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GATEHOUSE ANCHORAGE System for Prettyboy DAM

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ABSTRACT

Prettyboy Dam is a large concrete gravity dam that is a key component in the water supply for the 2.7 million residents of the Baltimore, Maryland metropolitan area. Following its construction in the 1930s, significant cracking developed in the gatehouse and adjacent dam.

Following inconclusive investigations of the cracking by several organizations, Alvi Associates performed a forensic investigation of the cracking and identified a complex relationship between causes and effects. A subsequent evaluation of the stability of the gatehouse revealed a serious stability issue, with potential failure consequences including loss of water supply, injury or loss of life due to flooding, economic consequences, environmental damage, loss of public confidence, and damage to a historic resource.

To address this risk, Alvi Associates designed a $6 million gatehouse anchorage system, believed to be the first of its type in the world. It consists of 38 post-tensioned steel anchor bars, drilled up to 70 feet (21 m) horizontally into the gatehouse and dam, and installed under the extremely demanding condition of working in water depths reaching more than 100 feet (30 m).

The project was successfully completed ahead of schedule and under budget in 2010. Due to innovations in all phases of the project—including forensic analysis, rehabilitation design, and construction—the project received the 2010 ASDSO National Rehabilitation Project of the Year Award, as well as two awards in 2011.

Figure 1. Aerial View of Prettyboy Dam

Figure 2. Downstream Face of Prettyboy Dam

Figure 3. Gatehouse at Upstream Face of Prettyboy Dam

PROJECT BACKGROUND

Prettyboy Dam was built during the early 1930s in Baltimore, Maryland and is owned by the city of Baltimore (Figures 1 and 2). It is a concrete gravity dam about 150 feet high (46 m) and 700 feet long (213 m), and classified as a large high-hazard dam. The ogee spillway crest is at elev. 520 (158 m). The dam is founded on rock and supports a multi-span concrete bridge which carries Prettyboy Dam Road.

The dam creates the Prettyboy Reservoir, which has a design storage volume of about 58,000 acre-feet. (7.1×10^7 m^3). Together with the downstream Loch Raven Reservoir, the two reservoirs provide about 60% of the water supply for the 2.7 million residents of the Baltimore metropolitan area. The dam is located along Gunpowder Falls within Gunpowder Falls State Park, one of the most scenic parks in Maryland. This park is a resource for recreational activities such as hiking, fishing, canoeing, kayaking, and bird-watching for a large number of people.

Control of water flow through the dam is via a concrete gatehouse that is monolithic with the dam (Figures 3, 8, and 9) and rests directly on the foundation rock. The gatehouse is located at the upstream (north) face of the dam, near the middle of the dam's length. The gatehouse is rectangular in plan and is 38'-0" (11.6 m) wide parallel to the dam axis and 63'-7" (19.4 m) wide transversely, of which 29'-3" (8.9 m) projects outside the upstream face of the dam. A stairwell is also present in the gatehouse in the portion of the gatehouse within the main body of the dam.

Based on available records, by 1978 extensive cracking was observed in the gatehouse and the adjacent

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main body of the dam, along with substantial water leakage into the gatehouse stairwell, to the extent of requiring staff to wear rain jackets at times. This cracking was observed above water, with the expectation that extensive cracking existed underwater as well, thus posing a risk of gatehouse instability failure, with corresponding potential impacts with respect to water supply, downstream flooding, remedial construction costs, and the natural environment. To respond to this concern, five organizations carried out six investigations of the cracking up until 1994, but with inconclusive and/or inconsistent findings. It was at this point that Alvi Associates became involved with the project.

REVIEW OF PAST PROJECTS, INSPECTION, AND TESTING

Alvi's first general task was a multi-phase dam inspection and investigation, which included review of available records, inspection, crack monitoring, concrete coring and testing, structural/geotechnical analyses and evaluations, and preparation of a comprehensive report with recommendations. Highlights from the records review, inspection, and testing elements of this work are described below.

Review of Available Records. Extensive available records were reviewed. Some key findings were as follows:

1. The dam was constructed between 1931 and 1933.
2. The upstream face of the dam consists of a 4-foot thickness of Class B concrete (2500 psi [17 MPa] nominal compressive strength) which lines the primary Class C concrete (1500 psi [10 MPa] nominal compressive strength) of the dam. However, concrete testing conducted previously, and also during this project, indicated that the concrete quality and strength are relatively high, with typical compressive strengths of 3800 psi (26 MPa) or more, as is often the case for older concrete dams.
3. The dam concrete is generally unreinforced.
4. There may be vertical construction joints between the gatehouse and the main dam. Such joints would create vertical planes of weakness at these interfaces, which are unreinforced.
5. The dam has thirteen vertical contraction joints in planes oriented perpendicular to the axis of the dam, forming thirteen monoliths. They are at various locations along the dam axis, including each side of the gatehouse. The contraction joints have vertical keys and are not grouted.
6. The foundation rock is a foliated micaceous schist which has nearly vertical joint and fault sets and strikes about normal to the dam axis.
7. Rock excavation for the foundation was carried much lower and wider than anticipated because rock at planned foundation elevations was much more weathered than expected. The revised excavation limits increased the rock excavation volume by nearly six times and approximately doubled the volume of required dam concrete.
8. For seepage and uplift control, the upstream third of the foundation rock was grouted in a grid pattern and there is an unreinforced concrete cutoff wall along the upstream face of the dam which is about 6 feet (2 m) thick and 10 to 15 feet (3 to 5 m) deep vertically.
9. From 1978 to 1994, several inspection projects related to the dam were conducted. In broad terms, all organizations reported generally similar findings regarding the extent of cracking. Therefore, at least the majority of the existing cracking seems to have been well established before 1978.
10. In addition to the various inspection projects, crack monitoring was conducted for cracks in the east and west gatehouse walls during the 16-month period from June 1990 to October 1991. However, after careful review, the crack monitoring program was determined to be flawed by numerous problems, and therefore did not provide useful data regarding whether the cracks were stable.

Phase I Inspection. Phase I inspection involved an initial underwater swim-by inspection to survey the general condition of the upstream face of the dam and gatehouse. A video record was kept for each dive, complete with running commentary by engineers and divers and supplemented with field notes and sketches. Major defects were approximately located and noted for reference during Phase II and Phase III inspections. Detailed measurements of defects were not made during this phase.

Phase II Inspection. Phase II inspection involved detailed mapping of all defects at the upstream face of the dam and gatehouse. This was performed by using a remotely operated vehicle (ROV) equipped with a video camera electronically linked to a 3D positioning system. Individual defects were saved by the software as a series of data points, from which drawings were automatically generated. These drawings were supplemented by supporting field notes and hand sketches prepared by the engineer at the monitoring screen.

