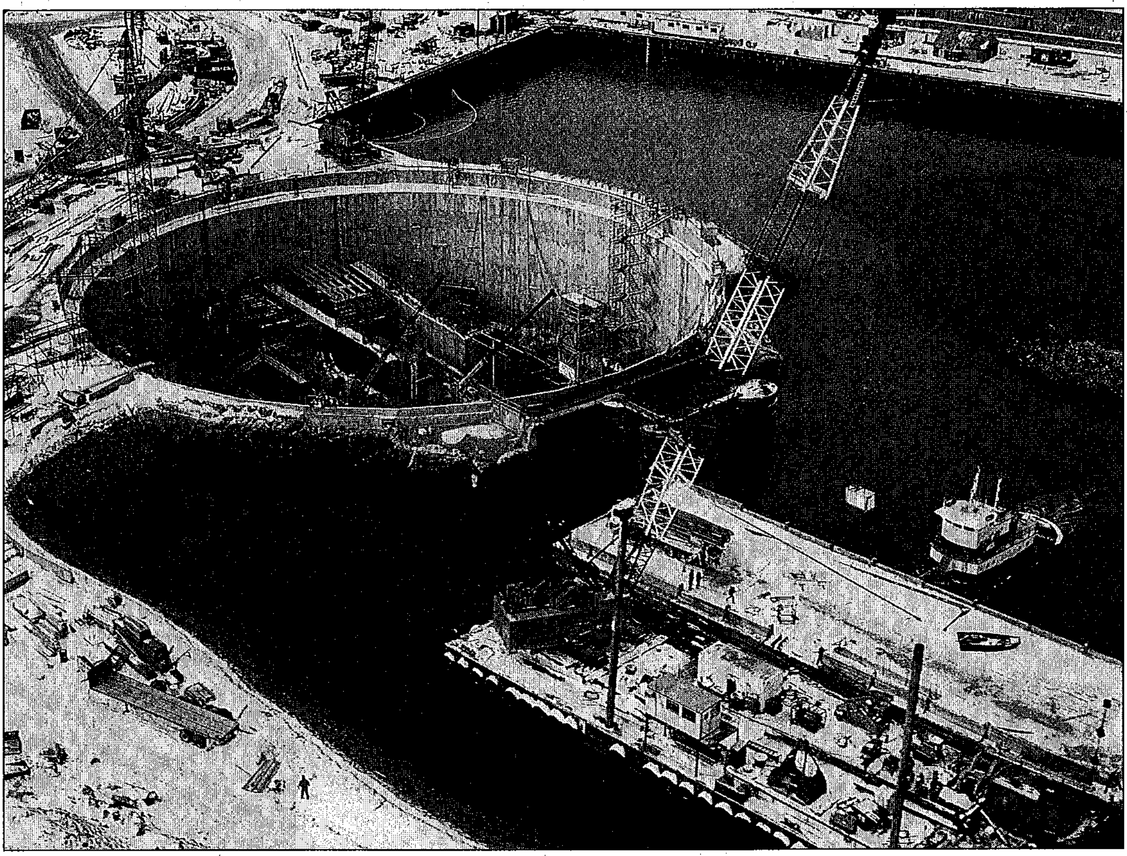

FIGURE 4. An aerial view of the cofferdam after the ITT tunnel section has been placed in the excavated trench. Note the secondary cofferdam and the circular steel access shafts for the tunnel adjacent to the cofferdam.

Ventilation Building No. 6. The building was to be constructed above and on both sides of the roadway tunnel. The ventilation and tunnel portions were designed as a monolithic structure constructed of reinforced concrete up to grade, with the above grade portion of the building constructed of structural steel. The below grade structure included four levels of framing and a number of walls and baffles in both directions to form the ventilation shafts. The foundation for the building consisted of a base mat supported by deep rock-socketed caissons with tiedowns to provide resistance to hydrostatic uplift. This complex structure would have to be built within the constraints presented by the excavation support system.

Water. The design water level was only 6 feet below grade. During the installation of the final section of the ITT, a trench would be created and the cofferdam would be exposed to the harbor. Consequently, an essentially waterproof wall would be required and would need to be socketed into the bedrock to create a seal. Since the condition of the rock was known to be variable and likely contained highly permeable seams, the system would need to anticipate possible grouting operations.

Watertight Seal for the ITT. To make the connection between the final tunnel section and the building in dry conditions, a watertight seal would need to be created between the end of the tunnel and the cofferdam.

Unbalanced Loads. To install the final section of the ITT, the soil on the north side of the cofferdam would be dredged to a depth of approximately 80 feet. Dredging would occur after the interior of the cofferdam was fully excavated. With soil and water pressure acting on the south face and only water pressure on the north, an unbalanced load of approximately 15,000 kips would be produced that would act in the direction of the proposed north opening (see Figure 4).

Penetrations of the Cofferdam. The roadway tunnel section within the cofferdam was to connect to the ITT on the north side of the building and was to continue into South Boston on the opposite end. Each end of the cofferdam would require a penetration that was roughly 40 feet high by 90 feet wide.

The contractor made an early decision to use reinforced concrete diaphragm (slurry) walls to construct the cofferdam. Selecting the bracing system was more difficult. The slurry wall system was believed to best address at least some of the issues described above. The system produces an essentially waterproof wall and can be constructed into the rock. Embedding the bracing system in rock would substantially reduce water penetration and would have the added benefit, in this case, of providing significant shear capacity that could be mobilized to resist the expected unbalanced loads. In addition, the excavation of the slurry trench would remove the debris anticipated in the layer of fill material.

The original cofferdam selected by the contractor consisted of a rectangular configuration of 3-foot-thick concrete slurry walls with internal bracing. This system was similar to the second option described in the contract documents. While planning and final design were getting under way, however, two facts became evident. First, the loads in the bracing would be very large and the unbalanced load produced by the excavation to install the adjacent tube section would make the behavior of the structure difficult to predict. The second issue was that this design would create difficult conditions for the construction of the ventilation building in a future contract. A maze of at least six levels of heavy bracing would be required, around and through which the concrete structure would have to be built. To complicate matters, access around the bracing for caisson installation would be very limited (see Figure 5).

In order to improve the conditions for the construction of the ventilation building, use of a circular cofferdam was then explored. The principal advantage of this shape is that under uniform loads it is essentially self-supporting. Lateral pressures from soil and/or water are resisted by the structure as hoop (axial) forces and produce no bending stress. Experience had been gained on recent projects using circular cofferdams constructed with slurry wall panels as the system for support of excavation. These projects included:

Charlestown Pump Station in Boston (1990). This project required a 103-foot diameter, 55-foot deep excavation. Water cut-off was achieved by extending the 3-foot thick slurry walls 45 feet below the depth of excavation into an impermeable layer of till. The bottom 30 feet of the slurry wall panels were of unreinforced concrete.

South System Pumping Station on Deer Island, Winthrop (1992). This project required a 140-foot diameter, 80-foot deep excavation. Water cut-off was achieved by extending the 3-foot thick slurry wall panels a minimum of 5 feet below the bottom of excavation into Boston blue clay.

Access Shafts at Nut Island & Deer Island (1991). This project required a 25-foot diameter, 100-foot deep excavation. The toe of the slurry wall panels was rock socketed. These 2-foot thick slurry wall panels were of unreinforced concrete.

In each of these projects, the cylindrical cofferdams were constructed using sections of straight slurry wall panels and were excavated without internal or external bracing. They were all checked for modest unbalanced loading due to non-uniform excavation within the cofferdam. The minor bending moments could be easily accommodated by the wall panels. The overall stability of the cylinder was assured by activating the passive resistance of the surrounding soil mass. Small penetrations of these cofferdams were accommodated using local areas of reinforcing.

