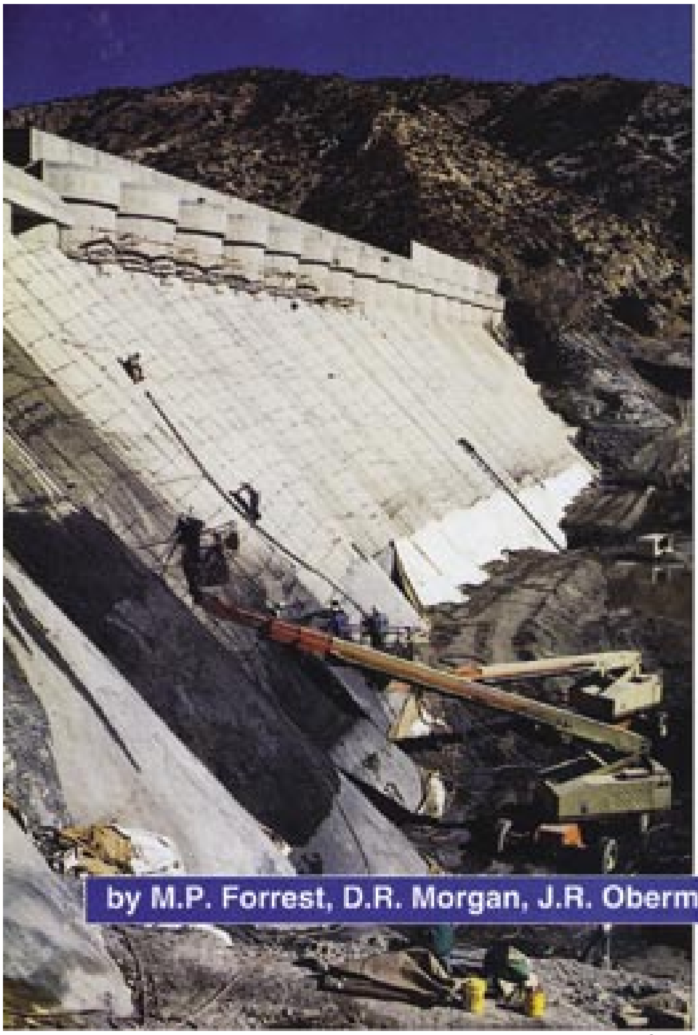

Photograph by Frank Collett

Shotcrete overlay does the job...

Seismic Retrofit of Littlerock Dam

by M.P. Forrest, D.R. Morgan, J.R. Obermeyer, P.L. Parker, and D.D. LaMoreaux

Littlerock Dam, located in the Angeles National Forest in southern California, was originally designed and constructed as a multiple-arch structure. Consisting of 28 arches, it has a maximum height of 190 ft (58 m) and a crest length of 720 ft (220 m). It was designed by John S. Eastwood, a pioneer in the design and construction of multiple-arch dams. When it was completed in 1924, Littlerock Dam was the highest multiple-arch dam in the United States. The dam provides a vital water supply for both the Palmdale Water District and the Littlerock Creek Irrigation District.

Throughout its life, there have been concern and controversy about the adequacy of its design, overall stability, and safety because the dam is located 1.5 miles (2.4 km) south of the San Andreas fault. The Maximum Credible Earthquake (MCE) originating on this fault is a moment magnitude 8 event resulting in a peak horizontal site acceleration of 0.7g. The results of stability and stress analyses completed by the California State Division of Safety of Dams and Woodward-Clyde Consultants showed that the dam did not meet required seismic safety criteria, principally because of its lack of lateral stability, a deficiency which is inherent in multiple-arch dams.

To provide adequate seismic stability of the dam, Woodward-Clyde Consultants of Oakland, California, developed a rehabilitation design that consisted of economical uses of (1) roller compacted concrete (RCC) to construct a gravity section between and around the downstream portions of the existing buttresses, and (2) bonded steel fiber reinforced shotcrete to stiffen the arches of the existing dam.

The RCC buttress was constructed between May 1993 and April 1994, and is described by Wong et al.[1, 2] This article describes the design basis, specification requirements, preconstruction testing, construction monitoring, construction testing, construction schedule, and cost summary for the shotcrete overlay. Woodward-Clyde Consultants constructed the overlay on a turnkey basis that included design, construction, management, and quality control. Agra Earth & Environmental Limited provided construction monitoring and quality control testing services as a subconsultant to Woodward-Clyde. The construction was completed in a three-month period between August 29 and November 29, 1994.