Phase III Inspection. After Phase II inspection, several areas were identified as requiring underwater hands-on Phase III inspection. These areas were predominantly on the east and west gatehouse walls, around the bottom of the bridge piers, and in a few areas of the dam face. Most of the inspection time was spent on the east and west gatehouse walls to obtain detailed information on the observed vertical and diagonal cracks. For this phase, since detailed information on crack width and location was needed, underwater power washing was used to clean efflorescence and marine growth from the concrete surfaces.

Observed Gatehouse Cracking. Cracking, spalling, and other defects were mapped throughout the exterior and interior of the dam and gatehouse. Of these defects, the cracking in the gatehouse (Figures 4-6) was the focus for this project.

In the east face of the gatehouse (Figure 4), nearly all of the cracks emanate from the interface of the gatehouse and main body of the dam, the crack orientation is primarily diagonal to nearly vertical, and the crack widths and density of cracks are greater at higher elevations. At least some of the cracks were confirmed to extend through the thickness of the gatehouse walls.

In the west face of the gatehouse (Figure 5), as with the east face, nearly all of the cracks emanate from the interface of the gatehouse and main body of the dam, the crack orientation is primarily diagonal to nearly vertical, the crack width and density of cracks are greater at higher elevations, and at least some of the cracks were confirmed to extend through the thickness of the gatehouse walls. However, comparing with the east face, the overall extent of cracking is greater in the west face and the orientation of the cracks is more vertical.