The proposed ITT cofferdam would differ from these previous projects in three significant ways:

- The trenching required to install the final tube section would subject the ITT coffer-

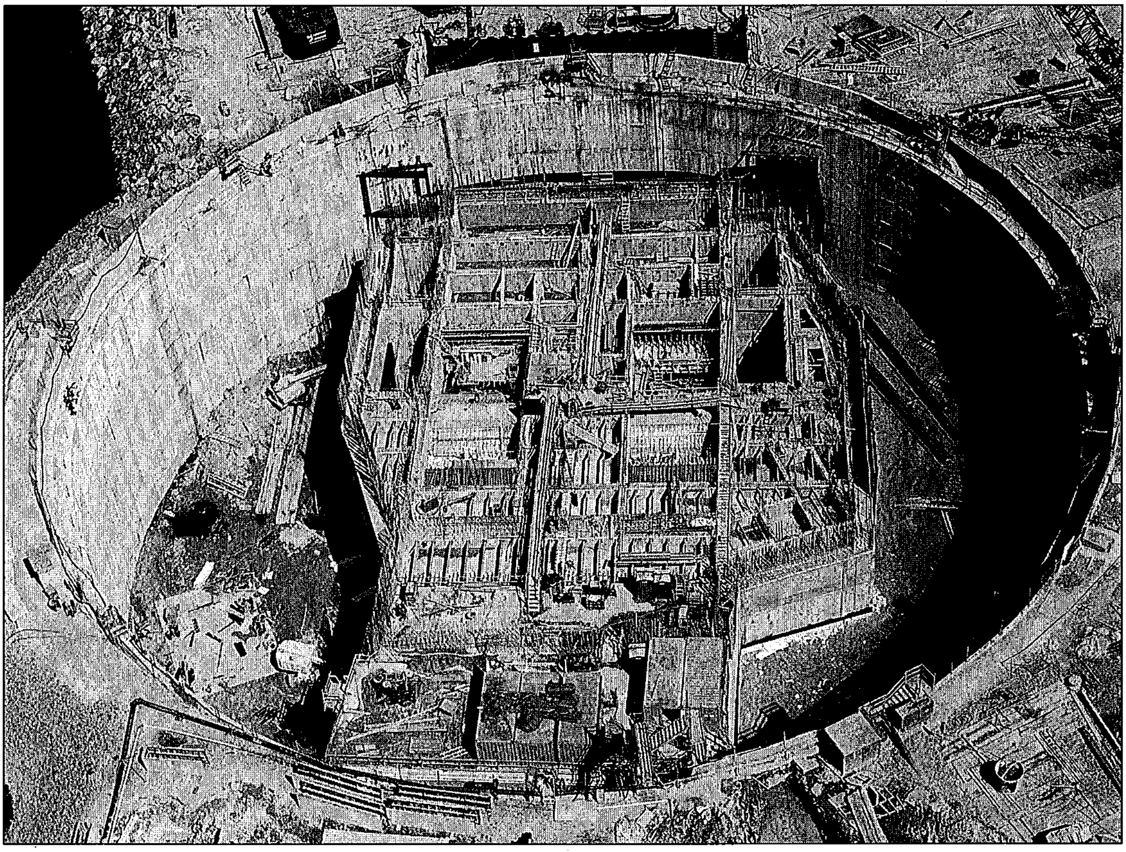

FIGURE 5. Construction of Ventilation Building No. 6 within the cofferdam.

dam to significant non-uniform loading conditions.

- The connection to the ITT and the continuation of the roadway into Boston would require two very large penetrations of the cofferdam.
- The cofferdam would have to be much larger than the earlier cofferdams.

To maintain the required clear dimensions for the ventilation building, the cofferdam would need to have a diameter of at least 250 feet. The unbalanced loads and required openings would produce significant horizontal bending stresses that could not be accommodated by the slurry wall panels. Since the axial forces in a circular cofferdam are proportional to wall pressure and to diameter, it was determined that even simple axial stress could not be accommodated with a 3-foot-thick slurry wall. Clearly, the capacities of the slurry wall would need to be supplemented.

Several methods of construction were explored. A structural system was finally selected consisting of a segmented cylinder of slurry panels with an internal concrete liner. The slurry panels were embedded in rock for lateral stability and for water cut-off. The cast-in-place concrete liner was constructed against the inside face of the slurry wall panels and was installed as individual ring beams in sequence with the excavation. Each of the ring beams were approximately 8 feet high and were constructed immediately below the previous ring beam to form a monolithic structure. The ring beams varied in thickness to respond to the horizontal bending stresses produced by the anticipated nonuniform loads and to the stress concentrations at the north opening. No internal or external bracing was required. Upon completion of preliminary analysis, the proposed system was submitted to the MHD and the concept was accepted. Figures 6 and 7 show the concept of the proposed cylindrical cofferdam.

Analysis & Design

A three-dimensional finite element model of the circular cofferdam was developed to subject the structure to a broad range of site and loading conditions, including:

- Varying the boundary conditions at the base due to differences in actual rock elevations;
- Sequencing of construction;
- Soil and rock parameters;
- Unbalanced loading due to non-uniform excavation within the cofferdam;
- Dredging operations outside the cofferdam;
- Localized dewatering outside the cofferdam; and,
- Construction surcharge in limited areas.

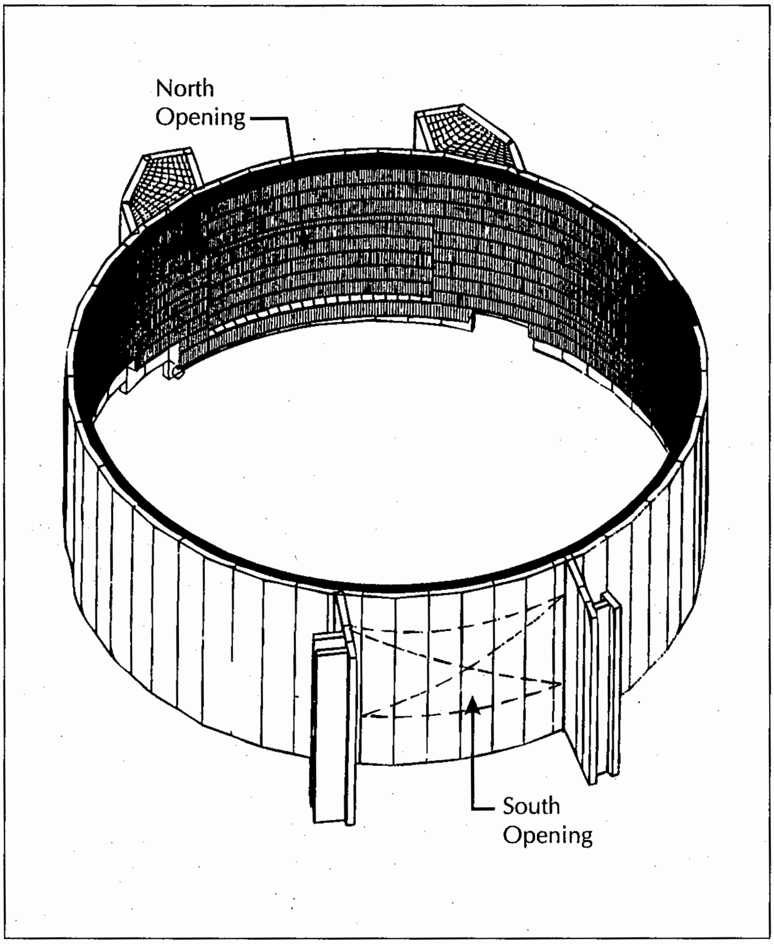

FIGURE 6. An axonometric view of the ITT cofferdam.

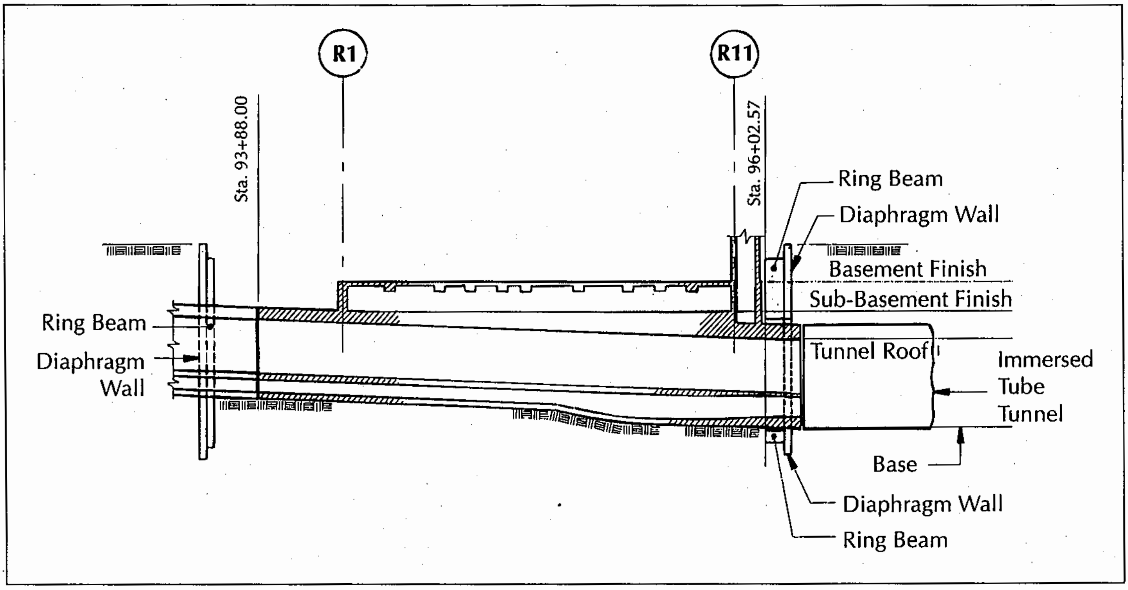

FIGURE 7. Longitudinal section through the cofferdam and Ventilation Building No. 6.