Design basis

Stress and stability analyses were performed to evaluate the design of the composite dam section. The analyses involved an iterative process, and the strengthening design evolved with it. During the analyses, it became apparent that the existing arched sections needed to be strengthened. The analyses were performed assuming a bonded 4 in. (100 mm) thick shotcrete overlay on the arches.

Finite element analyses were used to compute the stresses resulting from normal operating pool, Probable Maximum Flood (PMF), and MCE loading conditions within the composite dam and RCC buttress. The analyses were carried out to verify that the proposed dam modification would perform safely under critical static and dynamic loadings. Static loads included gravity, hydrostatic pressures, silt loads, uplift, and temperature loads. Dynamic loads included the inertia forces and hydrodynamic pressures resulting from the earthquake ground motions synthesized for the MCE.

The basic approach of the structural design was to remedy the lack of lateral stability of a multiple-arch dam by providing a continuous support system in the form of an RCC gravity section and shotcrete-bonded overlay to stiffen the arches. To evaluate the effectiveness of the composite structural scheme, a 3-D model was used, which spanned the maximum (most critical) bay and the two adjacent half-bays. Linear plate and solid elements were used to model the existing structure and the RCC buttress, respectively. The model for the design had fundamental frequencies of 3.0 Hz, 10.9 Hz, and 13.6 Hz, corresponding to the cross-channel, upstream-downstream, and vertical responses, respectively. The peak spectral accelerations occurred at frequencies between 4 and 5 Hz.

No overstressing was computed for any static load case. The results of dynamic time-history analyses using the modal superposition method based on 90 modes indicated that compressive stresses were well below the strength of all structural components. Tensile stresses exceeding the tensile strength of 300 psi (2 MPa) were computed near the tops of the arch barrels, but the overstressing was transient and only occurred a few times during the synthesized earthquake. The maximum shear stress computed at the interface between the shotcrete and concrete was 88 psi (0.61 MPa). All stresses in the RCC buttress were small, with tensile stresses less than 100 psi (0.7 MPa). Maximum dynamic deflection at the crest was less than 1 in. (25 mm).

In summary, the results of the stress and stability analyses indicated that:

- The arch barrels, wall buttresses, and the RCC gravity section are expected to maintain overall structural integrity under all anticipated loading conditions. During the MCE, localized cracking is expected to occur in the shotcrete-reinforced arch barrels, but not in the wall buttresses or in the RCC gravity section.
- The shotcrete-reinforced arch barrels, wall buttresses, and the RCC gravity section are expected to provide adequate margins of safety against sliding and overturning under all anticipated loading conditions.

Specification requirements

The seismic retrofit design for the arch component of the dam required a layer of shotcrete on the upstream side of the arch barrels to stiffen them and increase the tensile strength of the arch faces, where tensile stresses are predicted to be highest during earthquake shaking. Successful implementation of the design was predicated on achieving full bond between the existing arch barrels and the applied steel fiber reinforced shotcrete. Consequently, stringent specifications were prepared and rigorous construction monitoring and quality control testing procedures implemented to address this issue. Also, a system of anchors with reinforcing steel spanning vertically and circumferentially between the anchors was specified.

Surface preparation

Proper preparation of the concrete arch surfaces was essential to providing good bond. The tensile bond strength of 150 psi (1.0 MPa) was required to provide adequate strength and to transfer shear stresses at the interface between the concrete and shotcrete to resist dynamic, thermal, and shrinkage stresses.

The four components of the surface preparation were an adequate roughness profile, suitable moisture condition, cleanliness, and hardness.

Roughness profile — The arch surfaces were to be prepared to expose, but not undermine, the concrete aggregate. The required roughness profile (peak-to-valley amplitude) was 3/16 in. (5 mm). The roughness profiles were measured to document that the specified amplitude was being achieved.

Moisture condition — The arches were to be prewetted continuously for at least 24 hours prior to shotcreting. The surface moisture of the concrete at the time the shotcrete was applied was not to be too wet (a sheen or glisten of free surface water) nor too dry (no moisture present). Adequate surface moisture was specified to be a saturated surface dry (SSD) condition.

Cleanliness — The surface of the concrete had to be free of dust and laitance. The surface cleanliness was checked by wiping the prepared concrete surface with a dark cloth and observing if any dust was present.

Hardness — Adequate hardness was achieved when the prepared concrete surface could not be gouged with a knife blade.