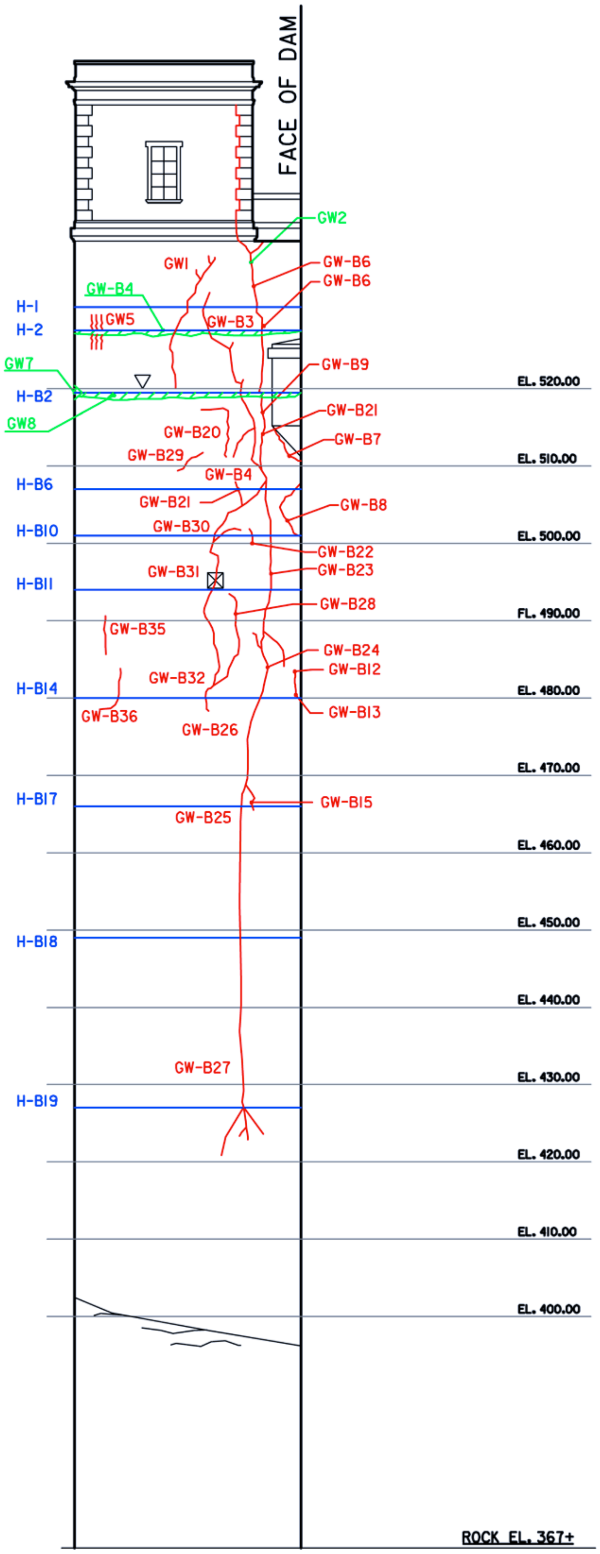

Figure 4. Cracking in East Face of Gatehouse

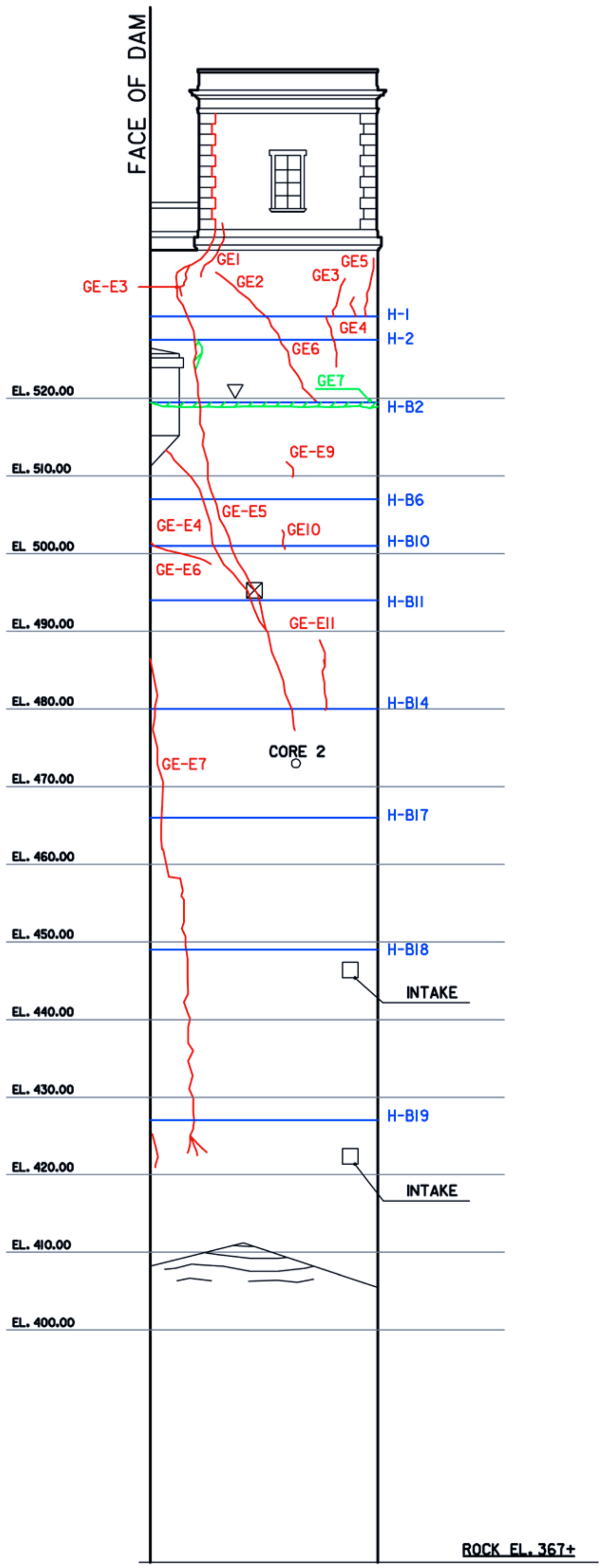

Figure 5. Cracking in West Face of Gatehouse

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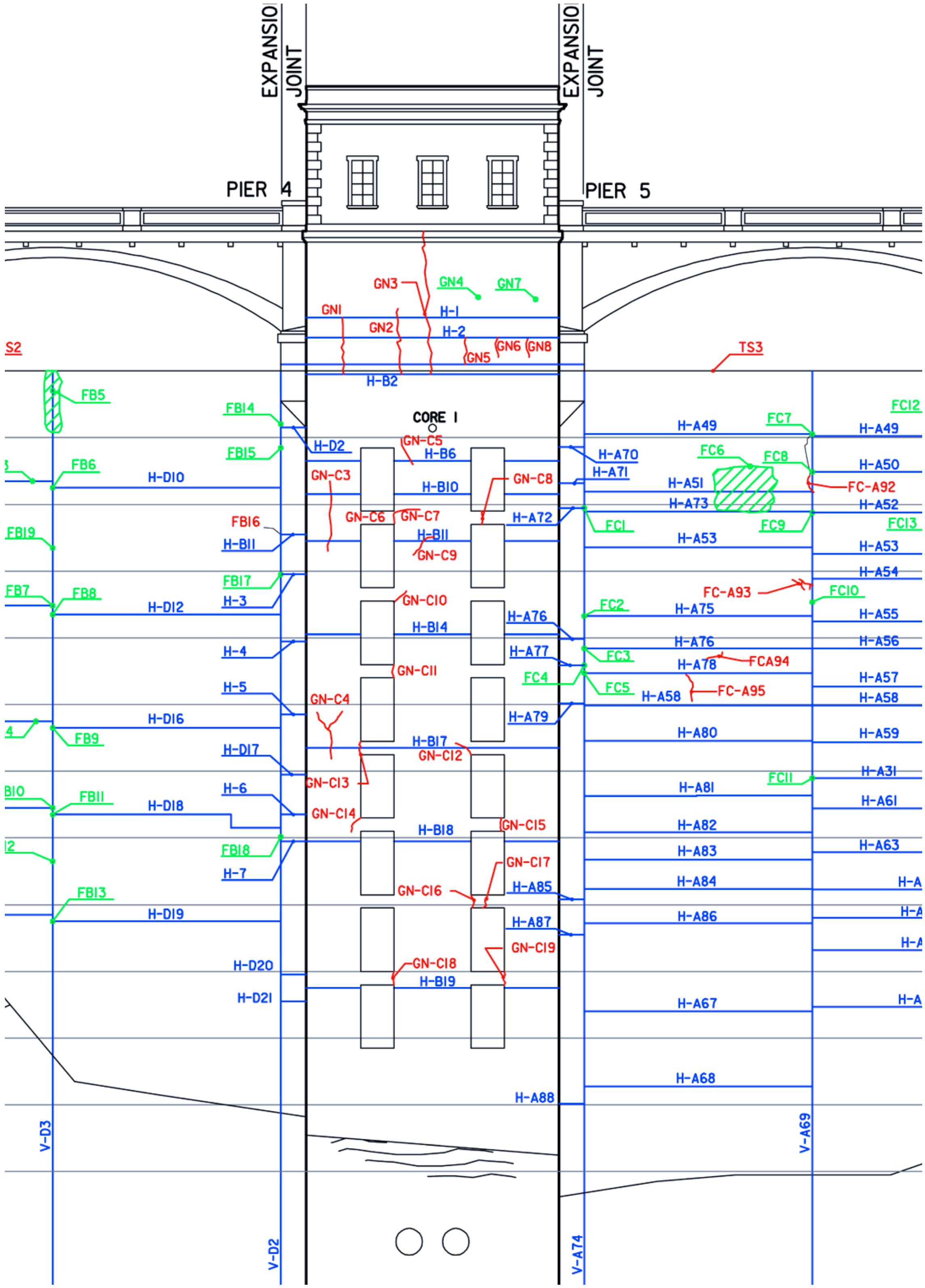

Figure 6. Cracking in Upstream (North) Face of Gatehouse

The cracking in the north face of the gatehouse (Figure 6) is not as extensive as the east and west faces, and the crack widths are generally narrower. Most of the cracks are vertical, although a few are horizontal and diagonal.

FORENSIC ANALYSIS OF CRACKING

Determination of the cause(s) of the cracking was a challenging process involving numerous iterations of formulation of causal hypotheses and "testing" of those hypotheses. This testing was done by comparing (a) the predicted cracking based on mechanistic models of dam/foundation deformations and stresses with (b) the observed cracking. Iterations continued until a good fit was obtained between predictions and observations. A key step in this process was conducting several lengthy sessions that isolated team members from distractions, allowing them to explore the problem at a high level of concentration.

