USING POST-TENSIONING TO MITIGATE ALKALI-SILICA REACTION DAMAGE

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Abstract

As a result of the alkalinity of Portland cement, a chemical reaction often occurs at the interface of the cement paste and certain types of aggregate. This chemical reaction results in the formation of a chemical compound at this interface area that is referred to as "ASR gel". ASR gel expands in the presence of water causing the aggregate to separate from the surrounding cement paste resulting in cracking and deterioration of the concrete. This paper presents the history and studies conducted to identify and mitigate ASR effects in a massive concrete abutment supporting a major arch bridge.

Introduction

Opened in 1931, the Bayonne Bridge connects the cities of Bayonne, New Jersey, and Port Richmond on Staten Island, New York. The bridge is owned by the Port Authority of New York and New Jersey (PANYNJ). The main span of the bridge, supported on massive concrete abutments founded on diabase bedrock, consists of a two-hinged parabolic steel arch 1675 ft. (510.5 m) long (see Figure 1). The New Jersey arch abutment is a concrete block, measuring approximately 133 ft. (40.5 m) x 107 ft. (32.6 m) at the base and is 40 ft. (12.2 m) high. Over the years, this abutment exhibited signs of deterioration in the form of random cracking of the exposed concrete surfaces. Attempts were made in the late 1970's and the mid 1980's to seal and repair the surfaces but were not successful in stopping further cracking.

Figure 1. Bayonne main arch span looking south toward Staten Island

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Through visual examinations and petrographic testing of concrete core samples Alkali-Silica Reaction (ASR) was identified as the cause of cracking.

Several possible rehabilitation schemes ranging from lithium injection to reconstruction were studied. A rehabilitation scheme consisting of encasing the original abutment in new concrete and tri-axially post-tensioning the abutment was selected.

<u>Description of the Problem</u>

ORIGINAL ABUTMENT - The original abutment consisted of a concrete block with a rectangular foot print measuring 133 ft. (40.5 m) x 107 ft (32.6 m). The abutment is founded on a diabase bedrock at an approximate elevation of -12.00 ft. (-3.6 m). The abutment is stepped at elevations -6.00 ft. (-1.8 m) and +6.00 ft. (+1.8 m) reducing the plan dimensions of the abutment to approximately 111 ft. (33.8 m) x 70 ft. (21.3 m) Figure 2 shows a schematic of the east elevation of the abutment. Between elevations -6.00 ft. (1.8 m) and +6.00 ft. (1.8 m) the abutment was faced with rough-cut granite.

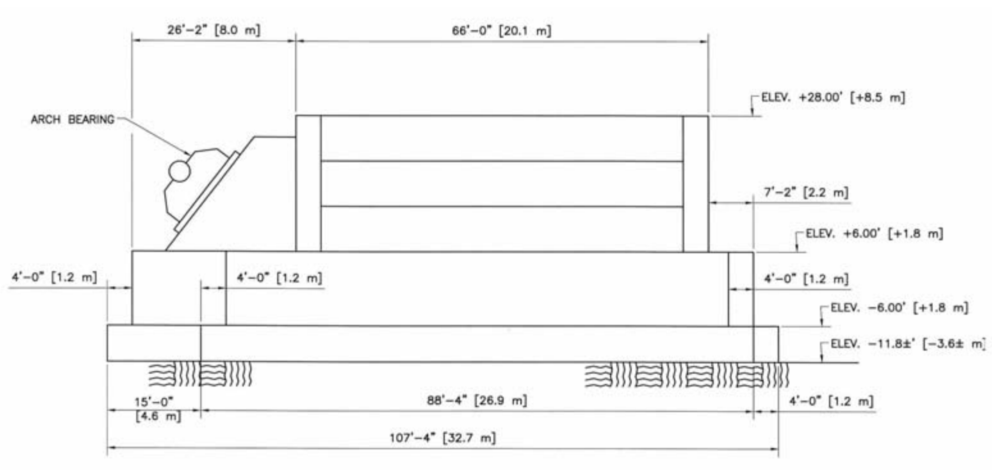

Figure 2. Abutment east elevation

The original abutment had heavy reinforcement only in the area surrounding the arch bearings. Only one layer of shrinkage reinforcement was used along the other exposed surfaces of the abutment.

