Boonton Dam: Partnering in a Conventional Bid Climate

by Steve H. Snider, P.E. and Donald A. Bruce, Ph.D.

Mr. Snider is a managing engineer with O'Brien & Gere Engineers, Inc., Syracuse, New York. He holds a BS in civil engineering from Clarkson College of Technology and Engineering. His technical expertise is in the areas of geotechnical investigations, evaluations and remediation designs; and materials engineering. Dr. Donald Bruce is Vice President and Technical Director for Nicholson Construction Company, Bridgeville, Pennsylvania and has worked on major earth retention projects throughout the world. He holds membership on several international technical committees involving ground engineering and dam safety.

Boonton Dam is located on the Rockaway River in northwestern New Jersey. The dam is owned and operated by the City of Jersey City and impounds the city's principal raw water supply. The structure consists of a 115-foot high, 2,150-foot long cyclopean concrete masonry gravity section with 500-foot long earth embankments extending to both abutments. The gravity dam crest is 17 feet wide. A 300 foot long spillway equipped with bascule crest gates is located near the northern (left) end of the gravity section. A masonry multistage intake structure abutting the upstream face of the gravity section provides drawdown capability and release into a 9-foot diameter, 24-mile long aqueduct via four, 48-inch diameter conduits.

Project History

The dam's masonry (214,000 CY) was placed in 15 months between May 1892 and November 1894. A contemporary article from **Engineering News Record** reported that the dam was "one of the twenty-four great dams of the world", and the first major structure to use American made Portland cement furnished by the Atlas Cement Company.

The physical plant used to build this structure is quite impressive, emphasizing the magnitude of this undertaking at the turn of the century. **Engineering News Record** reported that 35 stiff-legged and gut derricks were each in operation at the quarry and damsite. Timber trestle towers carrying two derricks were erected strategically upon the down stream face to place the uppermost sections of the structure. A central boilerhouse containing two steam boilers was used to supply steam for the "lifting engines" and seven concrete mixers. A standard gauge railroad spur was built from the quarry to the damsite.

The dam was built by raising upstream and downstream mortared cut stone block (ashlar) or selected rubble walls ahead of the interior. The syenite (granite) ashlar and rubble was hauled to the placement by steam train from a quarry four miles away. The interior (hearting) was filled with cyclopean masonry consisting of large blocks of irregular stones embedded in, and surrounded by concrete. The concrete was mixed "very wet" and placed between the facing walls. The 1/4 to 4 cubic yard quarry stones were washed upon arrival, lifted by stream-powered derrick, dropped closely together into the concrete and thoroughly shaken to insure proper bedding. The spaces between the large stones were filled with concrete and quarry spalls to

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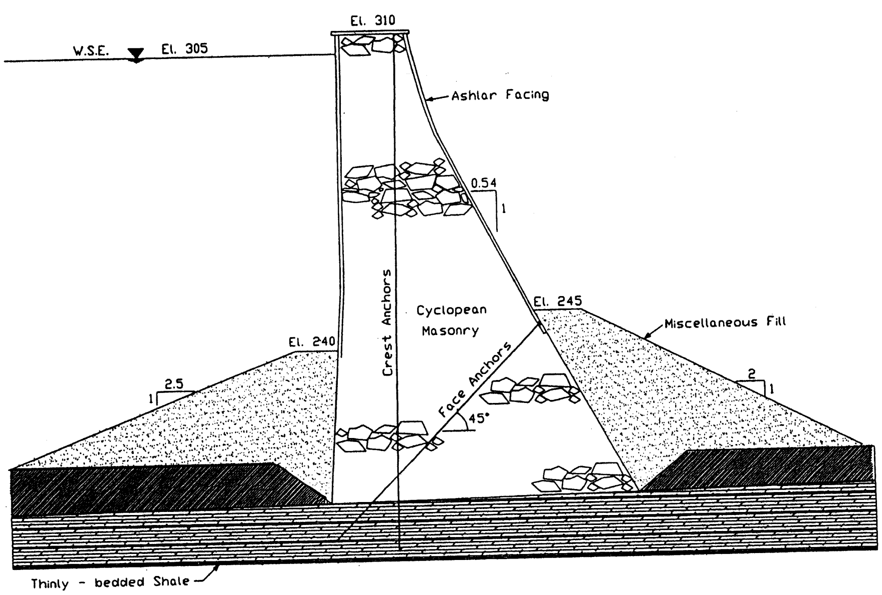

BOONTON DAM
MAXIMUM SECTION
(FIG. 1)

maximize the stone volume. Care was taken to have the stones break joints vertically and horizontally.