Anchorage, reinforcing

Although bond between the shotcrete and concrete was the prime emphasis of the design, additional benefits were accrued through the inclusion of an anchorage and reinforcing system in the design. The anchor and reinforcing system provided enhanced durability of the shotcrete overlay with time by providing increased resistance to 1) delamination of the shotcrete at and below the shotcrete/substrate concrete interface during a seismic event, and 2) shear and tensile stresses at and below the shotcrete/substrate interface imposed by shrinkage stresses, moisture gradient (curling effects), and thermal effects. Many failures of shotcrete overlays are the result of delamination within the concrete substrate just below the shotcrete/concrete interface.

The anchor system design was adapted from other designs used to anchor shotcrete overlays to concrete.[3] The anchor system consisted of a 4 ft (1.2 m)

Fig. 1: Sandblasted concrete in the arch barrels. The profile gauge was used for measuring the surface roughness profile. Note the exposed texture of the prepared surface

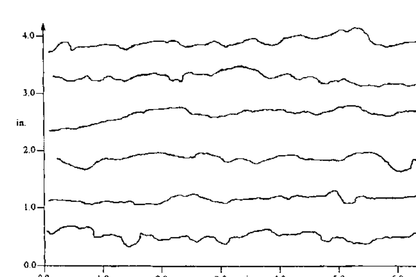

Fig. 2: Typical surface roughness profiles

Fig. 3: General view of shotcrete application from a boomlift to test panel

square grid pattern of anchors drilled and grouted into the face of the dam and connected with steel reinforcement. The anchors consisted of L-shaped Grade 60 No. 4 reinforcing bars that extended 9 in. (229 mm) into the substrate concrete. The hole depth was based on developing the full strength of the steel bar; i.e., bond failure was not allowed. The anchor bars were grouted into 5/8 in. (16 mm) diameter holes with epoxy adhesive. Grade 60 No. 3 reinforcing bars spanned vertically and circumferentially between the anchors. The anchors were tested to evaluate performance.

Shotcrete

Design considerations for the shotcrete overlay were compressive, tensile, and flexural strength, as well as freeze-thaw durability and good bond to the substrate concrete.

The specified 28-day compressive strength was 6000 psi (41 MPa). To impart toughness to the shotcrete, the mix included 100 lb/yd^3 (60 kg/m^3) of steel fibers. The minimum cement content was 675 lb/yd^3 (400 kg/m^3). Silica fume was included in the mix at the rate of 10 percent by weight of cement. The silica fume was added to increase durability and strength, improve bond strength to the existing concrete, and reduce the amount of rebound. Freeze-thaw durability was provided by requiring air entrainment of the shotcrete mix. An air content of 10 to 12 percent at the shotcrete pump was specified so that the resulting as-shot air content would be 5 ± 1 percent.

Preconstruction testing

Surface preparation

Test areas were prepared on two of the arch barrels by sandblasting. The surface roughness profile on the substrate concrete was checked using a profile measuring gauge (Fig. 1). Typical surface roughness profiles are shown in Fig. 2. The surface was deemed acceptable if no undercutting of the aggregate was observed and if five of the six profiles measured in a given area met the following requirements:

- Three peak-to-valley measurements of 3/16 in. (5 mm) exist in 6 in. (150 mm) of measured length, or
- Five peak-to-valley measurements of 5/32 in. (4 mm) exist in 6 in. (150 mm) of measured length.

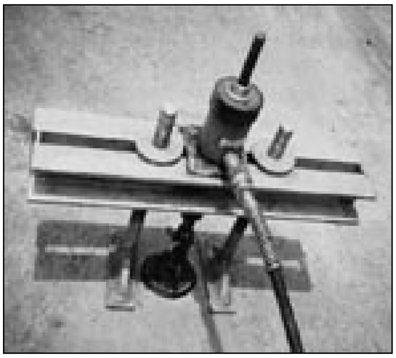

Fig. 4: Installed bond pull-off test apparatus

Bond testing

Wooden forms were mounted on the face of the prepared concrete test areas, and the concrete was then saturated with water for 24 hours prior to shotcrete application. Fig. 3 shows the shotcrete being applied in one of the 5 ft (1.5 m) square test areas to a saturated surface dry (SSD) substrate concrete. These shotcrete test panels were produced for the purpose of determining the bond pull-off strength (in direct tension) between the shotcrete and the substrate concrete.