The team's overall conclusion was that the cracks clustered into eight distinct groups, with three distinct causal mechanisms, each mechanism contributing in varying degrees to each crack group. Team members developed a "cause-effect matrix" (Figure 7), transcending the usual assumption of a simple one-to-one influence of cause to effect. The check marks in this matrix indicate strong (✓✓✓), moderate (✓✓), or slight (✓) quantitative contribution of each cause/mechanism to each effect/crack group. Key characteristics of each crack group are as follows:

- **Crack Group 1** consisted of cracks in the exterior of the east and west gatehouse walls, as described above (Figures 4 and 5). The cracks in this group were relatively interconnected, but not to the extent that portions of the gatehouse fractured and became dislodged. However, the gatehouse did appear to have displaced upward relative to the main dam by about 1/8" (3 mm). This displacement appeared to be somewhat larger at the west side of the gatehouse as compared with the east side. The displacement was evidenced by a crack in the gatehouse floor slab and a similar crack in the bridge sidewalk in front of the entrance to the gatehouse. At both of these cracks, the north (gatehouse) side was displaced upward relative to the south side, as though the cracks were out-of-plane shear cracks. This displacement suggested that the vertical cracking in the gatehouse walls adjacent to the dam face may have been interconnected enough to have resulted in a shear fracture along this plane. Vertical construction joints in this location, if present, would have facilitated this.
- **Crack Group 2** consisted of cracks in the exterior of the north gatehouse wall, also as described above (Figure 6). The most prominent of these cracks were vertical and located within the 20 feet (6 m) of gatehouse wall above the spillway crest. The longest and widest of these cracks was found at the mid-width of the wall and penetrated the full thickness of the wall. Less prominent cracks were also present below the crest, primarily toward the east side of the wall. Most of these cracks were vertical and the others were horizontal and diagonal.
- **Crack Group 3** consisted of cracks in the gatehouse floor slab located at about elev. 527.5 (160.8 m) (Figure 8). All cracks in this group were interconnected with each other, and also with several prominent cracks in Crack Groups 1 and 5. A prominent crack extended all the way across the floor between the east and west gatehouse walls, parallel to and about 2 feet (0.6 m) downstream of the upstream face of the dam. Additional cracks ran parallel to the east and west walls, about 2 feet (0.6 m) from the walls. As with Crack Group 1, shear displacement had occurred at these cracks, suggesting that the gatehouse had displaced upward relative to the main body of the dam.
- **Crack Group 4** consisted of cracks in the east and west gatehouse walls in the area below the bridge over the dam, within the interior of the dam (Figure 8). Of the most prominent cracks, most were full-height or nearly full-height vertical cracks, and the rest were horizontal. The less prominent cracks were generally diagonal. Many of the cracks emanated from the bridge girders where they framed into the gatehouse walls. These walls are reinforced concrete and about 18 inches (450 mm) thick, which is somewhat thin relative to the bridge reactions they must carry.
- **Crack Group 5** consisted of cracks in the east and west gatehouse walls north of the bridge area, also at the interior of the dam (Figure 8). These cracks were mostly full-height cracks or nearly full-height vertical cracks.
- **Crack Group 6** consisted of cracks in the interior of an open chamber room near the top of the gatehouse, at the interior of the dam (Figure 8). These cracks included a full-height vertical crack near the mid-width of the south wall, a horizontal crack in the north wall, and full-height vertical cracks with branching cracks in the east and west walls.
- **Crack Group 7** consisted of cracks in the stairwell walls (Figures 8 and 9). The vast majority of these cracks were vertical cracks in the east and west stairwell walls within the 100-foot (30-m) vertical range between the crest at elev. 520 (158 m) and the valve room at elev. 420 (128 m). These vertical cracks were concentrated toward the top of this range. In addition, there were a few horizontal cracks in the north and south walls, the most prominent of which were near the dam crest.
- **Crack Group 8** consisted of cracks running along the spillway crest about 4 feet (1.2 m) from the upstream face of the dam. These cracks extended through both west spillways of the dam, and the majority of the east spillways. The vertical penetration of these cracks into the dam is unknown.

The three identified causal mechanisms of the cracking were vertical flexure of the dam, differential settlement, and bridge deformation:

- ***Vertical Flexure of the Dam*** – In this mechanism, the dam acts as a beam-on-grade in a vertical plane parallel to the dam axis. The dead load of the dam is the primary load involved, which results in settlement by compressing the foundation rock. Primary factors affecting the settlement are the varying height of the dam, stiffness of the foundation rock, and creep behavior of the concrete and foundation rock. Because the dam is tallest and heaviest in the gatehouse region, more settlement would occur here than in the farther spillway and abutment regions. Resulting variable settlement would produce vertical flexure of the dam, with longitudinal compressive stress near the top of the dam, tensile stress near the bottom of the dam, and a neutral axis near elev. 420 (128 m). Increasing immediate settlements would develop during

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Crack Groups (Effects)	Causes		
	A Vertical Flexure	B Differential Settlement	C Bridge Deformation
Crack Group 1 Gatehouse East and West Wall Exteriors	✓✓	✓✓✓	
Crack Group 2 Gatehouse North Wall Exterior		✓✓✓	
Crack Group 3 Gatehouse Floor Slab	✓✓	✓✓✓	
Crack Group 4 Gatehouse East and West Wall Interiors in Bridge Area	✓		✓✓✓
Crack Group 5 Gatehouse East and West Wall Interiors Outside Bridge Area	✓✓	✓✓✓	
Crack Group 6 Fourth Level Chamber Room Interior	✓✓✓		
Crack Group 7 Stairwell Walls	✓✓✓	✓	
Crack Group 8 Spillway Crest	✓✓	✓✓	

Figure 7. Cause/Effect Matrix

construction as lifts were placed, and a magnification of these settlements would occur within an estimated five to ten years after construction because of creep in the concrete and foundation rock.

Compressive stress at the top of the dam based on this mechanism is estimated to be about 400 psi (2.8 MPa). Although difficult to model without detailed data and testing, creep of the rock could increase this value to 1000 psi (7 MPa) or more. Any tensile stress at the bottom of the dam would be much smaller, and would tend to be relieved by the vertical contraction joints in the dam. However, due to the presence of vertical steps in the foundation rock, particularly where the spillways meet the abutments, the vertical rock faces could resist expansion of the tensile zone at the bottom of the dam, thereby exerting a lateral thrust with an associated arching mechanism. This would shift the neutral axis down, with an estimated increase of compressive stress at the top of the dam of about 100 psi (0.7 MPa), and a decrease of tension at the bottom of the dam.

The compressive stress near the top of the dam would extend through the main dam, but not the gatehouse beyond the upstream dam face. This would produce "pinching" of the gatehouse near the upstream dam face, with resulting shear and bending in the east and west gatehouse walls, particularly near the wet wells (Figures 8 and 9). The shear and bending could produce vertical shear cracks and vertical flexural cracks consistent with Crack Groups 1 and 5. A similar mechanism, in which the top of the main dam is compressed while the bridge over the dam is not, could contribute to cracking consistent with Crack Group 4.

The relatively high compressive stress and strain near the top of the dam, along the axis, would produce a corresponding lateral tensile strain due to the Poisson effect. This tensile strain could contribute to Crack Groups 3, 6 (in the north wall), and 8.