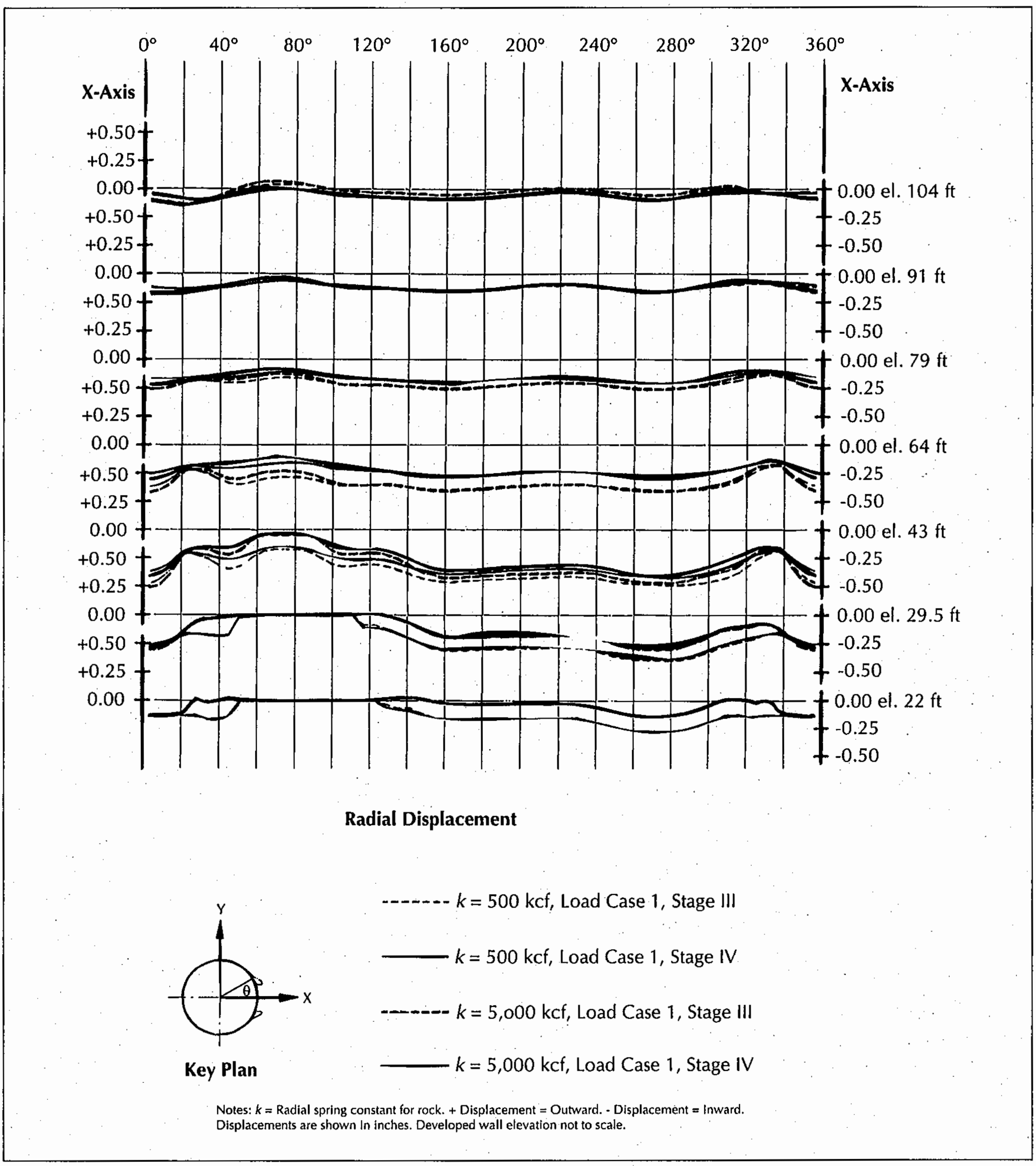

FIGURE 8. Graphical form of the results produced by post-processing program.

The analytical model was updated to incorporate field data as construction of the slurry walls and other construction activities progressed.

In order to expeditiously review and interpret the massive numerical output produced for each stage of analysis, a post-processing computer program was developed to graphically reproduce displacements and other pertinent design information. Figure 8 shows an example of this program for construction stages III and IV.

The cylindrical cofferdam consisted of two distinct components: the slurry wall panels and reinforced concrete ring beams. The methods of construction were such that the slurry wall panels would provide vertical continuity over the height of the structure but would be horizontally discontinuous since panel joints occur at roughly 26 feet on center. The circular ring beams were constructed completely at each level and provided the necessary horizontal continuity. The ring beams had a cold joint with only nominal reinforcing between each lift and, therefore, had limited vertical bending capacity.

The following assumptions were made for the load-carrying mechanisms of the cofferdam structure:

- Hoop stresses (axial compression) would be resisted by the combination of slurry walls and ring beams in proportion to their thicknesses.
- Horizontal bending would be resisted by the ring beams.
- Vertical bending would be resisted by the slurry wall.
- Buckling of the cylindrical cofferdam would be checked with the combination of the slurry wall and the ring beams.

The design requirements for the north opening of the cofferdam had always been well understood since it had to accommodate connections to the ITT. The design requirements for the south opening were less well defined because the sequence of construction for the ventilation building and the roadway into South Boston was to be the responsibility of a future contract. In fact, the contract was not awarded for the below grade portion of the ventilation building and the connecting roadway through South Boston until roughly midway into construction of the cofferdam. The ring beams were additionally reinforced in anticipation of the south opening, but the final design of the structure around this opening was completed after the cylinder was constructed. The designs for the cylinder and the north opening were completed in the initial design phase. Analysis and design for the south opening were completed in a subsequent phase.

Cylinder & North Opening

The results of the preliminary analysis confirmed that the primary mode of support for the structure was through hoop stresses (axial compression) of the cylinder. Based on this analysis, the slurry walls were sized at 3 feet thick. The minimum thickness of the ring beams was also set at 3 feet. Due to the unbalanced load during trenching operations and stress concentrations at the opening, the thickness of the ring beams varied between 3 feet outside the north opening and up to a maximum of 8 feet above and below the center of the opening. Additional vertical and horizontal stiffness was provided by buttresses on each side of the opening. The buttresses were to be the same height as the cylinder and approximately 25 by 25 feet in plan. They were also intended to act as gravity anchors in order to prevent possible sliding caused by unbalanced lateral loads. The watertight seal necessary between the cofferdam and the last section of the ITT was made at the buttresses. Figure 9 provides a summary of the cofferdam construction sequence.

The final design for the cylinder and the north opening was based on the analysis of seven stages of construction. The Stage I analysis assumed that the cylinder made of slurry wall panels was complete and excavation to 15 feet below grade had occurred prior to the construction of any ring beams. In Stages II through V, ring beams were added with the excavation (see Figure 10). Each stage of analysis roughly coincided with the construction of two additional ring beams. The loads for Stages I through V varied only with depth, but were uniform around the circumference of the cylinder. They were based on the net soil and water pressures and a construction surcharge pressure. In order to check the sensitivity of the structure to non-uniform loads, Stage IV was also analyzed assuming 15 feet of differential excavation across the width of the cofferdam. By Stage VI, the cylinder and buttresses were complete and subjected to highly unbalanced loads because of the excavation required for the installation of the final tube section. In the final Stage VII, the cylinder was penetrated to form the north opening.

The finite element model consisted of thin shell elements. The element thicknesses were modified for each stage of the analyses to correspond to the addition of ring beams. To simplify the analysis, the ring beams and slurry wall were assumed to act monolithically rather than as two distinct components. This assump-

