WATER OPERATION AND MAINTENANCE

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Cover photograph:

Stony Gorge Dam, Orland Project, California, is the spotlight of this issue. Aerial view of dam showing reservoir near full capacity. Photo taken before SOD (safety of dams) modifications.
Photo by J. C. Dahilig. 5/2/79

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WATER OPERATION AND MAINTENANCE BULLETIN

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JUNE 1990

THE BUREAU OF RECLAMATION USES GEOSYNTHETICS[1]

by Robert L. Dewey, P.E.[2]

The Bureau of Reclamation has used geosynthetic products in several structures over the last 10 years. Membranes have been used to line reservoirs at Mt. Elbert Forebay, Colorado, and San Justo Reservoir, California. Prefabricated ("wick") drains were used at Jackson Lake Dam, Wyoming, to increase drainage in the dam foundation being densified by dynamic compaction. Also at Jackson Lake, the downstream slope of the embankment was protected by a reinforced grass slope, and a geogrid was placed across the base of the dam to minimize cracking in the event of an earthquake. Geogrids were designed into Davis Creek Dam, Nebraska, to allow for a steeper downstream slope. At Merritt Dam, Nebraska, a geotextile was designed for use as a filter in a seepage collector along the left abutment downstream. At Pactola Dam, South Dakota, both a geotextile and geomembrane were used in raising the dam and to provide an impermeable barrier for the highest flood waters. At Cottonwood Dam No. 5, Colorado, a geomembrane was field tested as a lining for an emergency spillway. Geosynthetic materials were chosen for these projects because they were judged to be technically reliable substitutes for conventional materials or they provided supplemental benefits not otherwise achievable, and they were significantly more cost efficient. The Bureau of Reclamation has met with variable success in achieving the design purpose of each project that has been constructed involving geosynthetics. The following describes these experiences.

Mt. Elbert Forebay Reservoir.–In 1980 at Mt. Elbert Forebay Reservoir, Reclamation became concerned that leakage through a previously constructed earth lining was threatening the stability of an ancient landslide uphill from a powerplant. About 117 hectares (289 acres)[3] of 1.14-mm- (1/16-inch-) thick CPER (reinforced chlorinated polyethylene) geomembrane were installed for seepage control in the reservoir.

To monitor the performance of the lining, a test section was placed in the reservoir so that coupon samples could be retrieved periodically for evaluation. Test results to date indicate that the lining has experienced some water absorption resulting in a decrease in seam strength. Water absorption of the CPER sheet material is primarily responsible for the observed mechanical property changes in the CPER and CPER seams. Most of the changes occurred within the first 3 years of service and are not considered detrimental to the overall lining integrity.

San Justo Reservoir.–The steep perimeter slopes of San Justo Reservoir were partially lined with a HDPE-A (high density polyethylene with alloy) geomembrane to selectively control seepage through pervious beds of sand and prevent landslides outside of the reservoir. The liner was supplemented by a minimum 0.5-m (1-1/2-foot) impervious earthfill cover, a 0.15-m (1/2-foot) layer of bedding material, and a 0.3-m (1-foot) layer of rock fragments.

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[1] Reprinted with permission from the Managing Editor, Geotechnical News, June 1989 issue.
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[3] English measurements in parentheses are approximate.