SYMPTOMS OF THE PROBLEM AND EARLIER MITIGATION ATTEMPTS – Beginning decades ago, extensive random surface cracking on all vertical surfaces of the abutment as well as deterioration of the top layer of concrete at the top of the abutment were observed. Rehabilitation attempts using surface repairs and sealing techniques were made in the 1970's and mid 1980's.

The late 1970's rehabilitation of the abutment called for replacing the deteriorated outer 30 in. (762 mm) layer of concrete. The mid-1980's rehabilitation involved spraying the vertical faces of the abutment with a 1 inch (25 mm) thick layer of shotcrete. In spite of these efforts the deterioration continued with widespread random cracking and staining of the shotcrete and, in some areas, separation of the shotcrete from the original concrete. Figure 3 shows the staining and random cracking of the vertical surfaces of the abutment following the application of the shotcrete.

Figure 3. A view of the north side of the abutment at the start of the project

A Study of the Concrete Core Samples

In an effort to establish the degree of deterioration of the concrete throughout the abutment, a total of ten, full depth, 5 in. (127 mm) diameter, vertical cores were drilled in different areas of the abutment.

The samples from each core were arranged into a 40 ft. (±) (12.2 m) long concrete cylinder (see Figure 4). The cylinders were visually inspected and defects in the concrete were documented. In addition to the visual inspection, the drilling logs and the observations made during drilling were reviewed.

Figure 4. Vertical cores during inspection in the PANYNJ materials laboratory

The core holes in the abutment were also inspected using video equipment. The images captured by the video camera were recorded and the tape was reviewed. The relevant observations were documented and the major observations are listed below.

- Several wide cracks and voids with an estimated width of about 1.0 inch (25 mm) or more could be seen. In most cases, these cracks were seen extending around the full circumference of the holes.
- Many smaller defects were also noticed. The quality of the images and the orientation of the camera did not allow a clear look into the nature and dimensions of these defects.
- Water could be seen leaking from some cracks.
- In some cases the hole could not be dewatered despite having a pump running continuously. It was concluded that sea water penetrated the interface between the concrete and bedrock.

Many of the observed defects in the concrete were consistent with Alkali-Silica Reactivity. PANYNJ conducted petrographic and microscopic testing of some samples from the cores taken from the abutment. The conclusions from the petrographic testing can be summarized as follows:

- ASR gel existed in all samples.
- All samples showed signs of extensive cracking. In some cases, cracks started near the surface of the aggregate breaking a portion of the paste away from the remaining aggregate. In other cases, the cracks initiated inside the aggregate and extended into the paste.
- The bond between the cement paste and the aggregate was weak.
- The estimated water/cement ratio for all samples (0.6 ± 0.05) was characterized as moderately high to high. However, the degree of cement hydration did not appear to be uniform.

Compressive strength and modulus of elasticity testing was conducted in the PANYNJ Materials Laboratory. In all, eighteen samples were tested. Test results of the concrete samples indicated that the average compressive strength was 3100 psi. (21.4 MPa). The highest measured compressive strength was 3917 psi (27.0 MPa) and the lowest strength was 1775 psi (12.2 MPa). The compressive strength of the remaining samples varied from 2600 psi (17.9 MPa) to 3590 psi (24.8 MPa). Five samples failed in the modulus of elasticity testing. A sample was assumed to fail if it exhibited inelastic behavior before reaching 40% of the compressive strength. Among the 13 samples that passed the modulus of elasticity testing, the average measured modulus of elasticity was 1.45×10^6 psi (1.0E+04 MPa) with the highest modulus of elasticity being 3.05E+06 psi (2.1E+04 MPa) and the lowest being 0.95E+06 psi (0.66E+04 MPa). The modulus of elasticity of the remaining samples varied from 1.05 E+06 psi (0.72E+04 MPa) to 1.75E+06 psi (1.2E+04 MPa).