At ages varying between 28 and 29 days, thirteen 4 in. (100 mm) diameter cores were drilled into the test panels and bond pull-off tests were conducted. Fig. 4 shows a close-up view of the installed bond pull-off test apparatus. With the exception of two low test results that were influenced by some curling, all bond pull-off stresses exceeded the specified minimum of 150 psi (1.0 MPa). They ranged from 167 to 363 psi (1.15 to 2.50 MPa) and averaged 209 psi (1.4 MPa), including the two low test results.

Core grading

To assess their ability to properly place consolidated shotcrete, the nozzlemen were required to shoot preconstruction test panels, both at inclined and vertical positions, representative of the work. Cores were extracted from these test panels for core grading. These panels contained anchors and reinforcement representative of the design.

The cores were graded using a core grading system similar to that given in the new ACI 506.2-94 shotcrete specification. The project specifications required all cores to have a core grade less than Grade 3. All cores tested were graded either Grade 1 or 2, and the nozzlemen were approved for the project.

Shotcrete testing

Cores without any embedded reinforcing steel were extracted from the test panels for assessment of conformance of the supplied shotcrete to the project specifications. Test performance data are given in Table 1. At age 21 days, all core compressive strengths exceeded the minimum specified 28-day strength of 6000 psi (41 MPa).

Extracted cores were also tested at age 21 days for absorption after immer-

Table 1: Preconstruction shotcrete core test results (at age 21 days)

Sample orientation	Compressive strength, MPa (psi)	Absorption after immersion and boiling, percent	Volume after permeable voids, percent
Inclined	42.8 (6200)	5.3	11.8
	53.0 (7690)	—	—
	51.1 (7410)	5.2	11.6
Avg.	49.0 (7080)	5.3	11.7
Vertical	49.2 (7140)	5.1	11.5
	46.3 (6720)	—	—
	47.8 (6930)	5.2	11.5
Avg.	47.8 (6930)	5.1	11.5

sion and boiling, as well as volume of permeable voids, in accordance with ASTM C 642-90 (Table 1). All cores displayed values of absorption after immersion and boiling of 5.3 percent or less and volume of permeable voids of 11.5 to 11.8 percent. According to the criteria suggested by Morgan,[4] these numbers indicate shotcrete of "excellent" quality.

The air content of the shotcrete discharged from the transit mixer was 8.3 percent. The air content as shot (measured by shooting into an air pressure meter base) was 4.6 percent. This was within the specified air content range of 5 ± 1 percent after shooting. The specifications were consequently changed to allow concrete to be supplied at an air content of 8 to 10 percent as discharged from the transit mixer. Given the satisfactory air entrainment, compressive strength, and "excellent" quality of consolidation, this shotcrete is expected to display good freeze-thaw durability in the field.[5]

Toughness

Steel fiber reinforcement was specified for the shotcrete to enhance the resistance of the bonded shotcrete to restrained drying shrinkage cracking.[6] In addition, the fiber reinforcement provides enhanced toughness (energy absorbing capacity) and resistance to cracking and damage in a seismic event.[7] Different types of fibers provide different levels of toughness performance. Consequently, ASTM C 1018 toughness testing was specified to install suitably "tough" shotcrete. ASTM C 1018 toughness indices of $I_5 \geq 3.5$, $I_{10} \geq 5.0$, and $I_{30} \geq 16$ were specified. Typical load vs. deflection curves for vertical and inclined test panels are shown in Fig. 5. With the addition of 100 lb/yd^3 (60 kg/m^3) of Bekaert Dramix ZL 30/.50 steel fibers, the specified ASTM C 1018 toughness index criteria were satisfied for both the inclined and vertical test panels.

The toughness performance was also evaluated according to criteria suggested by Morgan, et al.[8] They defined various toughness performance levels for standard 4 x 4 x 14 in. (100 x 100 x 360 mm) beams tested in third-point loading on a 12 in. (305 mm) span using the ASTM C 1018 testing procedures. According to these criteria, the shotcrete in the vertical test panels performed at a Toughness Performance Level III and the inclined test panels performed at Toughness Performance Level IV (Fig. 5). This represents a good level of toughness performance.

Anchor testing

As a final component of the preconstruction testing, anchor pull-out testing was conducted on six epoxy-grouted No. 4 reinforcing steel anchors. The full tensile capacity of the anchors was developed in 5 of the 6 anchors, which failed in tension rather than pull-out, as intended. The failure loads ranged from 18,600 lb (82.7 kN) to 20,900 lb (93.0 kN) and averaged 19,900 lb (88.5 kN). This satisfied the specified performance requirements.