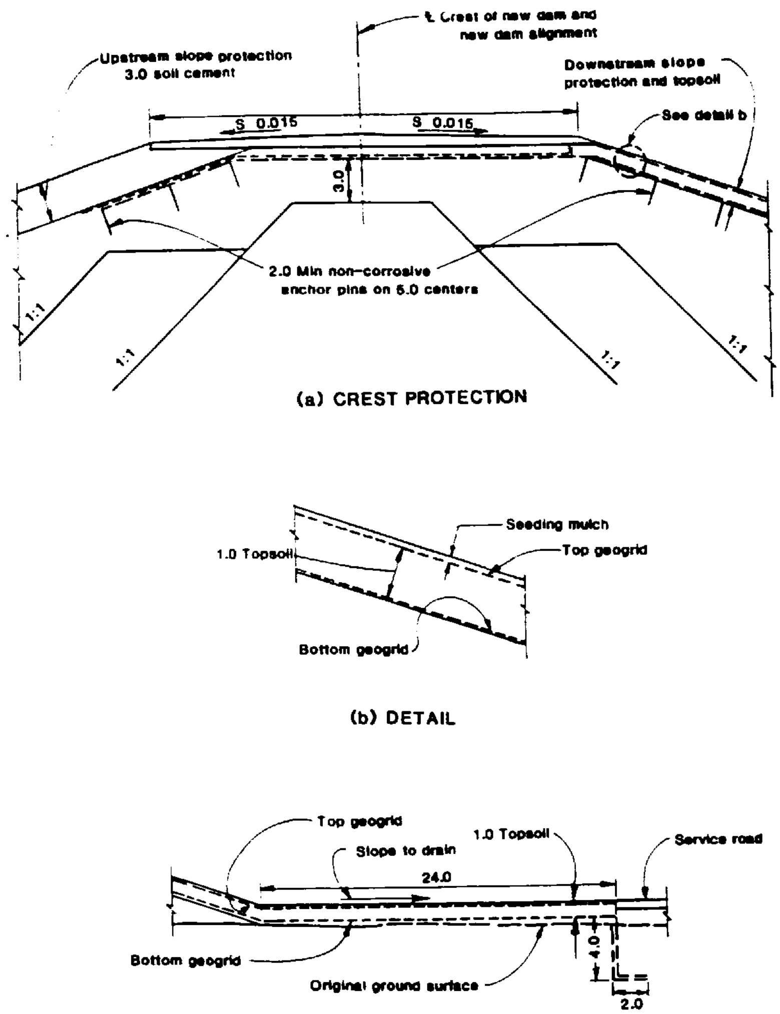
During February 1986, unusually heavy rainfall occurred in the reservoir area. During and immediately following the rainfall events, slippage of portions of the earthfill covering the geomembranes occurred. Of the approximate 19 hectares (47 acres) of membrane liner installed at the site, the slope failures affected approximately 4.2 hectares (10 acres) of which approximately 1.3 hectares (3 acres) of geomembrane was exposed.

The exposed portions of the geomembrane were repaired in 1986, and the reservoir was filled. Some seepage and instability external to the reservoir has been experienced but it is not currently attributed to any inadequacy of the membrane lining.

Jackson Lake Dam.–The liquefiable foundation of Jackson Lake Dam was densified by dynamic compaction. Prefabricated ("wick") drains were specified for use where areas of fine-grained materials were encountered under significantly high embankment sections. The drains were intended to facilitate drainage from materials underlying the fine-grained zone, as well as to enhance drainage from within the fine-grained zone. The drains were installed through the fine-grained zone but not through the underlying coarser material. A comparison using SPT (Standard Penetration Test) corrected blowcounts between two compacted areas of similar stratigraphy, one with and one without the prefabricated drains, provided a quantitative evaluation on the effect of the drains. The presence of the drains apparently enhanced the densification effort, as the blowcount improvement was doubled in the fine-grained zone from 4 blows (blowcount increase from 15 to 19) without the drains to 8 blows (blowcount increase from 14 to 22) with the drains. In the coarser material beneath the fine-grained zone, no blowcount improvement was observed in areas without the drains (blowcount equals 22, no change) whereas an improvement of 6 blows (blowcount increase from 22 to 28) was observed in areas with the drains. Thus, the presence of the drains apparently enhanced the dynamic compaction of the dam's foundation.

On the basis of durability, environmental impact, overall cost, and constructibility, a reinforced grass slope was selected to provide overtopping protection to Jackson Lake Dam (see figure 1). A 0.3-m (1-foot) layer of topsoil was placed on the downstream face of the dam to support a grass cover, and for overtopping protection, an erosion control geogrid was placed on the topsoil and pinned down to be in firm contact with the dam face (figure 1a.). Between adjacent layers of geogrids there was an overlap of at least 50 mm (2 inches). In addition, a separate geogrid was placed between the embankment material and the topsoil as a second line of defense should the reinforced grass layer fail and be washed away (figure 1b.). This geogrid was secured to the embankment with noncorrosive pins, and a minimum overlap of 150 mm (6 inches) was specified. The geogrid was laid across the crest and down the upstream face under the upstream slope protection. At the downstream toe of the dam, both the underlying geogrid and surface erosion control material was extended 8 m (26 feet) or about four times the tailwater depth beyond the embankment toe (figure lc.).