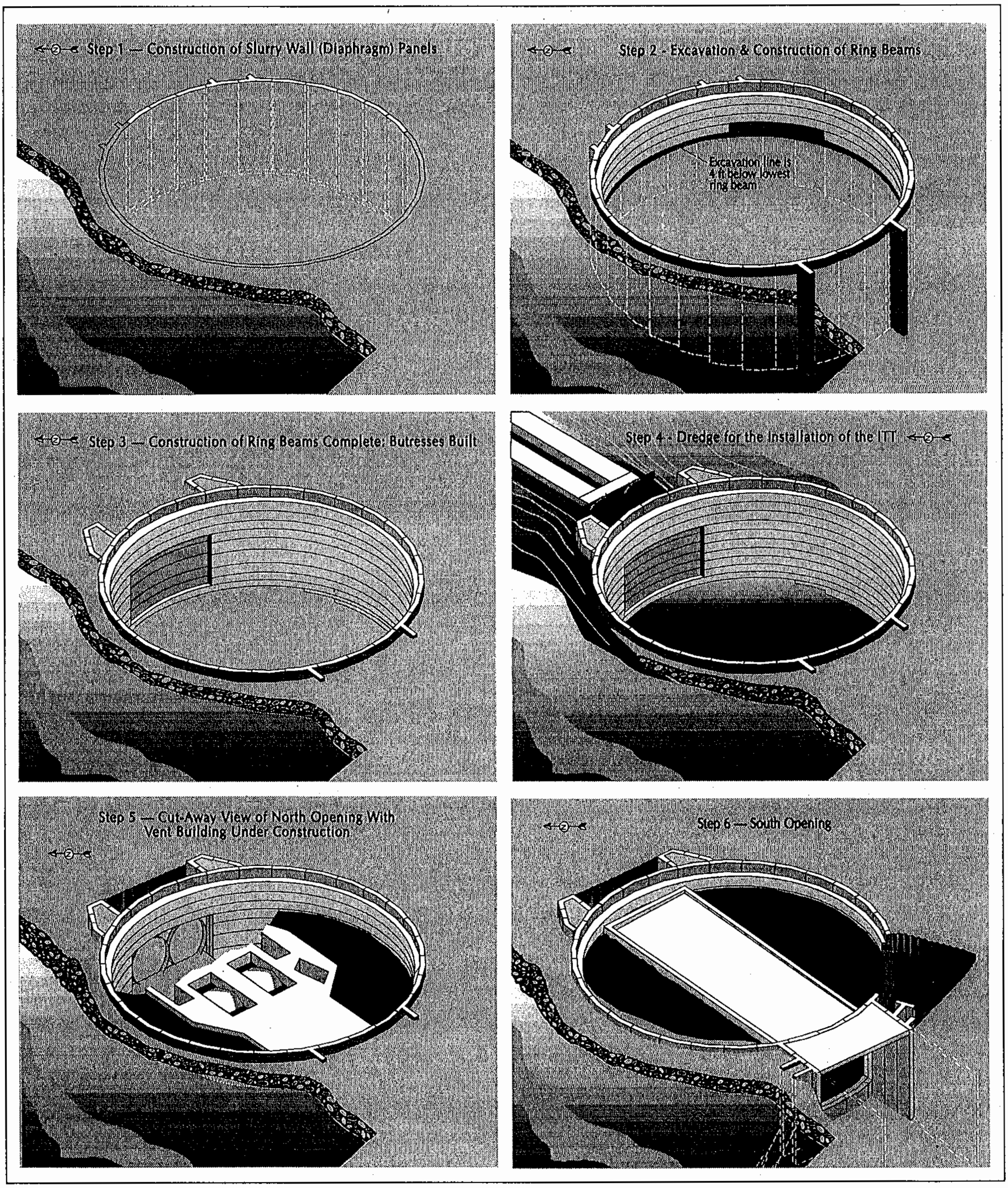

FIGURE 9. A summary of the principal steps in the cofferdam construction.

tion was reasonable since the primary load carrying mechanism was through hoop stresses that would be shared proportionately. The bending moments predicted by the model using this assumption were believed to be conservative since they would be based on elements with larger moments of inertia. This assumption was verified by a parametric study in which the axial stiffness of the structure was maintained while the moment of inertia was varied.

Support conditions in the model included rollers at the bottom of the slurry walls for vertical loads, radial spring support in the soil,

FIGURE 10. Construction of the second ring beam while excavation continues.

and radial and tangential springs in the rock. The behavior of the structure was found to be sensitive to the stiffness of the radial rock springs. For this reason, two sets of analyses were run for each stage of construction, one using a "soft" spring of 500 kcf and the other using a "stiff" spring of 5,000 kcf.

The results of the analyses indicated that axial compression in the cylinder remained the primary load-carrying mechanism during all stages of construction. The natural response of a circular structure to axial compression stress is to reduce its radius. If this movement were allowed to occur freely, no bending stresses would be generated. In this case, the toe support provided by the rock-inhibited radial shortening of the cylinder at its base, in turn, generated vertical bending moments. Moderate vertical bending moments occurred during Stages I through V, which were resisted by the slurry wall panels (see Figure 11). This rock/structure interaction was directly affected by the assumed stiffness of the radial rock springs with larger vertical bending moments occurring when the rock stiffness increases. While horizontal bending moments were insignificant during these stages, they became very important in resolving the unbalanced loads in Stage VI as well as in the stress concentrations produced at the north opening in the final stage. Maximum horizontal bending stresses occurred above and below the center of the opening in Stage VII (see Figure 12). In-plane shear stresses were found to be negligible except in the final stage when axial forces in the cylinder were shifted to above and below the opening. See Figure 13 for the deformed shape of the finite element model for the final stage.

South Opening

Work on the design for the south opening was not begun until the construction of the cofferdam cylinder was near completion. The ring beams were provided with additional reinforc-

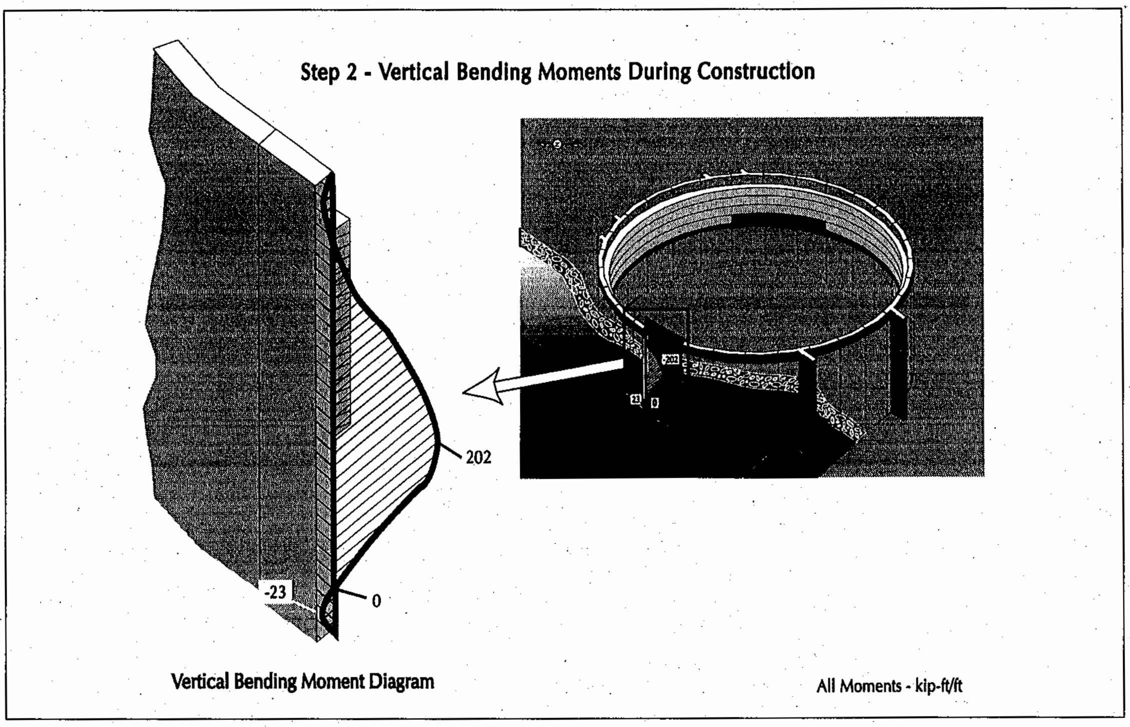

FIGURE 11. Vertical bending moments for a typical slurry wall panel predicted by analysis during construction Stage IV.

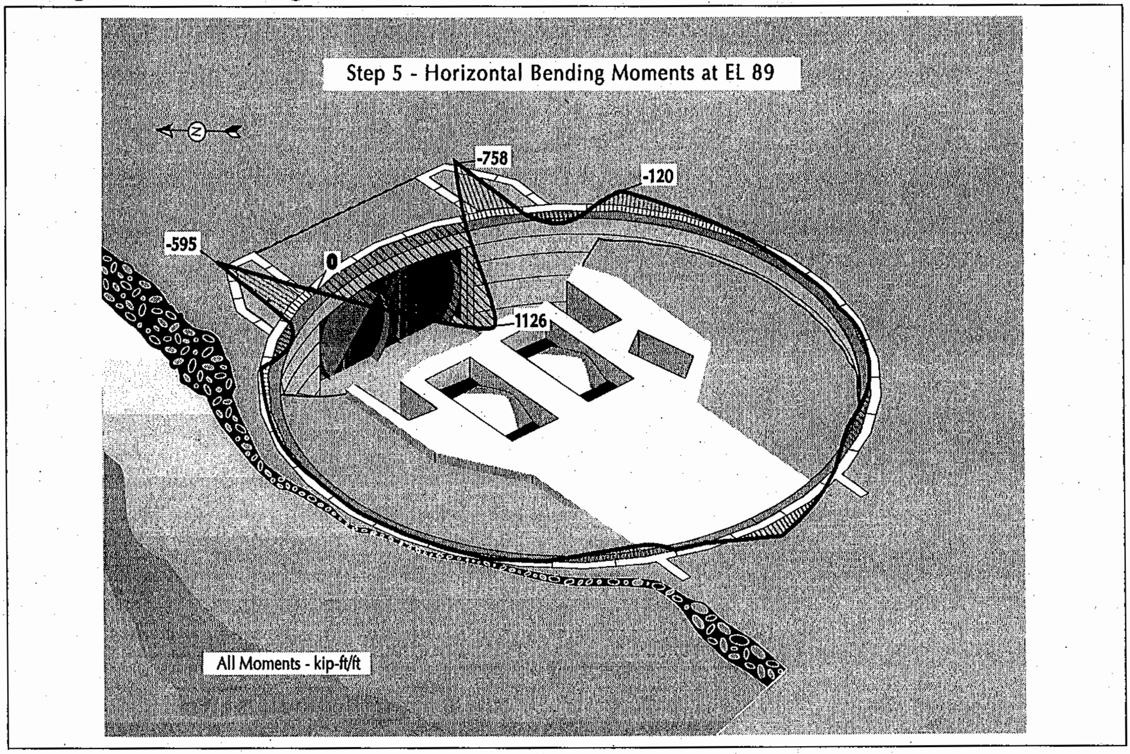

FIGURE 12. Horizontal bending moments predicted by analysis for the ring beams above the north opening.

ing steel in the area of the south opening in anticipation of final resolution. However, the ring beams had been maintained at the minimum thickness of 3 feet and buttresses similar to those provided on the north side had not been possible. The cofferdam cylinder, therefore, had a fixed and somewhat limited capacity. As a complication to the design, the top 20 feet of the cofferdam had to be removed in one area to accommodate an extension of the ventilation building. This notch was approximately 75 feet wide and was located immediately adjacent to the eastern edge of the south opening (see Figure 9).