On the basis of the successfully completed preconstruction testing, approval was given to proceed with the full construction program.

Construction monitoring

Surface preparation

The specifications allowed for various methods of surface preparation. Sand-

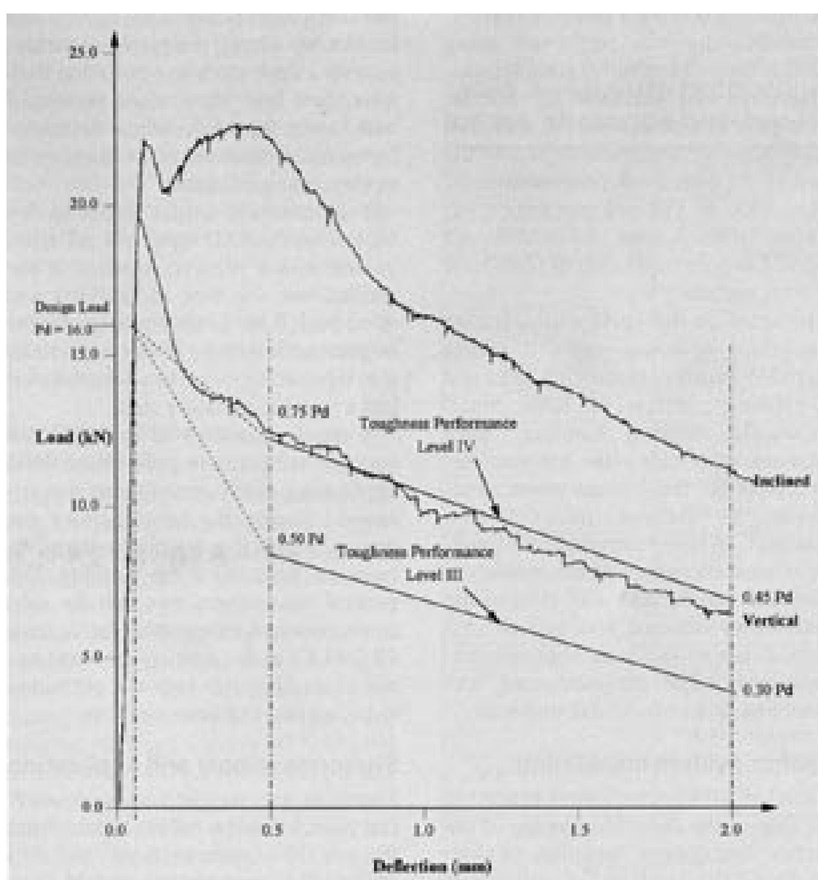

Fig. 5: Typical load vs. deflection curves in flexural toughness tests

Fig. 6: Anchors and reinforcing steel installed on the face of the dam

blasting, hydro-demolition, and waterblasting were permitted. Sandblasting was selected to prepare the arch surfaces. After a few trials, a coarse silica sand was chosen as the sandblasting abrasive. To prevent sand and dust contamination of the prepared surfaces and freshly applied shotcrete, sandblasting was scheduled so that it did not coincide with the shotcrete work. All surface preparation work was monitored during the operations using a profile gauge.

Sandblasting was performed using 1200 ft^3/min (34 m^3/min) compressors. Typically, two sandblasting nozzles were used at a time, with the operators using separate boomlifts to access the face of the dam. Peak production rates were 1000 ft^2 (93 m^2) per nozzle per 10-hour shift. A total of 27 shifts was worked to prepare 48,000 ft^2 (4500 m^2) of arch surface.

Included in the surface preparation was chipping along exposed and/or corroded existing reinforcing steel and at existing concrete lift lines. Small pneumatic chipping hammers were used to form V-cuts in the concrete. After chipping, these areas were sandblasted to remove microfractured concrete. Where corroded reinforcement was encountered, the reinforcement was augmented with steel of the same cross-sectional area spliced in a vertical lap to facilitate shotcrete encapsulation. The corroded steel was sandblasted to remove rust and scale.

Anchor system installation

A total of 3474 anchors was installed in the face of the dam. Monitoring of the anchor installation included random checks on the depth and cleanliness of the drilled anchor holes. The holes were cleaned by pressured air at the bottom of the holes and by a nylon brush. The epoxy was injected into the hole and the anchor was then inserted. Reinforcing steel was tied to the anchors in a gridwork pattern (Fig. 6).