Geogrids have also been incorporated into the Jackson Lake modification design to prevent potential cracking of the dam during an earthquake due to dynamic deformations of the relatively thick layers of clays and silts underlying the treated foundation. Geogrids were placed at the base to reinforce the dam and minimize differential settlement and cracking. The 1-m- (3-foot-) wide geogrid strips were placed perpendicular to the dam axis in a continuous strip across the base of the dam with a 0.1-m (4-inch) overlap.



(a) CREST PROTECTION

(b) DETAIL

(c) DOWNSTREAM TOE SLOPE PROTECTION

FIGURE 1. JACKSON LAKE DAM
(DIMENSIONS IN UNITS OF FEET)

Davis Creek Dam.–Davis Creek Dam, a new Bureau of Reclamation dam under construction in central Nebraska, is believed to be one of the first examples in the United States of a major embankment dam reinforced with geogrids. A rolled earthfill embankment with a height above streambed of 33.5 m (110 feet) and an embankment volume of approximately 2,300,000 m^3 (3,008,400 yd^3) (primarily silt and clay material), geogrids will be utilized to steepen the uppermost 7.6 m (25 feet) of downstream embankment slope from 2.5(H):1(V) to 1:1. It was determined that such steepened slopes at this dam would lead to a cost savings of approximately $1,000,000, due to reduced embankment volumes and related factors.

Six layers of uniaxial geogrids with an embedment length of 4.9 m (16.4 feet), will provide the primary reinforcement of the 1:1 slope. Biaxial geogrids with a 2.0-m (6.5-foot) embedment length will be placed at 0.3-m (1-inch) vertical spacings to stabilize the outer edges of the slope and permit hauling, placement, and compaction equipment to operate all the way to the edge of the slope (figure 2). The finished slope will be covered with topsoil, seeded, and protected by an erosion control matting in order to cultivate a grass cover and root system that will provide long-term slope protection.

Due to the rather unprecedented nature of the design for the Bureau of Reclamation, a very thorough instrumentation system consisting of strain gauges, strain meters, and extension meters will be installed to monitor and evaluate the performance of the geogrid reinforcement. It is believed that these instruments will provide valuable data which will lead to a better understanding of the performance of this type of reinforcement over time.