The expected modulus of elasticity for normal weight concrete with compressive strength between 3000 psi (20.7 MPa) and 4000 psi (27.6 MPa) is 3.1E+06 (2.1E+04 MPa) to 3.6E+06 psi (2.5E+04 MPa). Only one of the 18 tested samples approached these values. The low modulus of elasticity and the low compressive strength of some samples indicate a very poor quality concrete. The large variation in the compressive strength and modulus of elasticity indicated that the abutment concrete was not homogenous. Two causes of this large variation were suspected: variable degree of ASR damage as confirmed by petrographic testing and/or variation in the initial quality of the concrete.

Based on the results of the compressive strength and modulus of elasticity testing, it was concluded that the low values indicate an advanced degree of deterioration. Considering the age of the concrete, approximately 70 years, it was also concluded that ASR was likely to have consumed the majority of the alkalinity of the cement and that it is not likely that further significant ASR-related deterioration would take place. These conclusions could only be confirmed by long-term testing of the concrete which is conducted over a period of about 18 months. During the test, saturated concrete samples are monitored for dimension changes associated with active ASR. The amount of expansion of the samples is an indication of the remaining ASR activity. The lack of expansion of the samples is interpreted as an indication that no further ASR damage is expected.

Concrete samples from the abutment were subjected to long-term testing in the PANYNJ Materials Laboratory. While the test results indicated that limited ASR activities were expected to continue, the results confirmed the initial conclusion that the remaining ASR activities would not result in further significant ASR damage.

Abutment Condition Assessment

The information gathered from concrete test results, visual inspection of the abutment, and video inspection of the core holes all indicated that the abutment concrete had suffered damage due to ASR activity. It was certain that the severity of the damage varied depending on the exposure condition (mainly the exposure to moisture) and the local concentration of the basic ingredients of ASR, i.e., alkalinity of the Portland cement and the reactive silica in the aggregate.

The observations during drilling and testing also indicated that cracking of the concrete was not limited to the surface of the abutment and some of the cracks appeared to extend for a significant distance into the concrete. The drill wash water was seen pouring from the abutment as far as 35 ft. (10.7 m) away from core holes. In addition, relatively large voids were observed during the video inspection of the inside of the holes. It was evident that sea water was penetrating the abutment because of the water that was continuously refilling the drained holes.

No evidence of deformations or movements was detected in the abutment. It was clear that there was no immediate danger of a major failure. However, the condition of the concrete in the lower areas of the abutment was not known as these surfaces are not exposed and could not be inspected.

The need for rehabilitation was considered. The major factors considered in determining the need for rehabilitation were: the available information on the condition of the abutment, the importance of the bridge as a major transportation link, and, the need for maintaining uninterrupted traffic flow on the bridge. Additional consideration was given to the cost and even the impracticality of replacing the abutment should the deterioration continue and reach a stage that precluded rehabilitation and required replacement of the abutment.

The combination of all these factors led to the conclusion that the abutment should be rehabilitated. Implementing a rehabilitation program that improved the condition of the abutment and reduced the possibility of renewed or further deterioration of the concrete was deemed to be in the best interest of the public.

Rehabilitation Alternatives

The deterioration of concrete due to ASR requires the presence of ASR ingredients, moisture and temperature above 40 degree Fahrenheit. Testing proved that ASR ingredients existed in the abutment, thus, had the potential for further damage. In addition, the lower portions of the abutment are submerged in water that is likely to be above 40 degrees Fahrenheit (10 degrees Centigrade). These factors combined led the design team to conclude that alternatives which goal was to stop further ASR reaction, such as sealing the abutment to stop moisture penetration, were not feasible.

Other potential rehabilitation alternatives studied are summarized below.

LITHIUM DIFFUSION **-** Soaking ASR-prone concrete in a solution of lithium hydroxide or lithium nitrate allows the lithium to replace the sodium, potassium and calcium in the concrete. Similar to the materials it replaces, the lithium reacts to the alkalinity of Portland cements, producing ASR gel. However, the resulting gel is not expansive and, therefore, does not lead to the same degree of concrete deterioration suffered from expansive ASR gels. This alternative was deemed unsuitable due to the small penetration depth relative to the dimensions of the abutment. The penetration depth is estimated to be 1 to 2 inches (25 to 50 mm)and may be increased to 12 inches (305 mm) when improvements to the application methods are used.