In addition, the compressive stress would produce horizontal tensile stress in the east and west stairwell wall faces because the stairwell shaft is a void within a solid, and such a void causes a significant change in the local stress field. Because the shaft is a relatively square void (Figure 9), the tensile stresses would be about equal to the free-field compressive stresses (the compressive stresses which would exist if there was no void). These stresses are estimated at 400 psi (2.8 MPa) or more, as described above. The existing concrete has an estimated tensile strength of about 400 psi (2.8 MPa) and therefore the Group 7 vertical cracking in the east and west stairwell walls is consistent with this mechanism. A similar mechanism could contribute to the Group 6 cracking in the east and west walls of the open chamber room.

The presence of the stairwell shaft also produces a magnification of the free-field compressive stress in the north and south stairwell walls by a factor of about three times. The resulting compressive stresses in these walls could reach well over 1200 psi (8.3 MPa) and could contribute to horizontal cracks, due to the Poisson effect, which are consistent with the Group 7 cracking in these walls.

Development of cracking resulting from this mechanism would generally follow the development of settlement. Therefore, most cracks would have occurred early in the dam's life, probably within the first ten years.

- ***Differential Settlement*** – This mechanism involved an upward reaction under the gatehouse due to restraint of differential settlement between the gatehouse and the main dam. More specifically, if the gatehouse and main dam were not physically connected, the main dam would experience a larger settlement than the gatehouse because of its much larger footprint but similar bearing pressure. Therefore, the upstream face of the main dam would "slide down" relative to the gatehouse by the amount of the differential settlement between the main dam and gatehouse. Based on settlement calculations, this differential displacement was estimated to be about 1/10" (2.5 mm).

  However, because the gatehouse was designed to be physically connected to the main dam, the differential settlement would be restrained and the main dam would be unable to slide past the gatehouse. Instead, the main dam would "pull" the gatehouse down into the foundation rock by the amount of the differential settlement, with a resulting upward reaction at the bottom of the gatehouse due to compression of the foundation rock. This reaction would tend to be larger farther from the dam face.

  The key loads affecting this mechanism included the dead load of the main dam, the dead load of the gatehouse, the lateral water pressure against the upstream face of the dam, the uplift pressure under the main dam, and the uplift pressure under the gatehouse. The uplift under the gatehouse would tend to remain near full headwater because of the cutoff wall under the main dam. However, according to a prior investigation, the uplift head under the main dam behind the cutoff wall was reduced to about 40 feet (12 m) less than full headwater. In addition, for a given decrease in headwater, the uplift under the main dam decreased considerably more than the headwater decrease. Therefore, the larger uplift under the gatehouse compared with the main dam tended to increase the amount of any restrained differential settlement.

  Besides these loads, key factors which would influence the effects produced by this cause include construction sequencing, variations in stiffness of the foundation rock, and creep behavior of the foundation rock. Construction sequence in particular would play a significant role because the dam was constructed in lifts during a period of about two years. During placement of initial lifts, the projecting gatehouse would behave similarly to a cantilever beam and would be susceptible to vertical flexural cracking near the upstream dam face. However, dead loads and settlements would be relatively low during these initial stages.

  During intermediate stages, the projecting gatehouse would behave like a deep beam and would be susceptible to shear and diagonal tension cracking across most of its width. Tensile cracking along internal compression thrust lines due to the Poisson effect would also be possible. During the last stages, the projecting gatehouse would behave like a tall narrow shear wall, and cracking would be similar to that of a deep beam. After construction, restrained differential settlement could increase because of creep of the foundation rock, and cracking would likewise increase.

  Crack Groups 1 and 5 are clearly consistent with this mechanism. The high elevation of most of these cracks could be due to effects of construction sequence and the lack of voids in the gatehouse below elev. 420 (128 m). The presence of cracks near the wet wells is also consistent with this mechanism because the gatehouse walls are relatively thin in this area.

  This mechanism could also contribute to the Group 2 cracks in the north gatehouse wall if the restrained differential settlement varied across the width (east-west direction) of the gatehouse. Such variation would cause vertical shear and diagonal tension in the north wall. The lack of existing cracks at lower elevations could be due to the columns of openings in the wall (Figure 6), which would tend to relieve shear forces by accommodating deformation. The vertical rather than diagonal trends of the cracks in the upper portion of the wall could be a result of light vertical reinforcing steel in the wall, which would tend to limit diagonal cracking.

  The Group 3, 7, and 8 cracking is also consistent with this mechanism, particularly for Crack Group 3 because out-of-plane shear displacement was observed at these cracks. This displacement was discussed in the descriptions of Crack Groups 1 and 3. Because the observed 1/8" (3 mm) displacement is comparable to the predicted 1/10" (2.5 mm) or greater differential settlement, it appears likely that the force resulting from restraint of the differential settlement was sufficient to cause a full-height shear fracture at the gatehouse/ dam interface, thereby relieving restraint of the differential settlement and allowing it to occur. If this is the case, no further cracking due to this mechanism would be expected.

  Regardless of whether such a fracture already occurred, any differential settlement would be expected to have developed within ten years after the dam was constructed, and therefore this mechanism would not be expected to generate additional crack growth.

- ***Bridge Deformation*** – This mechanism relates only to Crack Group 4 and involves cracking resulting from reactions from the portions of the bridge that frame into the walls, specifically the slab and T-beams. This mechanism simply involves the negative moments at the ends of the T-beams inducing flexural and shear stresses in the walls. These stresses would produce cracking consistent with Group 4, including punching-shear type cracking near the ends of the beams. Due to the extent of the existing Group 4 cracking, further cracking is not anticipated unless unusually heavy live loads cross the bridge.

In summary, the general root cause of the cracking was differential settlement and creep, which manifested itself through the three primary causal mechanisms described above. Each of these mechanisms in turn produced particular stress patterns which were superimposed to produce the observed cracking for the eight described crack groups. With respect to timeline, as noted above, it appears that the cracking initiated within about ten years after construction of the dam, some of it likely occurring during construction, and then the cracking propagated at a limited rate after that first decade after construction. However, the possibility of at least some further cracking occurring could not be entirely ruled out. Moreover, it was possible that the cracking had already reached a stage of being extensive and interconnected enough to result in gatehouse instability. Alvi therefore performed a gatehouse stability analysis, as described below.

STABILITY ANALYSIS

Stability analysis for the gatehouse was performed in a manner similar to a rock or earth slope stability analysis, assuming planar failure surfaces relying on shear-friction (no cohesion). Due to the considerable uncertainties involved, a spreadsheet was developed and used to check a large number of potential failure scenarios varying with regard to assumed location of diagonal failure surface, water level, and friction coefficient (μ typically close to 1.0 to model irregular cracks). The general result was that factors of safety varied from less than 1.0 to more than 1.0, with a substantial number of scenarios having a factor of safety less than 1.0, thus indicating a significant risk of failure.