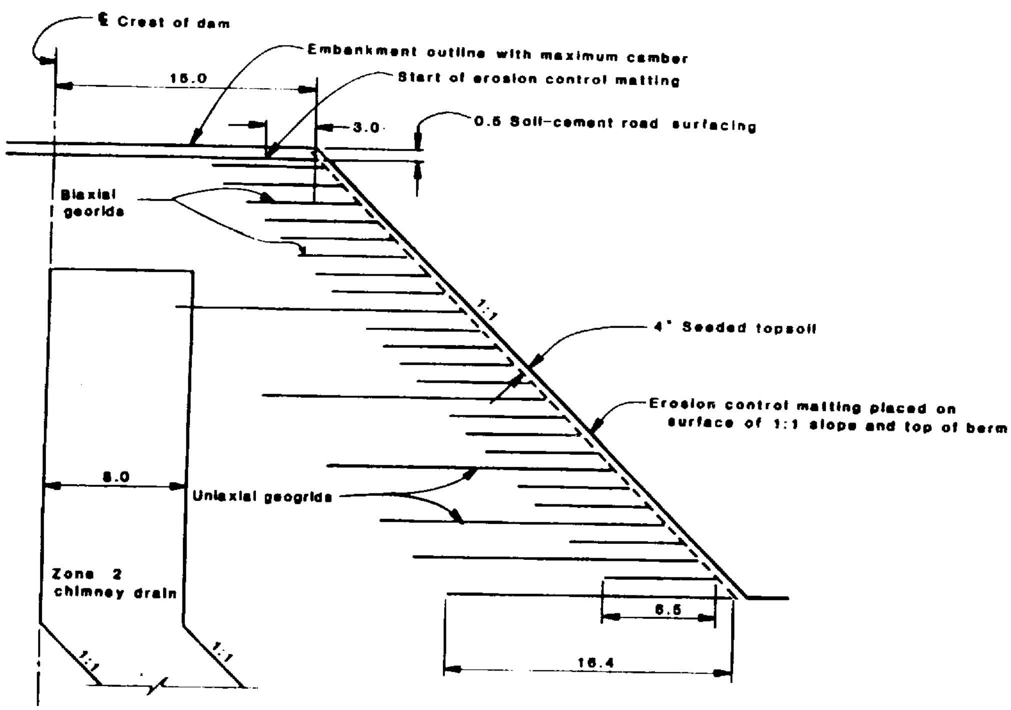
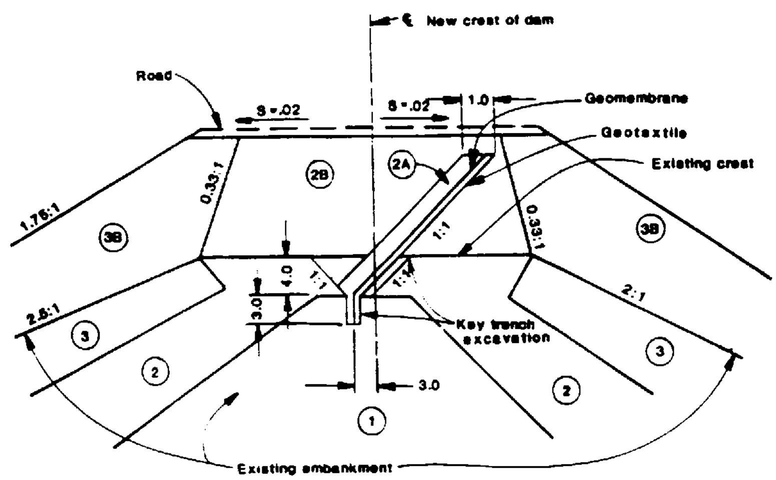


FIGURE 2. DAVIS CREEK DAM GEOGRID REINFORCEMENT
(DIMENSIONS IN UNITS OF FEET)

Merritt Dam.-Remedial designs for controlling the downstream seepage at Merritt Dam include a gravel toe drain held in place by a berm. A geotextile filter is to be used to separate the gravel of the drain from the sand of the abutment and the miscellaneous fill of the supporting berm. Considerations for the design of the geotextile include adequate permeability to pass water from the abutment to the drain, and filtering capability to prevent the abutment sand and miscellaneous berm material from contaminating the gravel drain. The geotextile is to wrap around the entire gravel drain. An outflow pipe will be placed at the bottom of the gravel to remove water from the drain. Should maintenance ever be required, access to the geotextile, gravel zone, or outflow pipe can be achieved by excavating through the berm without threatening the integrity of the drain or the dam.

Where the slope of the gravel drain exceeds 2(H):1(V), such as on the right cut slope of the stilling basin, the soil-fabric frictional strength is not adequate to prevent shallow slides along the geotextile. In this case, a granular filter will be used. Because the nearest source of suitable filter material is 130 km (80 miles) from the damsite, the geotextile was used whenever possible on all the shallower slopes at a considerable savings in cost.

Pactola Dam.-Pactola Dam needed to be raised to accommodate a revised probable maximum flood. To facilitate handling of the high traffic volume, reduce the number of construction seasons, and avoid unsightly borrow area construction, a geomembrane was selected as the impervious barrier for the raised embankment (figure 3). Use of a membrane significantly reduced the volume of the graded sand filter zone, for which a 96-km (60-mile) haul was anticipated. The geomembrane also replaced the clay core eliminating the need for borrow area development in the scenic Black Hills National Forest. To ensure embankment stability, a 0.3-m- (1-inch-) wide graded sand zone was placed upstream of the membrane to serve as a filter for the rockfines upstream of the sand. The sand and geotextile backing acted as cushions between the rockfines and membrane as the fill was compacted.