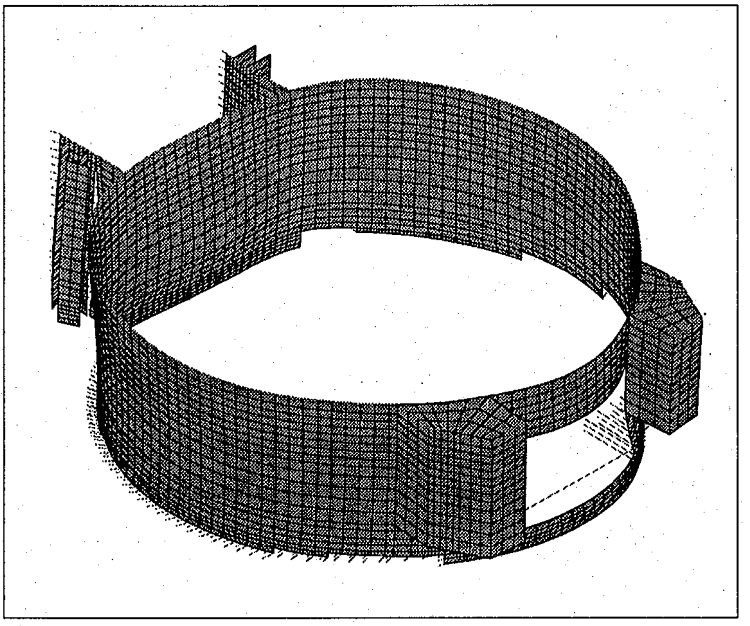

FIGURE 13. Deformed shape of the finite element model for the north opening.

In order to keep the design requirements within the fixed capacity of the cofferdam cylinder, several approaches to the construction of the opening were considered. An approach was adopted that required the least modification to the cofferdam structure. It was agreed that the construction of the ventilation building would precede creating the opening or notch. After the roadway tunnel portion of the building was complete, the areas between the building and the cofferdam would be backfilled up to within 16 feet of grade (although the water level would be maintained at the level of the subgrade). The soil backfill had the beneficial effect of reducing the net pressure on the cofferdam by about one -third. The South Boston roadway adjacent to the cofferdam was to be constructed as a cut-and-cover tunnel, using slurry walls for the support of excavation. Approximately 40 feet of these walls were tied into the south face of the cylinder and were used to stiffen the edges of the opening. These "wing walls" served much the same function as the buttresses on the north side. A cap beam was cast on top of each wing wall and vertical tiedowns were installed in each slurry wall panel to compensate for the lack of mass.

The effect of the notch was to drastically reduce the horizontal bending capacity of the cofferdam. The elevation of the bottom of the notch was less than 10 feet above the top of the opening. The ring beam immediately above the opening would be continuous below the notch, but the other ring beams above the opening would be interrupted at the edge of the notch. The reduced horizontal bending capacity was compensated for by constructing a 40-foot wide waling slab between the wing walls and against the south face of the cofferdam cylinder. The slab was installed at roughly mid-height between grade and the opening and a waling beam was constructed immediately above the opening. The concrete slab and beam were formed on grade and installed in sequence with the general excavation for the cut-and-cover tunnel (see Figure 14).

The south opening and notch could be created only after the internal backfill was placed and the wing walls, waling slab and waling beam were complete. Even so, certain areas of the cofferdam cylinder were subject to high bending stresses. For these areas, a moment redistribution was performed and the overall capacity of the cylinder was found to be sufficient.

Secondary Cofferdam

The final section of the immersed tube tunnel

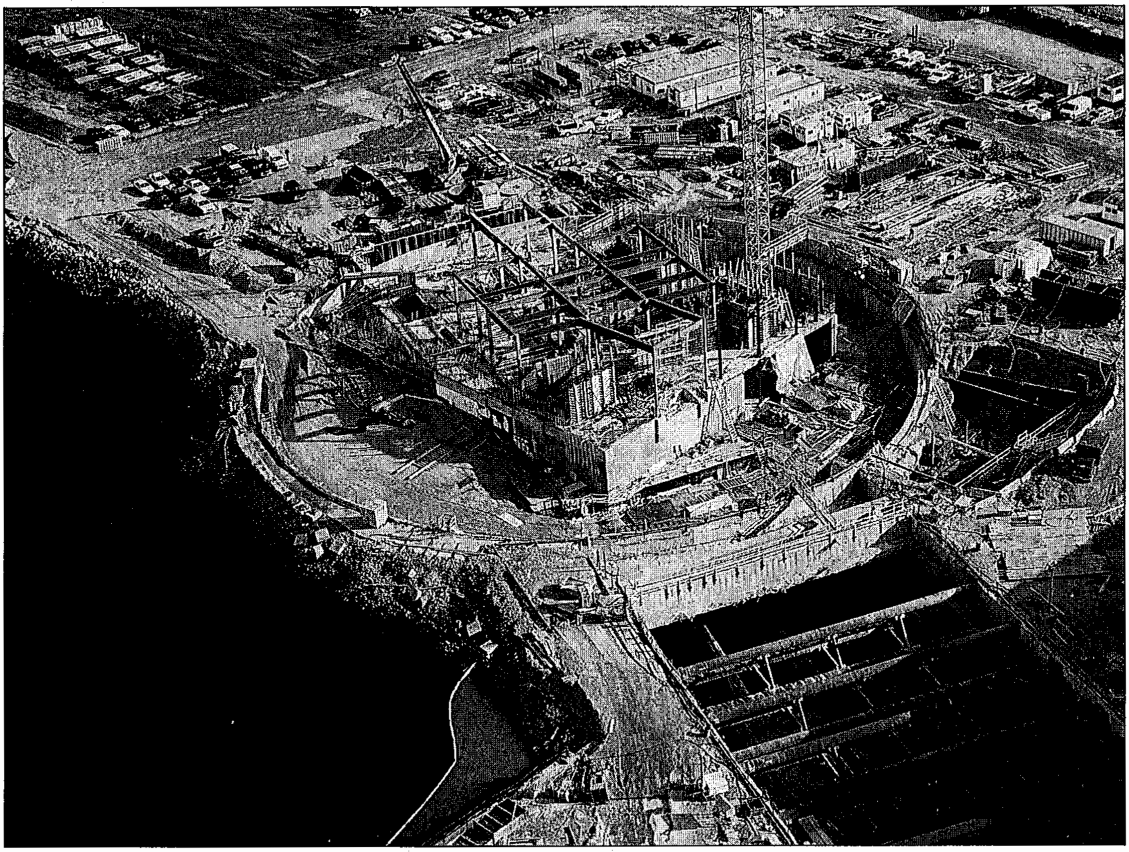

FIGURE 14. Construction of the ventilation building. The south opening and notch are in the foreground and to the right.

was placed in a trench between the buttresses and within four feet of the outside of the cofferdam cylinder. In order to make the final connection between the ITT and the roadway below the ventilation building, a secondary cofferdam was required that would allow the space to be pumped dry. The main component of the secondary cofferdam was steel sheet box piles placed on top of the ITT and extended up to grade. The box piles were laterally supported at the top of the tunnel and braced to the cofferdam cylinder. The challenge was to create a flexible watertight seal between the sides of the ITT and the buttresses that would accommodate thermal- and pressure-induced tunnel movements.

The watertight seal was achieved using two hinged steel gates that spanned between the tunnel and the buttresses. Each gate was connected by a hinge to the face of the tunnel section and to vertical extensions of the tunnel up to grade. The gates consisted of steel wide flange beams oriented in a horizontal position and a continuous vertical steel plate. The beams were spaced at approximately 4 feet on center and a hinge created by installing a steel pin through the web of each beam and through a seat bracketed off the tunnel. A continuous vertical rubber seal was attached to the opposite end of the gate and positioned against a steel channel embedded in each buttress. The tunnel extensions were 4-foot-square steel towers. One face of each tower aligned with the side of the tunnel. Each tower also acted as the terminus for one end of the steel sheet box piling. The towers were supported by, and laterally braced to, the tunnel (see Figures 15 and 16).