RISK ANALYSIS

From a risk analysis standpoint, a precise probability of failure could not be plausibly estimated, and so the design team instead "worked backwards" to determine the best course of action. This involved first determining the consequences of a catastrophic failure in which a substantial portion of the gatehouse literally breaks off and falls into the reservoir. Such a failure would prevent control of water flow through the gatehouse, with a spectrum of potential consequences:

- At one end of the spectrum, the flow through the gatehouse could become inadequate, particularly if the gatehouse valves

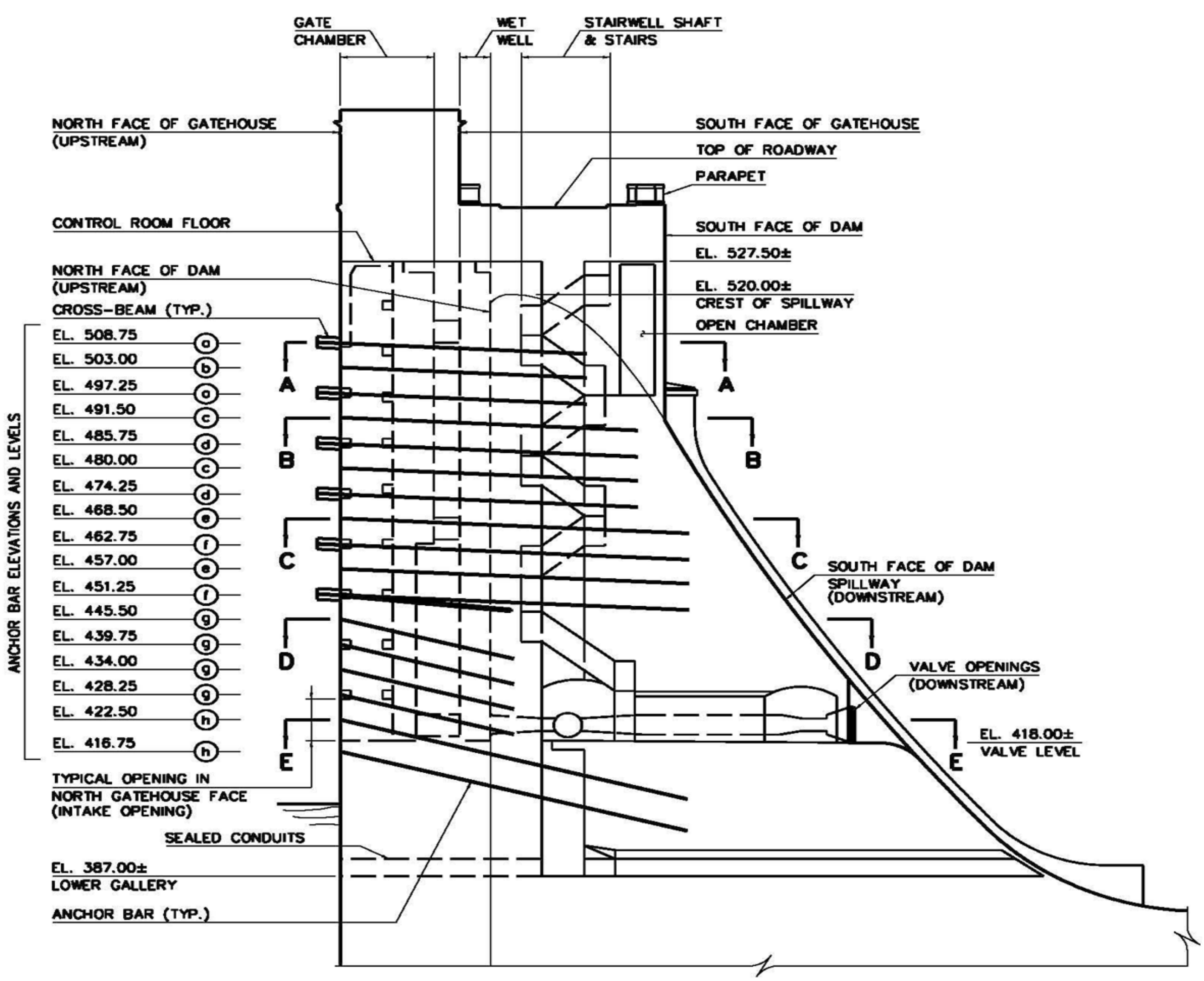

Figure 8. Gatehouse Typical Section (Elevation View)

could not be opened. This could jeopardize the water supply to the Baltimore metropolitan area as well as the downstream Gunpowder Falls State Park, with an associated cascade of economic impacts.

- At the other end of the spectrum, the downstream flow could become excessive, thus causing potential flooding impacts, as well as loss of water storage if the downstream Loch Raven Reservoir was already filled to capacity.
- In addition to these impacts to the downstream park and Baltimore's water supply, the cost of reconstructing the gatehouse and adjacent dam could easily reach several tens of millions of dollars.
- Last but not least, the loss in public confidence and political consequences associated with a gatehouse failure would be very substantial.

While many of these impacts cannot readily be quantified in terms of dollars, $100 million is perhaps a reasonable ballpark estimate of the combined financial consequences of failure. In comparison, the estimated cost to reliably rehabilitate the gatehouse was about $6 million. Therefore, if the probability of failure was at least 6%, the associated risk-cost would be least about $6 million; thus the appropriate action from a cost-benefit standpoint would be to rehabilitate the gatehouse.

Based on engineering judgment, considering the plausibility of the parameters used in the stability analysis and the plausibility of each associated failure scenario, the design team concluded that the (subjective) probability of failure could plausibly be more than 6%, possibly much more; accordingly, the team recommended proceeding with rehabilitation of the gatehouse and the city of Baltimore concurred.