REPLACING THE CONCRETE IN THE ABUTMENT - Based on the information shown on the original contract drawings, the dead load reaction on each of the main arch bearings is 11,650 kips (5,300,000 kg) in the vertical direction and 16,373 kips (7,440,000 kg) in the horizontal direction. Due to the high loads, particularly the horizontal loads, supporting the bridge temporary supports and replacing the abutments was deemed unfeasible.

REINFORCED CONCRETE PASSIVE ENCASEMENT - Providing a reinforced concrete encasement around the original abutment was expected to minimize water penetration to the top portions of the abutment. If further ASR expansion takes place, the encasement would be subjected to lateral loads that are not well defined as the percentage of ASR that already took place can not be determined with a reasonable degree of certainty. Designing the encasement to resist the full internal pressure from ASR would require providing means of support such as buttresses and/or post-tensioning tendons passing through the original abutment. The success of this alternative was deemed uncertain.

TRI-AXIALLY POST TENSIONING THE ABUTMENT - ASR-related concrete cracking is caused by the expansion of ASR gel when wet. The expansion of the gel is estimated to produce about 300 psi (2 MPa) of internal pressure. Should the ASR continue, applying external, tri-axial compressive stress higher than the internal pressure held the promise of arresting further concrete cracking in all directions. In addition, the confinement of the concrete that was already cracked is expected to improve its condition. Horizontal post-tensioning tendons installed inside drilled holes in the abutment and vertical rock anchors installed in holes drilled in the concrete and then continued into bedrock would provide the required compressive stress. This alternative had the following advantages:

- This alternative promised the prevention of further cracking of the concrete.
- Many of the existing cracks and voids in the concrete and the voids along the interface between the abutment and bedrock will be filled with grout when the tendons are grouted. This improved the condition of the concrete and ensured the integrity of the abutment.
- The improvement of concrete condition and the promise of stopping further cracking were not limited to the surface layer as in some of the other options.

It was concluded that tri-axially post-tensioning the abutment was the most feasible alternative.

Analytical Study of the Stress Distribution in the Abutment

A three-dimensional finite element model of one-half of the abutment was analyzed to determine the stress distribution under existing loads. Solid elements were used to model the abutment. The loads from the bridge were applied at the existing column and bearing locations. The model took advantage of the symmetry of the abutment and, thus, only one half of the abutment was included in the model. Figure 5 shows a view of the finite element model. Figure 6 shows the contours of the stresses in the abutment due to the arch reaction.

In addition, several models representing one corner of the abutment were analyzed. These partial models were used to study the stress distribution in the concrete due to the application of high post-tensioning forces at discrete locations. This information was used to optimize the size and spacing of post-tensioning tendons as well as the thickness of new concrete required to reduce the stresses on the existing concrete to within the limits allowed by the low strength of the existing concrete.