Zone (1) - Selected silt, clay, sand and gravel compacted in 6" lifts.

Zone (2) - Rockfines compacted in 12" lifts, max. size = 6"

Zone (3) - Rockfill placed in 3-foot lifts, max. size = 1 yd^3

Zone (2A) - Processed sand filter, 12" lifts.

Zone (2B) - Rockfines from required excavation, compacted by vibratory roller to 12" layers, max. size = 8"

Zone (3B) - Selected rockfill from required excavation, compacted by vibratory roller in approximate 3' layers.

FIGURE 3. PACTOLA DAM CREST RAISE
(DIMENSIONS IN UNITS OF FEET)

Design of the anchorage for the top of the membrane consisted of a 0.3- by 0.6-m- (1- by 2-foot-) deep anchor trench excavated behind the face of the supporting fill. The membrane was laid in the excavation and the trench backfilled. Placement and compaction operations on the slope created excessive tension in the membrane, which in one instance, resulted in splitting of the membrane. This was remedied thereafter by only loosely anchoring the top of the membrane. Anchoring the toe required measures to ensure that an impermeable seal was achieved between both the rock abutment and the original zone 1 core of the dam and dikes.

Anticipated difficulties arose with regard to field seam construction. The original design did not permit horizontal field seams and specified minimal vertical seaming. For ease of construction, a horizontal seam was later permitted. Elimination of buckling, resulting from the severely curved alignment, required special trimming and seaming. Following completion, several test pits were excavated to determine if damage may have occurred during construction. The membrane observed in the test pits was in good condition and showed no signs of distress.

Cottonwood Dam No. 5.–Recently, research has been conducted looking at the possibility of using geomembranes for emergency spillways. This is an attractive, low-cost alternative to meet the need to increase spillway capacity for existing structures. The research included a field study where Cottonwood Dam No. 5, Colorado, was retrofitted with a geomembrane emergency spillway covered with earth material. It was tested, and during overtopping, most of the earth material washed away, as planned, and the membrane carried the flow without any damage to the dam.

Conclusion

Aside from the projects mentioned above, the Bureau of Reclamation has been using geomembranes for a long time in canals and other types of facilities. Reclamation does not hesitate to consider the use of geosynthetic materials within its structures. Although not comfortable with specifying a geosynthetic material as a sole substitute for a major internal component of a large dam without assurance of adequate performance, the Bureau of Reclamation is interested in using geosynthetics in combination with conventional designs and multiple lines of defense to increase safety or realize cost savings. Geosynthetics are an attractive alternative where they can be tested and maintained. As the Bureau of Reclamation gains experience and more field performance data are available, it expects to specify the use of geosynthetic materials more often.

References

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WHEN IS YOUR DAM SAFE ENOUGH?[1]

by Gary D. Bachman and John J. Buchovecky [2]

FERC's rules sometimes put its project licensees between a rock and a hard place. The authors explore approaches that have worked to lessen project owners' burdens. They also suggest policy changes to improve the regulatory process.

A significant number of hydroelectric dam owners and developers are currently walking an administrative and regulatory tightrope — balancing the competing demands of government-enforced safety standards and economic realities. FERC licensees face particular difficulties in this respect, since this agency does not routinely consider the impact of its safety requirements on project feasibility.

FERC's lack of consideration for the whole picture, in part, has spawned the current controversy over the appropriate safety standard for use in new dam design and existing dam evaluation and rehabilitation [1,2][3]. FERC practice and policy almost categorically require the use of the Probable Maximum Flood (PMF) as the spillway design flood for all projects whose failure might result in a potential — however slight — for a loss of life or significant property damage. Licensees whose projects fail to meet the PMF may be required to make large expenditures to retrofit projects to meet such a standard, without any analysis of whether the cost of retrofitting is justified by any benefits. Compounding licensees' problems further is the fact that they typically do not have a favorable legal or political position from which to challenge FERC's authority.