REHABILITATION DESIGN

To stabilize the gatehouse, in consultation with contractors within and beyond the United States, the design team developed and considered a large number of rehabilitation alternatives, including grouting of cracks, various types and configurations of drilled

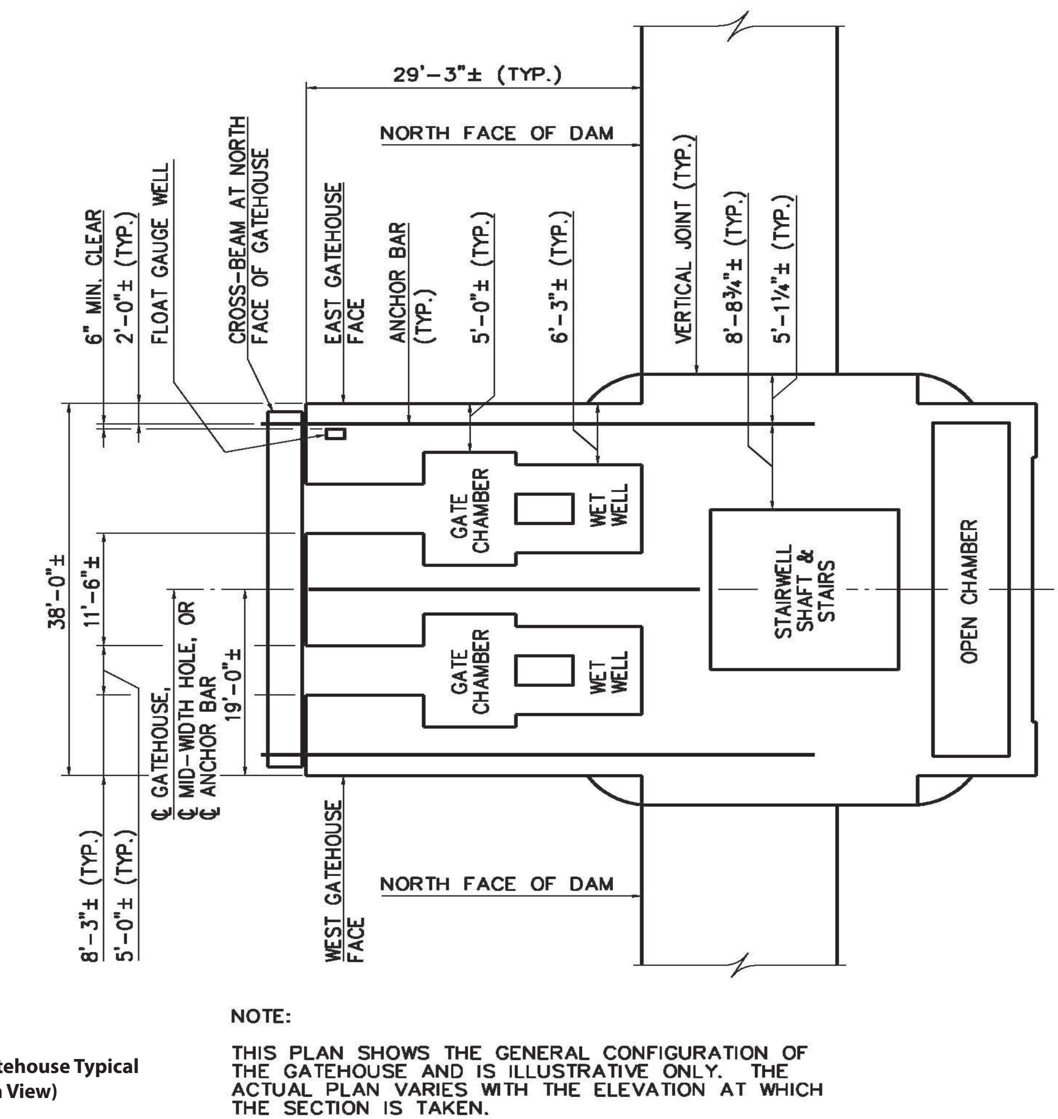

Figure 9. Gatehouse Typical Section (Plan View)

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anchors, installing external frames to secure the gatehouse, and combinations of these approaches. Since lowering of the reservoir was not an option, the team also considered a variety of construction methods, including working underwater, using a cofferdam, and working from the downstream dam face.

After development and evaluation of the alternatives, the selected (and clearly preferred) alternative was to grout the gatehouse cracks and then install an anchorage system consisting of 38 post-tensioned steel threadbar anchors oriented nearly horizontally (6° and 15° slopes) (Figures 8 and 9). Each anchor was 1-3/8" (35 mm) diameter, Grade 150 ksi (1034 MPa) steel, provided with a Class I double corrosion protection system, and had a 15-foot (4.6 m) bond length. The anchors were core-drilled with lengths from 48 to 70 feet (15 to 21 m) into the dam, all while working underwater in water depths up to 100+ feet (30+ m).

The precise location, orientation, and length of each anchor was selected to carefully avoid many fairly tight constraints within the gatehouse, including intake openings, gate chambers, wet wells, a float gage well, a stairwell, a valve chamber, various other chambers and galleries, and of course the downstream face of the dam (Figures 8 and 9).

To provide adequate stabilization force while working around these many constraints, two different anchorage systems were used in combination:

- One system consists of 26 anchors, with the anchors directly attached to the north face of the gatehouse (Figures 8 and 9). To prevent shearing of the gatehouse concrete during post-tensioning, these anchors were tensioned to a modest load of 50 kips (223 kN) and grouted in two stages, with the second grouting stage being bonded to increase the ultimate capacity of the anchors. A special rubber "wiper" detail was developed to enable this two-stage grouting in the relatively deep underwater conditions such that the bond zone would be fully grouted, while avoiding leakage of the grout into the free-stressing zone. The key in developing the wiper detail was to select a rubber material and geometry that would enable insertion into the drilled holes without binding, while also providing a tight enough seal to withstand grouting pressures.
- The other system consists of 12 anchors arranged in pairs, with each pair of anchors transferring load through a large high-strength prestressed concrete beam (weighing 32 tons [29 metric tons]), which bears against the mid-width portion of the gatehouse through a pair of elastomeric pads (Figures 8 and 9). These anchors were each grouted in a single stage, and relatively precise simultaneous jacking was required for each pair of anchors. The top four anchors were tensioned to 50 kips (223 kN) and the bottom eight anchors were tensioned to 150 kips (668 kN).

Because the dam is eligible for the National Register of Historic Places, the anchorage system was designed to minimize its aesthetic impact and to maintain historic integrity. This included use of concrete cross-beams with a color similar to the dam concrete, and painting steel hardware with a similar color.

VERIFICATION OF DESIGN RELIABILITY

To verify design reliability and proactively prevent construction problems and damage to the dam during construction, the contract documents included several key measures:

- Only prequalified contractors were allowed to bid on the project. Because of the uniquely challenging nature of the project, Alvi Associates contacted more than 100 contractors around the United States, of which only four became prequalified.
- To ensure adequate pre-mobilization research and planning for contingencies, rather than allowing piecemeal shop drawing submissions, Alvi required submission of a single coordinated shop drawing package covering all aspects of the work. If any element required revision, the entire package had to be resubmitted. In addition, the contractor was not allowed to mobilize until the shop drawing package was accepted. At the end of the project, there was a consensus that this rigorous shop drawing process had proven to be highly beneficial, if not vital.
- While the coordinated shop drawing review process was a necessary step, empirical evidence that the process would be successful was also necessary. This need was met by a program of four preproduction test anchors to be installed in a location of the dam away from the gatehouse, so that any problems encountered during preproduction testing would provide a useful learning experience without risking permanent damage to the gatehouse or other sensitive portions of the dam. Only a few relatively minor issues were encountered during preproduction testing, and this experience was used to fine-tune the shop drawings.
- The cracking pattern in the gatehouse was such that, with increasing water depth, the cracks were less dense and less wide (Figures 4 and 5). In view of this pattern, the design team decided to "zip up" the gatehouse by specifying a particular anchor stressing sequence, which generally involved working from the bottom up.
- Given the challenging conditions of working underwater, underwater inspections were performed at key milestones during the construction process. These inspections enabled close monitoring to ensure that the construction process stayed on track, thus allowing any issues to be promptly identified and effectively corrected while still at a readily manageable stage.