The dam was founded upon stratified Triassic sandstone and shale beds dipping 5° upstream. A keyway, 8 feet wide and 2 feet deep, was reportedly built into bedrock a few feet inside the upstream face. An earth berm was constructed against the upstream face to aid in construction and to hopefully reduce uplift. Likewise, construction and miscellaneous fill was used to construct a berm against the downstream face to facilitate erection and increase sliding resistance.

It appears that Boonton Dam was built using some of the most advanced dam engineering of the era. "Numerous diamond drill borings" were undertaken which indicate 10 to 15 feet of shale overlying conglomerate. Stability analyses were undertaken during design where uplift was assumed to vary from two-thirds of full head at the heel reducing to zero at the toe. The resultant of forces under "the most disadvantageous conditions" was maintained within the middle third of the section. Hydraulic model testing was conducted in a small stream to establish the geometry of the spillway chute which successfully directs overflow to the middle of the channel. A sensitivity to thermal cracking was evidenced by the installation of "thermophones" during construction. The accounts state that the instrumentation failed "after a certain time". End of construction inspection revealed "few temperature cracks and very little leakage".

Remediation Design

The USCOE Phase 1 Inspection report included a preliminary stability analysis which indicated that the structure might not meet safety criteria. Subsequently, more detailed analyses supported by geotechnical investigations of the dam and foundation confirmed this conclusion. It was found that the PMF was the critical loading condition for all six representative sections of the dam. For this load case, the resultant of all forces was outside the middle third (kern) and the sliding factor of safety was substantially below criteria. Thus, strengthening requirements were controlled by insufficient sliding resistance. This result was attributed to an inability to confirm the keyway, the absence of bond between the dam and its foundation, and the very thin and poorly-bonded Triassic shales. Post-tensioning by high capacity rock anchors was selected as the remedial method.

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A series of iterative calculations were undertaken during design to optimize the anchoring system with respect to capacity, spacing, tendon angle and constructibility. Particular sensitivity was given to selecting a design which was within the capabilities of a range of specialty contractors thereby ensuring an acceptable number of competitive bids. A combination of face anchors at 45° and near vertical crest anchors at a 5° inclination upstream was found to meet these criteria in the maximum sections (Figure 1). Crest anchors were found sufficient for 450 feet of the non-overtopping section and the spillway.

Although this design requires removal of the concrete overlay, the expense of doing so is justified since experience has shown that most reported anchor failures are the result of tendon corrosion immediately beneath the anchor plate.

Horizontal and vertical strengthening requirements were defined on the contract drawings in kips/linear foot for the crest and face anchors. A complete tabulation of tendon location, inclination and required capacity was included in the specifications to establish a bid basis. Design working loads ranged from 493 to 1127 kips.

Consideration was given to the installation of load cells to monitor long-term load loss and/or to assess the need for re-testing of selected anchors. The justification would be to quantify load changes in the tendon and anchorage assembly, creep within the bond length, and to detect capacity reduction from corrosion. It was decided that instrumentation was not warranted at this site for multiple reasons. For one, the owner did not have sufficient resources to interpret and maintain instrumentation. Secondly, the anchors were locked-off at 70% of the guaranteed ultimate tensile strength (GUTS) of the tendon while the design load was computed using 60% GUTS. Since seating losses are as much as 4% GUTS, a reserve capacity of 6% GUTS, or 10% of design load is available for creep and long-term load losses.

Double corrosion protection was specified for the tendons to combat the slightly saline chemistry of the reservoir water. However, it was thought that some corrosion could still occur over an extended period such that some provision for re-testing would be prudent. Accordingly, the design incorporates a "standard" grease-filled cover over each anchor head assembly which is, in turn, surrounded with asphaltic cement. The top of each anchor pocket is then closed with a six-inch thick concrete overlay. This arrangement minimizes moisture penetration while simplifying access to, and preparation of, the anchor head

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for re-testing. Although this design requires removal of the concrete overlay, the expense of doing so is justified since experience has shown that most reported anchor failures are the result of tendon corrosion immediately beneath the anchor plate.

The project was bid on a unit price basis for all major payment items including consolidation grouting. The quantity for this item was increased substantially over typical norms to account for excess take in the thinly-bedded bedrock and, possibly in the dam masonry itself.

Construction

The project was bid conventionally with Nicholson Construction Company (NCC) being the successful bidder. NCC elected to re-configure the tendon capacities and spacing as allowed by the specifications while still meeting the design strengthening requirements. NCC elected to reduce the number of tendons and increase design working loads to a range of 1373 to 1547 kips.

The non-over topping crest anchors were installed without consequence, except for the anchor pockets. The extreme hardness and joints of the granite masonry coping severely reduced production via offset percussion hammer. Ultimately, NCC was forced to line drilling of the anchor pockets.