However, a number of licensees have successfully demonstrated to FERC — through structural, meteorological, and other analyses — that its spillway design flood standards can sometimes be decreased or that alternative mitigative measures can be put in place while still maintaining necessary levels of safety. Such alternative measures can cost substantially less than massive spillway expansions or other project retrofits. Proving such a case to FERC, however, is an expensive and time-consuming undertaking, and one which is neither explicitly nor officially encouraged by FERC.

Why Is the Debate Happening Now?

To understand the difficulty involved with persuading FERC to deviate from its perceived mandate in the dam safety area, one must be aware of the genesis of FERC's present dam safety regulations. In 1976, the Kelly Barnes Lake Dam in Toccoa Falls, Georgia, failed during a severe storm and resulted in 38 fatalities. That catastrophe, combined with several other dam failures during the 1970's (including the massive Teton Dam

[1] Reprinted with permission from Hydro Review, October 1989 issue, HCI Publications, Kansas City, Missouri.
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[3] Numbers in brackets refer to the listing in "Notes" at end of article.

failure), provided the impetus for the creation of an interagency task force. This task force consisted of representatives of all Federal agencies with jurisdiction over dams; its purpose was to review procedures and criteria used in the construction and operation of those dams under their jurisdiction. The recommendations formulated by the task force were subsequently incorporated in the early 1980's — in varying degrees — in the safety guidelines of Federal agencies, including the Bureau of Reclamation, the Corps of Engineers, and the FERC.

Pursuant to its statutory authority under the Federal Power Act, FERC adopted comprehensive regulations requiring dam owners, among other things, to report safety-related incidents or conditions affecting the safety of the project; to develop and file FERC emergency actions plans to protect the public in the event of a project emergency; and, for dams that meet certain criteria, to have the dams periodically inspected and evaluated by an independent consultant.

Protecting Against Extreme Floods

One aspect of the required independent consultant inspection involves an evaluation of the ability of the dam's spillway and impoundment structure to withstand extreme flood flows without failure. For projects whose failure would present a potential hazard to human life or cause significant property damage, FERC's policy has been that the dam's spillway or impoundment structure must be designed to withstand or pass the PMF or the "incremental flood." The incremental flood is the flood above which failure of the dam will not incrementally increase the size of the downstream flood. In facilities with little storage, the incremental flood is often smaller than the PMF. FERC's approach to the PMF standard assumes a worst-case combination of weather and other conditions, resulting in the most severe flood which could be hypothesized. No attempt is made to calculate, or incorporate into the analysis, the probability that all of the events required to produce a flood of such extreme magnitude will in fact occur at the same time. In some cases, the PMF represents such a rare event that the probability of its occurrence, while calculable with the aid of computer generated models, is nonetheless infinitesimal and accounts for far less a risk than society accepts daily from bridges, highways, tunnels, and buildings. Nonetheless, some dam owners under FERC jurisdiction are currently faced with multimillion dollar retrofits of their dams in order to prepare for a PMF event.

FERC's approach to the PMF standard also raises other related problems: (1) Because FERC's application of the PMF does not assess the probability that the PMF will occur, it does not represent a true measure of risk; (2) Because there is no probability assessment tied to the PMF, application of the PMF standard produces drastically inconsistent and needlessly burdensome requirements, depending on the location and characteristics of particular project sites; and (3) FERC's insistence on the use of PMF as a design standard tends to exclude consideration of equally effective but less costly alternatives to dam redesign.

Costs Go Up When Theory Meets Practice

The foregoing criticisms do not exist only in theory. On the contrary, the effect of FERC's application of the PMF standard upon licensees can be real and direct. For example, one northwestern utility determined that the once-in-200,000-year storm would result in a maximum flood flow at the project of 77,000 ft^3/s. This was within the spillway's capacity of 80,000 ft^3/s. FERC, however, rejected the licensee's statistical methods and, instead, insisted that the proper design flood flow was 128,000 ft^3/s. Under the applicable regulations, if it cannot be established that the dam would survive overtopping at flood flows above its spillway capacity, the project may have to undergo expensive modifications to prepare for a flood that has a chance of occurring less than once in 200,000 years. This example is not atypical. In another case, the frequency of recurrence of the PMF for a project in Wisconsin was calculated at 35 million years, and for a project in Utah the frequency of recurrence was 10^{27} years.