The sheet piling, towers and gates were assembled onto the tunnel section during the final fitting-out process in South Boston (see Figure 1). In order to facilitate transportation, the gates were temporarily swung into an "open" position and anchored to the face of the tunnel. The tunnel section was then floated into position against the cofferdam and lowered into the trench. The gates were swung to their "closed" position against the buttresses, and the sheet piling placed braced against the cofferdam. Backfill was then placed on top of the tunnel and against the gate and the sheet-pile wall, creating a net positive pressure that activated the rubber seal of the gate. All this work had been completed under wet conditions. The space between the secondary and primary cofferdams could then be pumped dry. The primary cofferdam was penetrated to create the north opening, thereby allowing final connection between the roadway below the ventilation building and the ITT.

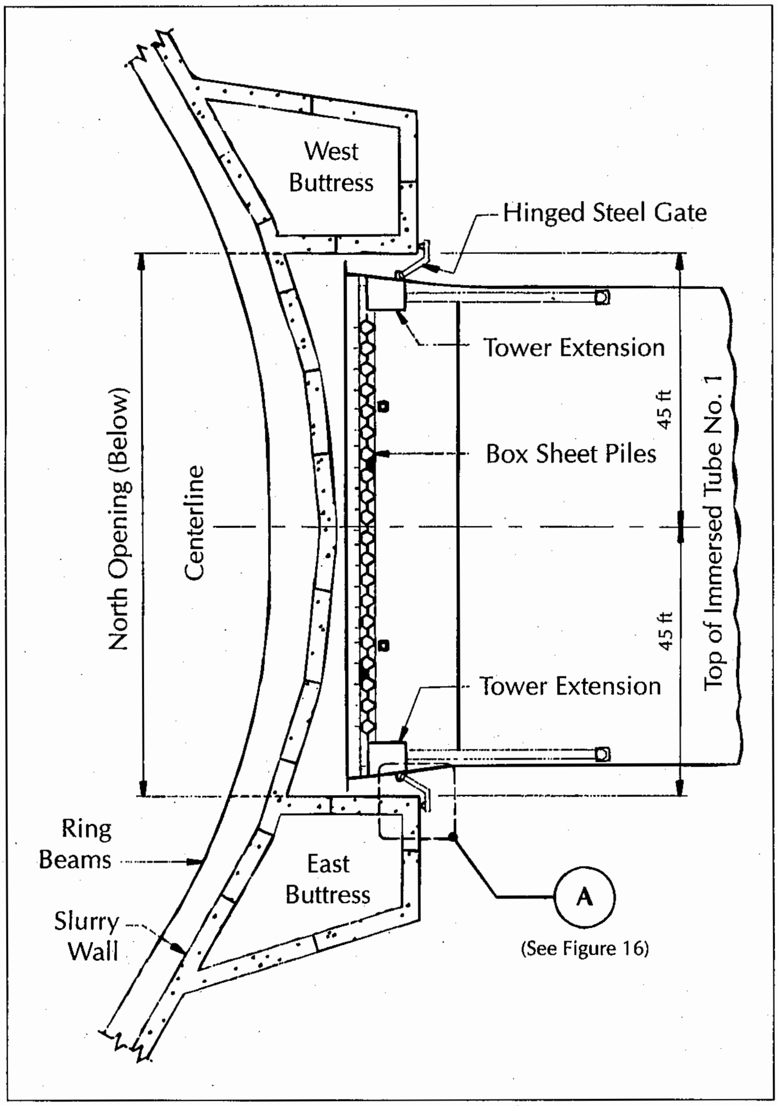

FIGURE 15. Plan view of the secondary cofferdam adjacent to the main cofferdam.

Construction

The slurry wall method involves an excavation operation that includes removing obstructions. To ensure that the debris could be removed, a pre-trenching phase of the excavation was performed. The full 50-foot depth of debris was excavated using a large backhoe with an extended boom. The cavity created by this operation was backfilled with an engineered material consisting of fine and course aggregate, cement, fly ash and bentonite. The 800-foot circumference of the cofferdam was pre-excavated and backfilled prior to commencing slurry wall construction.

Installing the slurry wall through the engineered fill, clay, glacial till and into rock was accomplished using conventional slurry wall construction methods. The wall was extended between 3 and 5 feet into competent rock to achieve a water cut-off. Rock was removed using heavy star chisels and conventional grab buckets. A total of 32 wall panels and 10 buttress panels were constructed. The average depth of panel was 80 feet, while the average panel length was 24 feet, bringing the total area of the slurry wall constructed to 75,000 square feet (see Figure 17).

Excavation within the cofferdam began after the entire cylinder of slurry wall panels was complete. A total of 10 concrete compression ring beams were installed as the excavation proceeded to subgrade. Transverse shear keys

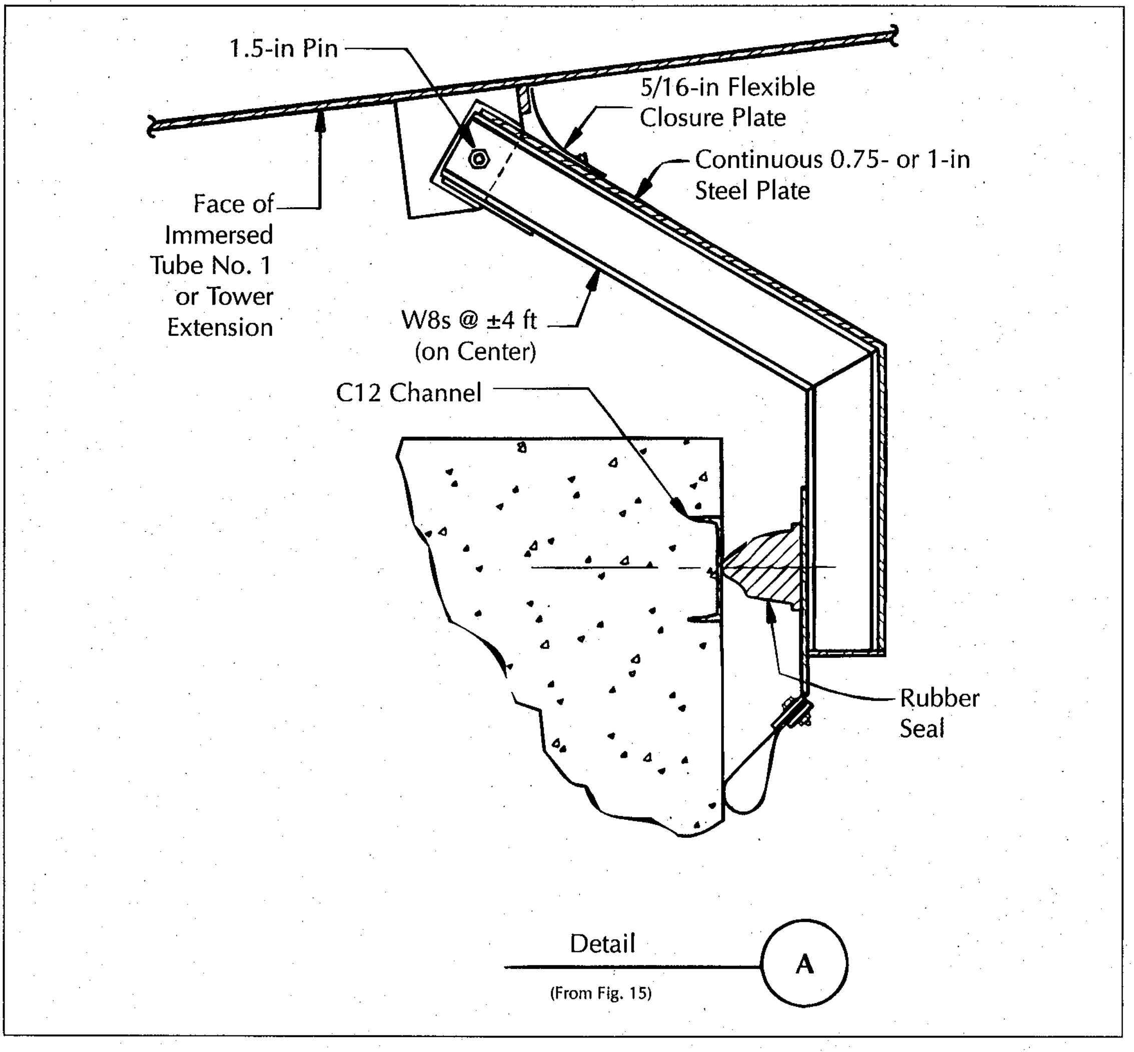

FIGURE 16. Secondary cofferdam — plan detail A of the hinged gate.

were formed in the soffit of all ring beams to ensure the transfer of in-plane shear. This capacity was locally enhanced with reinforcing steel dowels installed between ring beams on both sides of the openings (see Figure 18).