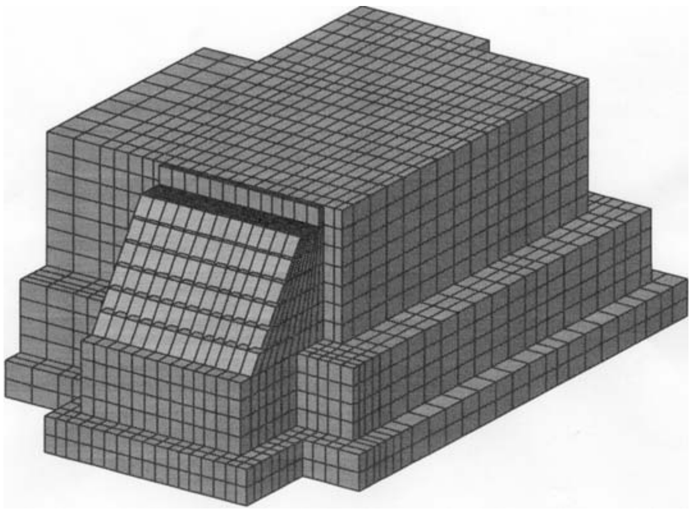

Figure 5. Three dimensional finite element model of one-half of the abutment

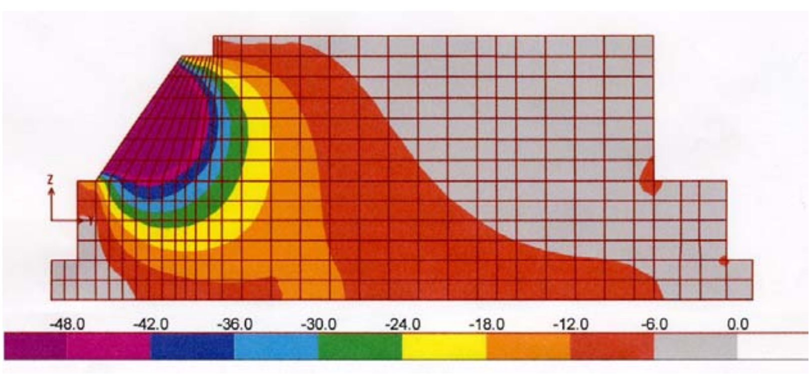

Figure 6. Stress distribution due to arch bearing reactions discrete locations.

Selected Rehabilitation Scheme

The study concluded that the most feasible rehabilitation alternative was tri-axially post-tensioning the original abutment to counteract the internal pressure caused by the expansion of the ASR gel. The design included the following features:

- A concrete encasement was specified to be constructed around the original abutment. This encasement is meant to minimize the moisture penetration of the original concrete. It also allowed smaller bearing plates for the post-tensioning tendons as the bearing plates would rest on the new concrete which has a compressive strength significantly higher than the original concrete. The post-tensioning force would also be distributed through the encasement thickness, thus applying approximately uniform stress on the surfaces of the original concrete.
- The tendons and rock anchor forces were designed to provide 320 psi (2.2 MPa) of compressive stress on all surfaces of the original concrete. To minimize the number of tendons, the force per tendon was maximized by using 0.6 in. (15.2 mm) diameter strands. The largest tendons allowed for the horizontal tendons and rock anchors to be arranged in a 7 ft. x 7 ft. (2.1 m x 2.1 m) grid. However, the final spacing varied to accommodate the geometry of the abutment. To minimize the possibility of errors in the field, a limited number of tendon sizes were used; 56, 46, 40 or 32 strand tendons. Similarly, rock anchor sizes were limited to 60, 54, 46, 40 or 36 strands.
- To minimize the number of drilled holes in the abutment, the thickness of the encasement was increased in some areas to allow for the placement of ducts and steel pipes in the new concrete to accommodate some of the horizontal tendons and rock anchors, respectively. This resulted in the thickness of the encasement varying from 3'-3" (0.990 m) to 6'-0" (1.8 m).

  The total number of horizontal tendons was 181 tendons; of which 143 require drilling in concrete with the others installed in ducts placed in the encasement walls. The drilled horizontal tendon holes in the upper areas of the abutment (54 tendon holes) are 7 in. (178 mm) in diameter while the ones in the lower parts of the abutment (89 tendons) are 8 in. (203 mm) in diameter. The larger diameter for the lower holes was required to accommodate a corrugated duct to be inserted in the holes. These ducts were specified to provide another layer of corrosion protection to the strands where the tendons are located in or near the areas of the abutment submerged in water.
- The design includes 254 vertical rock anchors. With regard to drilling, the rock anchors may be divided into the following three groups:
  - **Group 1:** Anchors installed in steel pipes embedded in the encasement walls reaching bedrock. These anchors only require drilling in bedrock. This group consists of 52 anchors.
  - **Group 2:** Anchors installed in steel pipes embedded in the encasement walls but not reaching bedrock. These anchors require drilling a portion of their length in existing concrete then continuing into bedrock. This group consists of 33 anchors.

- **Group 3:** The remaining 169 anchors require drilling throughout their full length. Drilling starts by coring through the top of the encasement followed by drilling through existing concrete and then continuing into bedrock.