For some projects, the cost of retrofitting spillways to meet the PMF standard has been estimated to be millions of dollars. Moreover, FERC has not completed its PMF review of all of the more than 2,200 dams subject to its jurisdiction, so the eventual total cost and impact of retrofitting the nation's hydroelectric facilities to prepare for an event that may not occur for millions of years is not known.

Political Realities

There is an explanation for FERC's reluctance to consider the costs associated with its safety program. First, unlike other Federal agencies, FERC does not own or operate any dams, and is therefore not directly faced with justifying the cost of project retrofitting. Without the need to justify the cost of safety expenditures in light of competing budget demands, FERC operates its program purely on a command and control basis. Second, FERC — as a regulatory agency subject to the vicissitudes of Congress — can only envision disadvantages in compromising with licensees on issues of dam safety standards. In the absence of a Congressional policy directive establishing society's expectations in the dam safety area, FERC has adopted the most conservative position on dam safety and established a worst-case, state-of-the-art standard with little or no consideration of costs or probabilities.

Other Agencies Are More Flexible

FERC, the Bureau of Reclamation, and the Corps all incorporate PMF into their design flood selection process to varying degrees. However, Federal agencies which more directly bear the costs of construction, operation, and maintenance of dams (such as the Bureau of Reclamation and the Corps) have established guidelines for dam safety which include procedures for conducting probability-based or site-specific determinations of appropriate dam safety requirements, instead of using PMF as an exclusive determinant of overall project safety. Procedures developed by the Bureau of Reclamation are illustrative. The Bureau's "Guidelines to Decision Analysis,"[3] provides for a risk-based, site-specific determination of appropriate dam safety requirements. The procedure described by the Bureau includes a requirement to estimate the probability of failure of a particular dam in response to events (including extreme floods or earthquakes) which have the potential for causing dam failure. The Guidelines also discuss warning systems as an appropriate safety measure.

Another fundamental difference between FERC's approach and that of other agencies lies with the method used to determine the possibility of loss of life. Both the Bureau of Reclamation and the Corps make a distinction between the Population at Risk (PAR) and the Probable Loss of Life. The PAR includes "all individuals who, if they took no action to evacuate, would be exposed to flooding of any depth."[3,4] Possible loss of life is then determined by estimating the amount of warning time the PAR would receive, and using historical relationships between warning time and loss of life. For example, PAR's sufficiently downstream may be susceptible to severe flooding upon a dam's failure, yet there may be no projected loss of life because the floodwave would not arrive until after the PAR had evacuated the area. Since (in this example) there is no projected loss of life, the adequacy of the dam's spillway may be evaluated based on a flood of lesser magnitude than PMF. In comparison, FERC's method of projecting loss of life does not draw the distinction between Populations at Risk, and probable lives lost. This limits agency consideration of flood warning and dam monitoring devices which, if implemented, might eliminate the risk of loss of life and thereby eliminate the need to redesign or retrofit the dam.

The Opportunity for Creativity Exists

FERC's dam safety policy places the burden of demonstrating project safety squarely on the licensee, with little or no room for interpretation. This does not mean, however, that a licensee cannot make an argument which would persuade FERC to be more flexible in applying its regulations. In fact, this is exactly what some licensees have accomplished. In spite of the apparent rigid regulatory structure, FERC has created a de facto regulatory scheme whereby it has, in some cases, negotiated compromises with licensees who have gone to the expense of demonstrating that their particular circumstances warrant a special application of FERC's regulations. FERC has recently relied on the existence of its informal procedure to deflect Congressional criticism that it is rigid and inflexible in its application of dam safety standards. FERC claimed to have implemented a flexible approach to its dam safety policy. Though it may be willing to consider alternative safety measures when forcefully brought to its attention by a licensee, we have not found that this policy is public knowledge.