Surface cleaning, moisture conditioning

After sandblasting, prepared areas were washed with fire hoses to remove residual sandblasting sand, debris, and dust. After wiping the prepared surfaces with a dark cloth to verify that they were dust-free, they were prewetted with water from hose-pipes or soaker hoses for a minimum of 24 hours prior to shotcrete application.

If the concrete surface started to dry back from the SSD state, it was lightly misted with a pressure washer. If the surface was too wet, the surface was dried back with an air blowpipe by the nozzleman's helper, or the nozzleman was directed to move to a location that had a suitable moisture state.

A check was kept of the substrate concrete temperature prior to shotcrete application. Such surveillance was increased during the latter part of the project when the overnight temperatures started to fall below freezing. The general requirement was that the substrate concrete temperature be at least 40 F (4 C) at the time of shotcrete application. Shotcrete was not permitted to be applied to frozen surfaces.

Shotcrete supply and application

Shotcrete was supplied by a commercial plant located 6 miles (10 km) from the job site. Loads of 8 yd^3 (6.1 m^3) each were transported in standard 11 or 12 yd^3 (8.4 or 9.2 m^3) mixers. The shotcrete was applied using a Reed 4000 concrete pump. Access to the face was accomplished by boomlifts, and for the highest portions of the dam, an articulated boomlift was used.

The application team consisted of three workers on the lift platform: the boomlift operator, who regularly checked shotcrete thickness with a spiked probe; the nozzleman, who applied the shotcrete; and the nozzleman's helper, who controlled rebound and overspray with an air blowpipe. The nozzleman typically held the gun between 4 and 6 ft (1.2 and 1.8 m) from the arch surface.

Shotcrete was applied to the arches in the following three sequence patterns:

- Shotcrete was initially applied in three 16 ft (4.9 m) high panels across the lowest portion of the dam; this was done to minimize damage to the work in the event of flooding.
- Shotcrete was then applied to the left side of the dam along the full height of the arch barrels. Unfortunately, this shotcrete application sequence complicated finishing operations and was discontinued.
- For the remainder of the project, shotcrete was applied in a lateral sequencing; several arch barrels were shot in a lateral progression up to about one-third the height of the arch barrels before returning to the first arch barrel to complete the coverage. In this way, hoses dragging on the already setting and hardening shotcrete did not damage the shotcrete, and finishing and curing crews had more room to operate.

Applying the shotcrete from the bottom up was the preferred means of completing the work because it minimized the potential for rebound and overspray contamination of the existing concrete surface. The nozzleman's helper was required to continuously use a blowpipe to prevent rebound from accumulating in areas about to receive shotcrete and to remove buildup of overspray on the reinforcing steel.

A total of 720 yd^3 (550 m^3) of shotcrete was applied to cover the 48,000 ft^2 (4500 m^2) of arch surfaces. Thirteen shifts were required to apply the shotcrete, averaging about 56 yd^3 (43 m^3) per shift with a maximum of 72 yd^3 (55 m^3) per shift. The average thickness of the applied shotcrete was about 4.9 in. (124 mm).

Shotcrete finishing

A four-person crew was generally employed for finishing; two persons used bullfloats, one person used a hand float, and one person cut construction joints with a trowel.

The surface was finished with long-handled bullfloats by a two-person crew usually operating from a boomlift (Fig. 7). Hand floating with a steel trowel was also utilized. Bullfloating and steel trowelling were effective in embedding the exposed steel fibers at the surface of the shotcrete and left a satisfactory final surface appearance.

Fig. 7: Shotcrete finishing using a long-handled metal bullfloat

The shotcrete edges at construction joints were finished to an approximate 45 degree angle by cutting with a steel trowel and then green cutting with a pressure washer to provide a textured surface conducive to good bond. The project specifications required that care be taken to avoid feather-edging at joints, and to provide the embedded reinforcing steel with the required cover.

Shotcrete curing

Rigorous attention was given to shotcrete curing, as this was considered critical to a successful shotcrete installation. A monomolecular film evaporation retardant was applied after shotcreting and before finishing. This, together with water-misting, proved to be effective in preventing plastic shrinkage cracks from developing in the fresh shotcrete, in spite of the highly evaporative conditions (warm temperatures and strong winds) prevailing at times. As soon as the shotcrete had set, hoses and pressure washers were used to keep the shotcrete wet until water-saturated, plastic-coated burlap curing blankets could be fixed to the hardened shotcrete (Fig. 8).