The inflow of water over the height of the cylinder was negligible during the excavation and construction of the building due to the naturally low permeability of the slurry wall panels, which was enhanced by the axial compression in the panels and across the slurry wall joints. A very effective water cut-off was achieved between the bottom of the slurry wall and the rock. Only two areas required minor grouting at the wall/rock interface to stop a minimal inflow of water. The cofferdam subgrade was remarkably dry and only two small electric pumps were necessary to handle the inflow (see Figure 19).

Additional analysis and redesign of the cofferdam occurred throughout the construction process. The top of rock elevations and the depth of slurry wall panels were adjusted in the finite element model as the as-built information became available. Information on actual field conditions prompted, among other adjustments, a change in the construction process for the north opening.

The design of the cofferdam assumed that the buttresses would contribute to the resistance of unbalanced lateral forces such as those created by the dredged condition north of the cofferdam. The assumed lateral resistance would also have had the effect of reducing the horizontal bending moments above, and especially below, the north opening. In order to place the final tunnel section against the cofferdam, a limited area of rock had been removed during the dredging operation that had required blasting. After excavating to subgrade within the cofferdam, the actual condition of the rock below the buttresses was found to be much worse than anticipated due to the poor condition of the on-site rock, which had been worsened by the blasting. The cofferdam had behaved as anticipated throughout the stages of construction even for the dredged condition (construction Stage VI in the analysis). Through additional analysis, this performance was attributed to the bearing and shear capacity of the rock outside the blast-damaged areas.

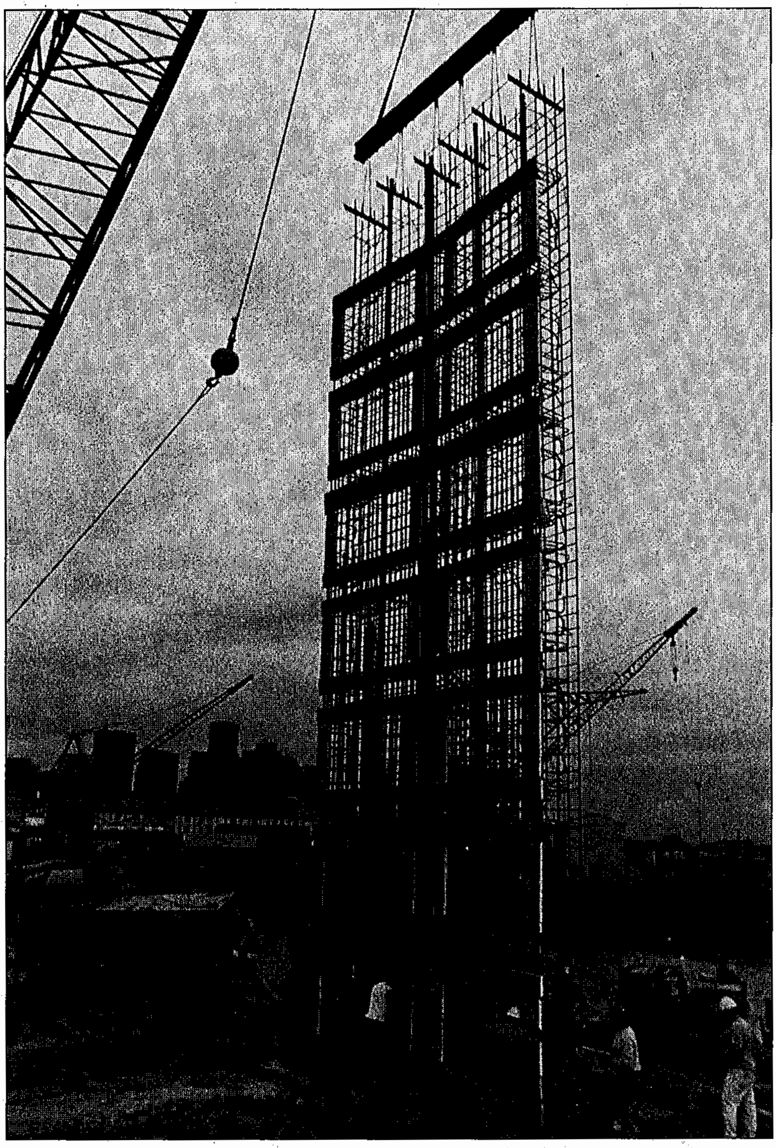

FIGURE 17. Installation of a reinforcing steel cage for a slurry wall panel. Each set of formed blockouts provides shear capacity between the slurry wall panel and one ring beam.

The final stage was reanalyzed assuming that the buttresses were supported on rollers. This analysis showed that the horizontal bending moments below the opening would exceed the capacity of the ring beam. The concern was that the sides of the opening would tend to close together. Further analysis suggested that the solution was to create the north opening in two steps.

After the space between the primary and secondary cofferdams was pumped dry, the first step was to demolish the lowest 8 feet of concrete at the bottom of the proposed opening and to construct the bottom half of the roadway base slab through the slot created. The slab was used as a compression strut between the buttresses and was preloaded by jacking against the cylinder. This system effectively eliminated the bending problem. The remainder of the concrete at the opening was demolished during the second step. The clear width of the opening was monitored throughout the demolition process and the net movement was measured at less than 0.125 inch.

The behavior of the cofferdam was monitored throughout the construction process by reading inclinometers embedded in the slurry

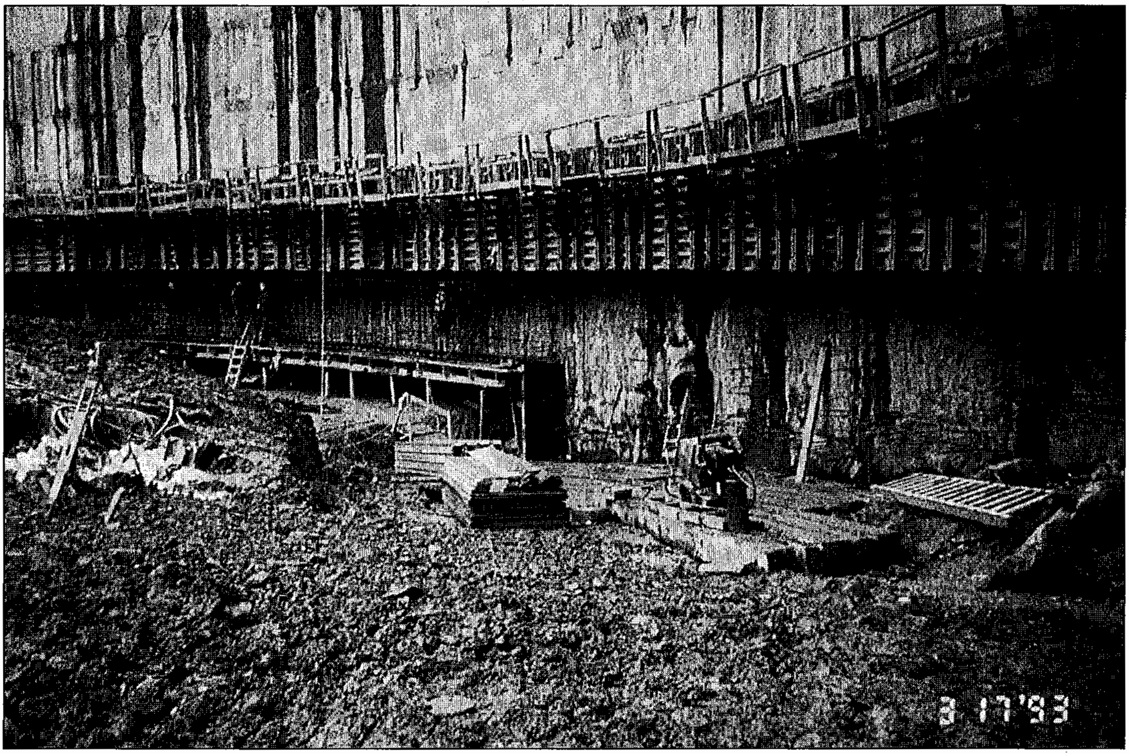

FIGURE 18. Construction of a ring beam.

wall panels. The radial deflection of the cylinder was read after the first stage of excavation and found to be much greater than anticipated. Review of these data suggested that this behavior was not induced by stress, but rather by the natural shrinkage of the reinforced concrete slurry wall panels during curing. The overall behavior of the cofferdam was found to be consistent with that predicted by the finite element model for the remaining states of construction when corrected for concrete shrinkage (see Figure 20 on page 50).

Conclusions

The ITT cofferdam, at 250 feet in diameter, is among the largest structures of its type ever built. The two very large penetrations required to accommodate the connection to the ITT and the continuation of the roadway into South Boston are unique features of the design. These openings constituted discontinuities in the cylindrical structure and, together with the large unbalanced loading conditions, required that multiple load-carrying mechanisms be available. This need was met by using the combination of vertical slurry wall panels and cast-in-place concrete ring beams. Due to these redundant load paths, the structure has been shown to readily accommodate a variety of unforseen and as-built conditions, including the re-design of the cofferdam to accept the south opening and the modifications at the north opening necessitated by the poor base conditions at the buttresses. These, and other, design modifications were implemented during the construction process without significantly affecting the overall behavior of the structure. Furthermore, the use of the slurry wall method of construction allowed the contractor to build a stable, water-tight structure prior to any excavation and provided a shear key between the cylinder and bedrock.