  Two sizes of hole diameter were specified, 12 in. (305 mm) diameter for 36-strand anchors and 14 in. (356 mm) diameter for all other anchors.
- Only core drilling was allowed when drilling in the new concrete in order to minimize the damage to the new concrete during drilling.
- Low alkaline grout was specified to minimize the possibility of replenishing the alkalinity content of the concrete; which would result in further ASR.
- Twenty-nine, 4 in. (102 mm) diameter, vertical drilled holes reaching to bedrock were specified for pressure grouting. These holes were grouted under 200 psi (1.4 MPa) pressure. This process, referred to as the initial pressure grouting, was used to fill as many cracks and voids in the original abutment before drilling the holes for the rock anchors and horizontal tendons. This was intended to minimize problems with the grout leaking between the horizontal tendon and rock anchor holes when grouted.
- A grout pressure of 100 psi (0.7 MPa) was specified for the post-tensioning tendons and rock anchors. This was meant to aid in filling any remaining cracks in the concrete.
- High density polyurethane corrugated tubes were specified to encase the full length of all rock anchors and to encase the horizontal tendons in the lower portions of the abutment. These tubes will serve as another layer of corrosion protection.
- The grout inside the anchor heads was inspected. When voids are found, the use of vacuum grouting was specified to ensure these voids are filled.

SECIAL DRILLING REQUIREMENTS

The accuracy of drilling was of a great concern to the design team. Tight drilling tolerances had to be maintained to eliminate the possibility of drilled holes intersecting and causing damage to previously-installed tendons or rock anchors. For the horizontal drilled holes, the project specifications called for drilling tolerances of 2% of the length of the tendon but not to exceed 9 in. (229 mm). With the longest holes, approximately 145 ft. (44.2 m) long, the 9 in. (229 mm) tolerance represented 0.5% of the hole length. For the vertical holes, the specified drilling tolerance is 2% of the length of the hole but not to exceed 5 in. (127 mm) in existing concrete and 10 in. (254 mm) in bedrock.

To achieve the tight tolerances, the drilling contractor devised a system to continuously monitor and correct the direction of drilling. This system included two major components; (1) the use of special drill bits that allow for continuous corrections to the direction of drilling and (2) the placement of a light emitting diode (LED) target on the back of the drill bit. Surveying equipment was used to point a laser beam through the hole along its specified axis of the hole. The location of the laser beam on the LED target indicated the magnitude and direction of the hole misalignment. By continuously monitoring the misalignment and continuously correcting the drilling direction, most horizontal holes were within 5 inches from the specified location.

The driller selected to use the down-the-hole-hammer method to drill through existing concrete. This method has relatively high production rate compared to coring. As mentioned earlier, only coring was allowed in new concrete.

The design team was concerned about damage to the concrete as the drill bit exits the concrete at the exit end of the hole. To eliminate this damage, the drill was stopped before reaching the new concrete at the exit end. Knowing the location of the hole inside the concrete, a core drill was placed at the far end and was used to back-core toward the drilled hole. The location of the center of the cored hole at the surface of the encasement concrete was determined based on the location of the end of the drilled hole and the direction of the drilled holes. This allowed the drilled and cored hole to meet smoothly without having a "kink" in the hole. Figure 7 illustrates the drilling procedure at the exit end of the horizontal holes.

ROCK ANCHOR TEST INSTALLATION

To ensure the adequacy of the rock anchors design, the design team requested a full-size rock anchor test at the job site. The rock anchor test setup consisted of a concrete block 17 ft. x 15 ft. x 8 ft. (5.2 m x 4.6 m x 2.4 m) constructed on bedrock adjacent to the abutment