The project specifications required continuous water-curing for a minimum of seven days from the time of shotcrete application. As a result, there were no instances of either plastic or restrained drying shrinkage cracking in the completed shotcrete work. It appears that the fiber reinforcement and the specified curing regime has been very effective in inhibiting plastic and drying shrinkage cracking.

Construction testing

Surface preparation, shotcrete supply

Rigorous testing was conducted to assess the conformance of the sandblasted substrate concrete profile to the project specifications, using the profile gauge shown in Fig. 1. A total of 336 surface roughness profiles was measured, averaging one set of six profiles every 860 ft^2 (80 m^2). Any nonconforming areas were re-sandblasted until the surface roughness profile was in conformance with the specifications.

The same shotcrete mixture design used in the preconstruction testing was approved for use in construction (Table 2). Approximately 55 percent of all loads of shotcrete supplied to the project were tested for air content as discharged from the transit mixer and 17 percent of the loads were also tested for air content of shotcrete as shot into an air pressure meter base. Slump was also randomly tested. With the exception of the first load of shotcrete supplied to the project, all as-shot air contents were within the specified range of 5 ± 1 percent. The average as-delivered air content was 8.7 percent, while the as-shot air content averaged 4.4 percent.

Shotcrete thickness, core grading

Using a spiked probe, the boomlift operator regularly checked the thickness of the plastic shotcrete during shotcrete application. In addition, cores extracted from the dam face for bond pull-off testing and core grading were used to measure the hardened shotcrete thickness. The project specifications required the extraction of two test cores for core grading for every 5000 ft^2 (460 m^2). A total of 28 cores was extracted. The measured shotcrete thickness ranged between 3.6 and 6.0 in. (91 and 152 mm) and averaged 4.7 in. (119 mm). Note that the theoretically calculated average shotcrete thickness was 4.86 in. (123 mm), based on 720 yd^3 (550 m^3) of shotcrete applied to a 48,000 ft^2 (4500 m^2) area. A few cores with a thickness of less than the specified 4 in. (100 mm) were found, but in only two areas was the average thickness of three cores from an area less than 4.0 in. (100 mm), and then by no more than 1/4 in. (6 mm). This is within the normal range of tolerance that can be expected for shotcrete construction.

With respect to core grading, all cores were rated either Grade 1 or 2. All those tested met the grading requirements of the specifications.

Bond pull-off testing, compressive strength

The project specifications required a minimum bond pull-off strength of 150 psi (1.0 MPa). Furthermore, the specifications required that if a low test result was found, two more cores be taken within 5 ft (1.5 m) of the unsatisfactory core, and that these two cores show the specified bond strength. One bond pull-off test was required for every 5000 ft^2 (460 m^2) of shotcrete, or a total of 10 cores for the project.

In two locations, cores with bond strengths less than the specified 150 psi (1.0 MPa) were encountered, so two

Table 2: Mixture proportions for the shotcrete overlay

Material	Design mass, lb/yd^3 (kg/m^3)
Type II portland cement	682 (405)
Silica fume	70 (41)
Coarse aggregate (3/8 in. SSD)	820 (486)
Sand (SSD)	1900 (1127)
Water	338 (200)
Water-reducing admixture	2 oz per 100 lb cement (125 mL per 100 kg cement)
Superplasticizer	12 oz per 100 lb cement (750 mL per 100 kg cement)
*Air-entraining admixture	3.30 oz per 100 lb cement (205 mL per 100 kg cement)
Dramix ZC 30/.50 steel fibers	100 (60)
TOTAL	3915 (2320)
*as required for: 8 to 10 percent air content as batched 4 to 6 percent air content as shot	

Fig. 8: General view of shotcrete repair with installed curing blankets

additional adjacent bond pull-off tests were performed at each of these two locations. These cores satisfied the specified performance requirements. Overall, the shotcrete tested satisfied the project bond specifications.

The project specifications required that two test cores be extracted from the in-place shotcrete for compressive strength testing for every 5000 ft^2 (460 m^2) of shotcrete placed. A total of 26 of the core grading cores were tested for compressive strength, some for early-age indications of strength. The specified minimum 28-day compressive strength was 6000 psi (41 MPa). All cores were not tested at exactly 28 days, because of logistical constraints. Actual ages at test, ignoring the early age cores, ranged from 28 to 47 days. Compressive strengths of shotcrete older than 28 days ranged between 5740 and 9700 psi (40 and 67 MPa) and averaged 7300 psi (50 MPa). Only one core had a strength less than specified. Overall, the shotcrete cores conformed to the project specifications for compressive strength.