Use of the circular cofferdam provided distinct advantages to the project. The multiple capacities of the structure were well matched to the set of unusual loading requirements and site conditions. In addition, conventional braced excavations would have made construction of the ventilation building very difficult and, therefore, expensive. Net cost savings to

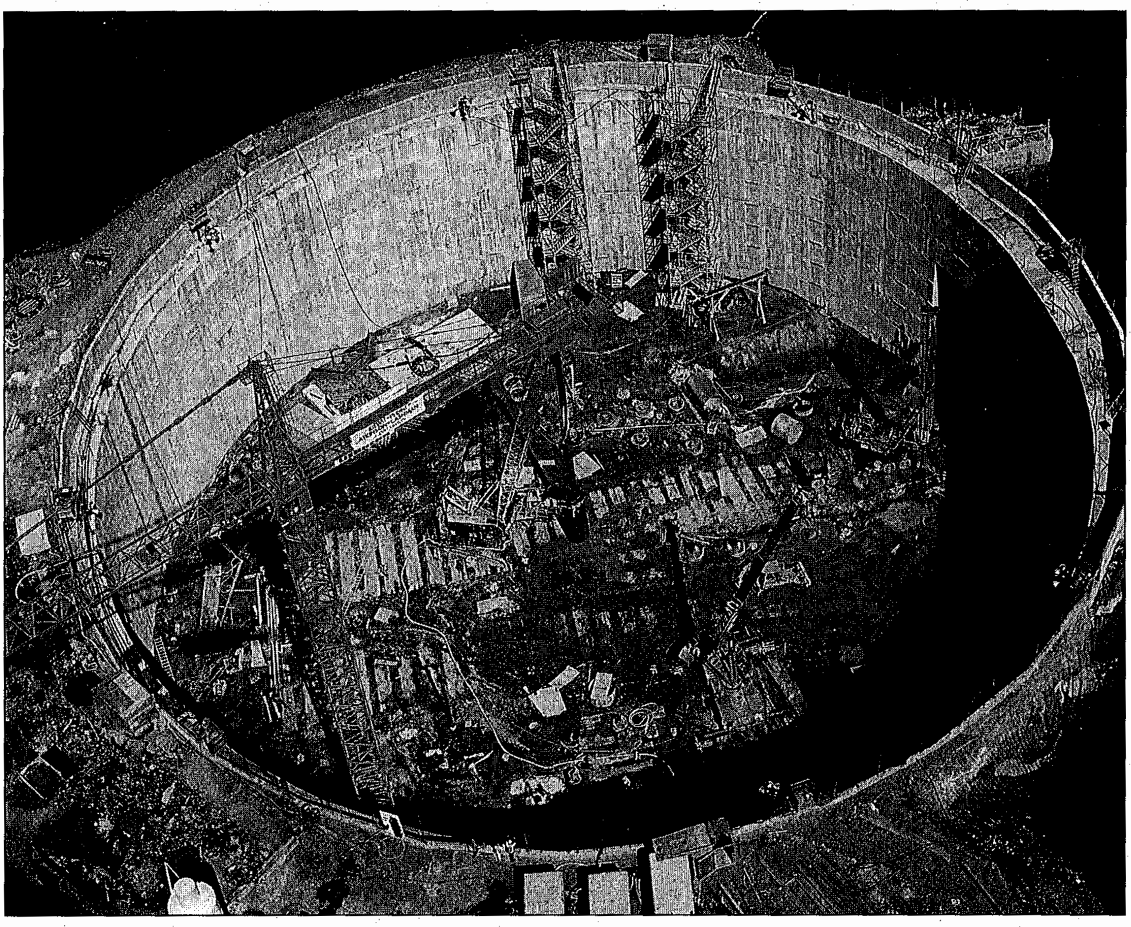

FIGURE 19. Construction of the cofferdam cylinder is complete. The outline of the varying ring beam thicknesses indicates the north opening is to the right. Installation for the ventilation building caissons has begun.

the CA/T Project achieved by the ITT cofferdam was estimated to be $4 million compared to the costs of conventional systems. The success of the cofferdam project suggests that structures of this type have the potential for a wide range of applications when customized to specific project conditions.

Pre-trenching operations for the cofferdam cylinder were begun in March 1992. The cylinder construction, including excavation to subgrade, was substantially complete in the summer of 1993. Roadway and ventilation building construction continued through the summer of 1995.

ACKNOWLEDGMENTS – *The project owner is the Massachusetts Highway Department. Management Consultant for the project is Bechtel/Parsons Brinckerhoff. Sverdrup Civil, Inc., was the immersed tube tunnel designer. Ventilation Building No. 6 designer was HDR Engineering, Inc. The ITT contractor (C05A1) was a joint venture of Morrison Knudsen, Interbeton and JF White. JF White was responsible for design and construction of the ITT cofferdam. Tunnel and below grade construction of Ventilation Building No. 6 (C04A2) were performed by Kiewit, Perini, Atkinson, Cashman – a joint venture. Above grade construction of Ventilation Building No. 6 (C04A3) was performed by Walsh Construction Co. Structural design of the ITT cofferdam was by Weidlinger Associates, Inc. The geotechnical consultant for the ITT cofferdam was GEI Consultants, Inc. The figures were prepared by Penny Regenos, Charles Hamlin, Julie Sant and Dave Dolan – all of Weidlinger Associates, Inc. Photographs were prepared by Abbott-Boyle, Inc., construction photographers.*

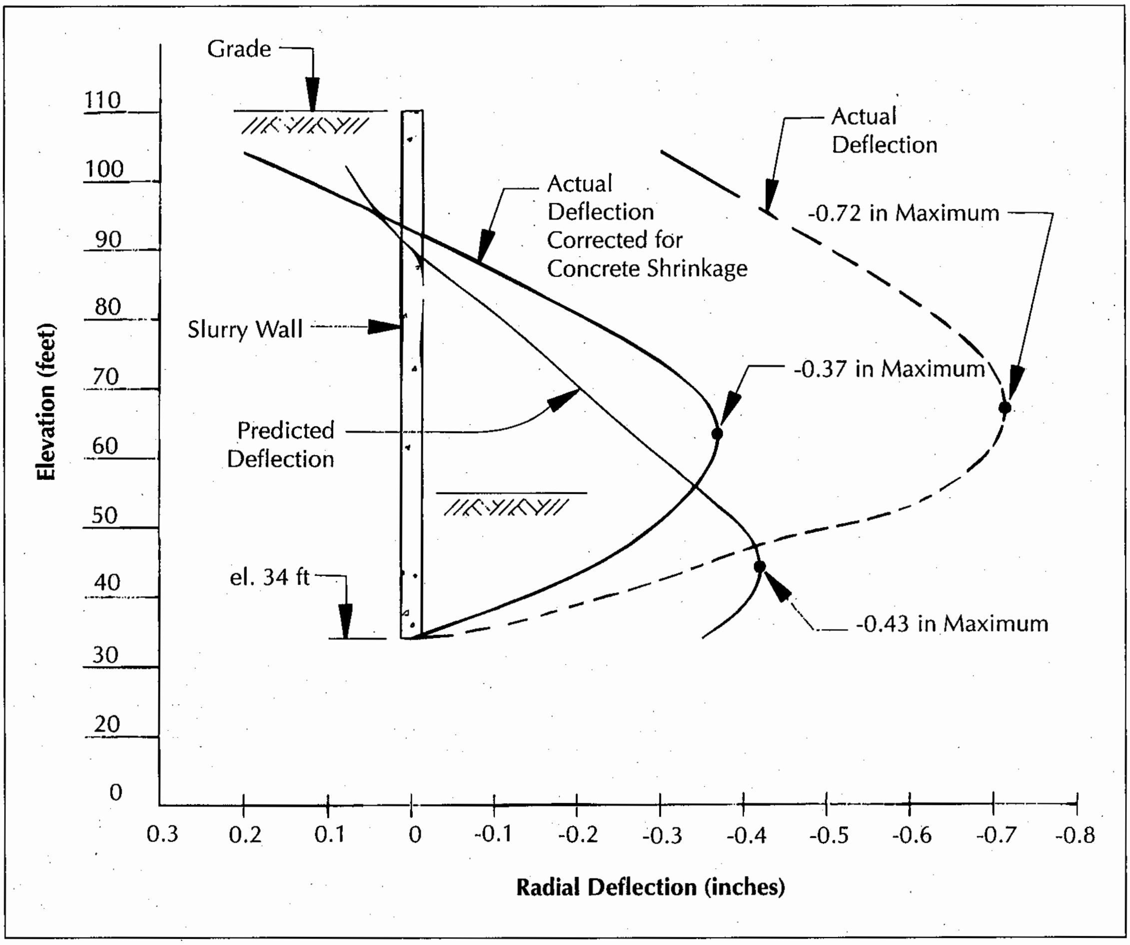

FIGURE 20. Actual versus predicated wall movements for construction Stage IV.

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