Figure 10. Underwater Installation of Cross-Beam Bracket

Figure 11. Custom-Modified Core Drill

CONSTRUCTION

The anchorage system was constructed by Brayman Construction. Alvi and Brayman worked in close partnership to review technical submittals and resolve issues during construction. The following are some highlights from the construction phase:

- As noted above, nearly all of the work needed to be performed underwater, in water depths reaching more than 100 feet (30 m). This required rotating teams of divers using two decompression chambers and helmet-mounted underwater cameras, and maintaining continuous audio communication between the divers and supervisors at the surface (Figure 10).
- For anchor drilling, a core-drill was custom-modified to "breathe" underwater and was equipped with sensors to monitor hydrostatic water pressure (Figure 11). The drill was positioned on a specially designed drill cart equipped with four cameras, and was operated remotely from the surface. The drill and drill cart were precisely positioned to meet the tight construction tolerances by attaching them to an extensive drilling template consisting of vertical and horizontal steel H-beams.
- Two anchor holes in deeper water severely failed to pass watertightness tests and had high grout takes during pregrouting, thus indicating substantial leakage through the dam. The team worked together closely and quickly to investigate a wide variety of potential solutions and, as "Plan A," agreed to add polypropylene fibers to the grout mix, with corresponding adjustments to the grouting equipment and procedures. This solution was successful.
- To ensure adequate bond strength after core-drilling, the anchor holes were carefully roughened using roller bits, with a special underwater camera used to inspect the holes and evaluate the level of roughness.
- As noted above, specially designed rubber "wipers" were used to allow underwater two-stage grouting, and simultaneous jacking was required for the pairs of anchors at the cross-beams (Figure 9).
- Since the dam is located along the Gunpowder Falls State Park, mitigation of environmental impacts was a high priority. To meet this need, a stream monitoring program was developed to protect the aquatic habitat, home to a carefully controlled population of blue-ribbon trout, among other species. The program monitors pH, temperature, and turbidity at stations both upstream and downstream of the dam. Measures implemented to prevent water contamination included use of biodegradable and environmentally-safe hydraulic fluids in all underwater equipment, as well as innovative methods to contain drill shavings and grout overflow, pipe these waste materials to the surface, and isolate them for safe disposal.

Because of the cooperation of all parties, the construction process went exceptionally well and the project was completed ahead of schedule and under budget in 2010.

LESSONS LEARNED

The following are some of the key lessons learned from this project which may be applicable to other projects in the future:

1. The effects of differential settlements, including creep effects after construction, should be carefully considered in the design of large concrete dams.
2. The effects of varying dam height, and varying footprints among portions of a dam, should be considered when estimating foundation settlements.
3. When potentially serious cracking is observed in a concrete dam, detailed crack mapping should be performed as an initial step in investigation of the cracking. Subtleties revealed by such mapping can provide important forensic clues.
4. When investigating cracking or other distress in dams, consider the possibility that there may be a complex relationship between causes and effects, including presence of multiple causes and multiple effects, with different causes simultaneously contributing in different degrees to different effects.
5. In forensic investigation, particularly in complex situations, it is often not possible to definitively prove the validity of a causal hypothesis. Instead, "goodness of fit" between observations and predictions based on a hypothesis is the best available criterion for accepting or rejecting a hypothesis.
6. Forensic investigations of complex situations may require "intellectual critical mass" associated with a sustained high level of concentration, as achieved by isolating investigators from ordinary daily distractions for long blocks of time.

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7. For both forensic investigation and design, develop analytical models at a level of sophistication commensurate with the available data and the needs of the project – neither oversimplified nor unnecessarily complex. Pay as much attention to fundamental qualitative modeling assumptions as quantitative specifics.
8. When substantial parametric uncertainties are involved, perform sensitivity studies rather than relying only on "best guess" estimates of parameters.
9. Carefully consider the effects of construction sequencing in both analysis and design.
10. When performing risk analysis, if probabilities can not be precisely estimated on any objective basis, consider "working backwards" to determine what probabilities would be needed to result in choosing one course of action versus another, then subjectively judge the plausibility of those probabilities.
11. For relatively unique projects, cast a wide net in seeking relevant past experiences and advice from colleagues, including contractors.
12. Installation of horizontal anchors in concrete dams can be successfully performed underwater at considerable depths, but beware that standard procedures, materials, and equipment may need to be substantially modified, and costs are likely to be much higher than when working above water.
13. Core-drilling is a suitable method for installing long horizontal anchors in concrete dams while meeting relatively tight tolerances. However, beware that core-drilling will produce smooth holes which may need to be roughened to reliably ensure adequate bonding.
14. For challenging projects, consider (a) prequalifying contractors, (b) requiring a single coordinated shop drawing submittal detailing all elements of the work, and not allowing contractor mobilization until this submittal is accepted, (c) performing preproduction testing, (d) performing thorough milestone construction inspections to ensure that the work stays on track, (e) having contingency plans in place to promptly address any problems that may arise (and be imaginative in identifying such potential problems), and (f) partner closely with the contractor in good faith to ensure an effective working relationship with the contractor from start to finish.
15. Be sure that all necessary environmental protection measures are implemented during construction. ≋

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Mr. Alvi has more than 22 years of experience in structural, geotechnical, water resources, and transportation engineering for dams and other infrastructure facilities. He has completed a diverse range of dam projects involving inspection, materials testing, hydrologic and hydraulic analysis, reservoir routing and spillway capacity analysis, dam break modeling and inundation mapping, stream geomorphic study and restoration design, fish passage design, seepage and stability analysis, three-dimensional structural analysis, forensic investigation, risk analysis, remedial design, design of new concrete and embankment dams, evaluation and design for dam removal, and construction management. His projects have received 7 design awards during the past 5 years, including the 2010 National Rehabilitation Project of the Year Award from ASDSO for the Prettyboy Dam rehabilitation project described in this paper. He is also a member of the ASDSO Dam Failure Investigation Committee.