Construction schedule, cost summary

The shotcrete project was completed in 90 days, including all surface preparation work, anchor system installation, shotcrete application, and testing.

The sandblasting unit price was \$3.33/ft^2 (\$35.86/m^2) and the shotcrete unit price was \$3.00/ft^2 (\$32.28/m^2). The total cost of the 48,000 ft^2 (4500 m^2) shotcrete overlay, including construction, quality control, construction management, and engineering, was \$745,000.

The project was completed within budget and on time.

Acknowledgments

The design and planning input from the California Division of Safety of Dams, the Palmdale Water District and the Littlerock Creek Irrigation District is gratefully acknowledged. Construction management was provided by Thomas Scotese and design assistance was provided by David Hughes, both of Woodward-Clyde Consultants. Construction monitoring and quality control inspection and testing was provided by Philip Parker and Neil McAskill of AGRA Earth & Environmental Limited, with quality auditing by D.R. Morgan. The subcontractors included: Western Gunite, Inc., who applied the shotcrete; R.R. Evans Sandblasting, who performed the arch surface sandblasting; and Hi-Grade Materials Co., who supplied the shotcrete.

References

1. Wong, N.C.; Forrest, M.P.; and Lo, S.H., "Littlerock Dam Rehabilitation, Another RCC Innovation," Association of State Dam Safety Officials, 10th Annual Conference, Kansas City, Missouri, 1993, pp. 303-314.

2. Wong, N.C.; Forrest, M.P.; and Lo, S.H., "Improving the Seismic Safety of a Historic Multiple-Arch Dam," International Congress on Large Dams, Durban, South Africa, Q.68, R.89, 1994, pp. 1501-1519.

3. Pataky, T., and Kemp., B., "Rehabilitation of Jordan River Concrete Buttress Dam for Seismic Loads," *Concrete International*, V. 15, No. 5., May 1993, pp. 55-60.

4. Morgan, D.R.; Neill, J.N.; McAskill, N.; and Duke, N., "Evaluation of Silica Fume Shotcrete," CANMET/CSCE International Workshop on Silica Fume in Concrete, Montreal, Quebec, May 4-5, 1987, 60 pp.

5. Morgan, D.R., "Freeze-Thaw Durability of Shotcrete," *Concrete International*, V. 11., No. 8, August 1989, pp. 86-93.

6. Shah, S.P.; Sarigaphuti, M.; and Karaguler, M.E., "Comparison of Shrinkage Cracking Performance of Different Types of Fibers and Wiremesh," *Fiber Reinforced Concrete Developments and Innovations*, SP-142, 1994, pp. 1-18.

7. Katzensteiner, B.; Mindess, S.; Filiatrault, A.; and Banthia, N., "Dynamic Tests of Steel Fiber Reinforced Concrete Frames," *Concrete International*, V. 16, No. 9, September 1994, pp. 57-60.

8. Morgan, D.R.; Mindess, S.; and Chen, L., "Testing and Specifying Toughness for Fiber Reinforced Concrete and Shotcrete," Second University Industry Workshop on Fiber Reinforced Concrete and Other Advanced Composites, Toronto, March 26-29, 1995, 22 pp.

Selected for reader interest by the editors.

***Michael P. Forrest** is a project manager with Woodward-Clyde Consultants in Oakland, CA. He is principally involved with the design, construction, and rehabilitation of embankment and conrete dams.*

*ACI Fellow **Dudley R. Morgan** is chief materials engineer with AMEC Earth & Environmental, a division of AMEC Americas Limited. He specializes in concrete and shotcrete technology and the rehabilitation of infrastructure. He is a member of ACI Committees 506, Shotcreting; 544, Fiber Reinforced Concrete; 234, Silica Fume in Concrete; and C 660, Shotcrete Nozzleman Certification.*

***James R. Obermeyer** is a senior vice president of Woodward-Clyde Consultants. He is responsible for the firm's construction management activities. He is based in Denver, CO, and has extensive experience in design and construction of new dams and rehabilitation of existing dams.*

***Philip L. Parker** is a civil engineering graduate from the University of British Columbia. He works in the Materials Engineering Division of AGRA Earth and Environmental Limited and has experience in construction monitoring and quality control testing.*

***Dennis D. LaMoreaux** is the current general manager and principal engineer for Palmdale Water District and has been involved throughout the Littlerock Dam seismic retrofit project. He is a civil engineering graduate of the University of Wyoming.*