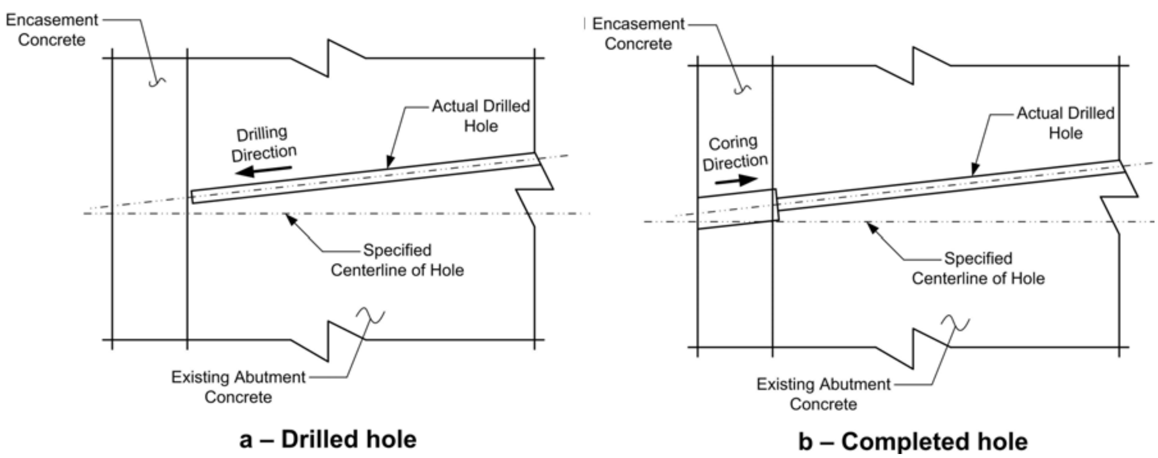

Figure 7. Drilling procedure for horizontal holes

inside the cofferdam. Four vertical holes were drilled through the concrete block and into bedrock. Rock anchors representative of the anchor sizes to be used in the abutment were installed in the four holes. Drilling, installation, grouting and anchor stressing and testing procedures specified for production anchors were used for the test anchors. Test results of the four anchors showed the adequacy of the rock anchor design to achieve the specified pretensioning loads.

CONSTRUCTION SEQUENCE

The design called for the following construction sequence:

- Construct a cofferdam around the abutment and dredge the space between the two. Figure 8 shows the cofferdam during construction. The area enclosed by the cofferdam is kept dry by continuously pumping the water that seeps in.
- Map and remove the existing granite facing that covers the lower portion of the abutment. Measure, clean, mark and store granite blocks.

Figure 8. Cofferdam cells during construction

- Demolish the top 3.25 ft. (990 mm) of concrete at the top of the abutment. At the locations of the steel columns supported on the abutment, jack and support the columns on temporary jacking frames during demolition and replacement of the concrete under the columns. Remove a 3 inch (75 mm) thick layer of concrete from all other surfaces of the abutment.
- Construct the concrete encasement around the abutment. Figure 9 shows the abutment during construction of the encasement.
- Drill 29 vertical holes, 4 inches (102 mm) in diameter, to reach bedrock. Grout the holes using low-alkali grout under 150 to 200 psi (1.0 to 1.4 MPa) grout pressure. This initial grouting process was meant to fill many of the existing cracks and voids in and under the original abutment.
- Drill the horizontal holes and install the tendons in the drilled holes and in the ducts that were placed in the encasement walls. A grout pressure of 100 psi (1.0 MPa) was specified for grouting the horizontal tendons. Using this pressure was intended to fill the cracks and voids that were not filled during the initial pressure grouting. No more than 10 drilled holes were allowed to be open at any point of time and no open holes were allowed within 11 ft. (3.3 m) from each other. The latter requirement was meant to minimize the possibility of the grout leaking from one hole to the surrounding open holes during the grouting process.

- Construct the rock anchor test setup. Install and test the four test rock anchors. Based on the test results, adjust the length of the rock anchors bond length if required. Construct the rock anchor test setup. Install and test the four test rock anchors. Based on the test results, adjust the length of the rock anchors bond length if required.

Figure 9. Construction of the encasement

- Construct the rock anchor test setup. Install and test the four test rock anchors. Based on the test results, adjust the length of the rock anchors bond length if required.
- After completing the installation of the horizontal tendons and reviewing the drilling records to ensure that the horizontal holes are within the specified tolerances, drill the vertical rock anchor holes and install the anchors. No more than 20 open holes were allowed at any time and no open holes were allowed within 11 ft. (3.3 m) from each other. Figure 10 shows the top of the abutment after completion of the rock anchors.
- Reinstall the granite facing including additional granite blocks required due to the increased size of the abutment.
- Install miscellaneous items such as fences, railings and drainage components.
- Fill the space between the cofferdam and the abutment, re-grade the fill to the original ground elevation and remove the cofferdam.

Figure 10. Photo of the top of the abutment after completion of work

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