The final issue, which caused the most difficulty on this project, was the apparent presence of "micro" fissures in the thinly-bedded foundation shales detected during face anchor drilling.

As noted above, spillway strengthening was limited to inclined anchors on the crest to avoid damaging the very attractive cut stone face and because of the very difficult access for the face anchors. The specification called for the, what is now considered conservative, installation of a spiral reinforcing at the client's request. Noting the reduced production rate for excavating the non-overtopping recesses, elimination of this detail was requested by NCC. Core samples of the ashlar joints were retrieved and subjected to compressive tests at the engineer's request. Although shear stresses

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might challenge the mortar, the laboratory tests demonstrated that the mortar was excellent and that it had sufficient reserve capacity to resist post-tensioning stresses. Consequently, the spiral reinforcing was eliminated from the project.

The final issue, which caused the most difficulty on this project, was the apparent presence of "micro" fissures in the thinly-bedded foundation shales detected during face anchor drilling. The micro fissures are very thin fractures or joints, or minute separations in the shale layers. These features are interconnected sufficiently to be in hydraulic contact with the reservoir. Conventional cement grouting procedures were able to reduce exfiltration rates to Post Tension Institute Standards (PTI 1986). Small flow paths were developing along the grout/bedrock interface around the hole perimeter and projecting to the dam face. Unfortunately, these steps were under high pressure; some were field-demonstrated to have pressures in excess of 30 psi. This pressure corresponded to the full differential hydraulic pressure from reservoir level to the anchor entry elevation.

This situation was considered to be of paramount concern for long-term performance of the re-stressable anchors. Documentation indicates that the majority of anchor fissures throughout the world have occurred because of strand corrosion at the anchor head. Installing the anchors, even though in conformance with PTI standards, might circumvent the multiple defenses incorporated into the design to block any moisture from reaching the tendon steel.

Multiple alternative grouting procedures from that specified were tested including higher pressures, 48-hour maintenance of head pressure, revised conventional cement grout formulations, and hydraulic cement plugs. None of these methodologies was successful in reducing exfiltrations below about 5 gpm. It was concluded that the small size of the fissures would not accept conventional cement particles (Type 1 or 2), and therefore, could not be plugged. Immediately following grout introduction, and during grout hydration, groundwater in the blocked fissures would displace the grout particles adjacent to the drill hole role wall and develop a tiny path to the hole entrance. It is speculated that this phenomenon has been encountered previously in this type of geology. It was unrecognized because the majority of rock anchors are installed from the dam crests above reservoir elevation, as with the Boonton crest anchors.

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A successful test series was undertaken using micro-fine cement injection under pressure exceeding reservoir head, allowing the grout to set under positive pressure. It is believed that the fine grind of this material penetrated the small aperture of the fissures. This procedure, in combination with a hydraulic cement plug in the upper few feet of the drill hole, has allowed the installation of re-stressable anchors with negligible seepage.

The Role of Partnering

In specialty geotechnical works of this type, it is inevitable that changes from the foreseen plan will have to be made. This can occur for a variety of reasons, but the most common cause relates to the subsurface conditions actually encountered once the work commences. The manner in which the risk can be managed by each of the contracted parties is reflected in the type of procurement process. The owner and/or engineer primarily carry the risk for jobs secured under a prescriptive specification, provided the contractor has performed in a manner consistent with the job specifications. With performance specifications, and contracts won with a turnkey approach the contractor may bear a larger share of the risk.

In all cases, however, change brings the potential for conflict, and the threat of legal action. From the description of the work conducted at Boonton Dam, it is clear that several changes were necessitated, involving substantial sums of money. However, the fact that these modifications typically brought little more than healthy debate followed by pragmatic consensus speaks volumes for the attitudes of the parties in general, and the successful application of the principles of partnering in particular.

The concept of partnering is not new—it has always been the firm basis of doing business between honest and responsible parties. In recent years, however, the formal process of partnering has attracted much favorable attention as an aid to avoiding conflict between contracted parties.

At Boonton the atmosphere of partnering was in evidence during the work and during the solution of problems arising from the changes. This atmosphere encouraged the parties to devote their own specific resources to obtain the best practical result. The owner provided financial resources, the engineer technical expertise, and the contractor specific human and mechanical resources. The alternative would have been the scenario all too common on complex evolving jobs of this type: an outraged owner, a disillusioned engineer, an impoverished contractor, and a wealthy attorney.

Common sense, strong personal and professional relationships, and the application of the principles of Partnering together helped ensure that this project was regarded as a success by all its stakeholders (excluding the legal profession). ☐