Licensees who have been successful with this strategy have typically gone beyond the limited authority of FERC's Regional Offices and petitioned their case to the FERC Washington Office. As a general rule, because of the expense involved with assembling and presenting such a case, the FERC-required safety modifications must be substantial. One approach to challenging the application of a particular PMF standard to a project involves questioning the underlying assumptions upon which the PMF is based; namely, the validity of the National Weather Service statistics on probable maximum precipitation (PMP). At least one northeastern licensee has been successful with this approach.

A variation on this tactic is to challenge the antecedent conditions to the PMF. Since the PMF assumes a worst-case scenario, some licensees have argued that assumptions about antecedent conditions such as snowpack, ground saturation, and previous rainfall are not applicable in their case. If this argument is successful, the PMF may sometimes be significantly revised.

Licensees have also challenged the PMF directly as being too improbable, and therefore establishing too high a standard of safety. Although FERC has not accepted this position outright, in some cases it has agreed to suspend final judgment pending industry group research on the scientific cogency and rationale of the PMF methodology as applied to a particular region of the country.

Another available alternative to licensees involves conducting a probabilistic risk assessment of the project. A risk assessment study may be used to demonstrate the feasibility and effectiveness of alternative structural modifications such as fuse plugs, embankment reinforcement and refacing, or warning systems which would allow evacuation before threat of failure manifested itself. FERC generally will permit and review such a study, even though its official policy does not encourage this approach.

Some Suggestions for the FERC

Although FERC will often consider approaches such as those suggested in the foregoing, its current case-by-case approach does not go far enough to increase industry awareness of these solutions, nor does it foster the kind of information sharing that results in continued development and perfection of technical standards and methodologies. For these reasons, FERC should modify its regulations or guidelines accordingly so that all licensees are aware of valid alternatives and to promote a greater understanding of the technical and policy issues involved in their decisions.

One recent development in a related context offers some hope that FERC may be amenable to a technical revision of its dam safety criteria. FERC recently adopted a more realistic, cost effective, and reasonable approach to certain of its design standards. Late last year, FERC reduced the safety factory criteria of gravity dams for the PMF loading condition from 2.0 to 1.5 so long as that condition is monitored.[5] Like spillway design flood standards, the controversy over loading safety factors for gravity dams derives from the statistical reliability of the bases and methodology used to calculate the safety factor. Industry research indicates that the statistical reliability of the safety factory calculations is greater than originally believed.

Similarly, as advances are made in the safety evaluation of earthen dams, FERC should reevaluate its almost exclusive use of PMF as the spillway design flood standard. At a minimum, FERC should adopt regulations which make official its de facto position on acceptable alternative safety procedures. The informality and lack of uniformity of the current case-by-case approach may benefit a few licensees, but in the long run it only hinders research in this important area of public policy and diminishes FERC's reputation as a fair and open regulator. Although FERC's important dam safety inspection program should continue — and FERC must have continued authority to order emergency corrective measures for any immediate threat to human life, health, property, or the environment — design upgrade decisions based only on the PMF should be deferred pending development of a reasonable and cost-effective safety policy that is consistent with our society's safety expectations.

Notes:

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2. Faulkner, E., "Estimating Extreme Floods for Spillway Design," Hydro Review, April 1989.

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5. FERC "Engineering Guidelines for the Evaluation of Hydropower Projects," Chapter III, December 2, 1988.

SONAR EVALUATED FOR AQUATIC WEED CONTROL[1]

More Research Required Before Licensing

Over the past year, researchers have been evaluating the effects of a new aquatic weed control herbicide called Sonar [2], which was applied to a canal near Vauxhall. The project is a cooperative effort involving Elanco Products Company, Alberta Environment, Agriculture Canada, and the Bow River Irrigation District.

Sonar, which contains fluridone as the active ingredient, was recently approved in the United States for the control of aquatic weeds in lakes, ponds, reservoirs, ditches, and irrigation canals. Sonar is not, at this time, approved for use in Canada, as research is required to determine how it can be used under Canadian conditions.

Affected plants turn white. Only 10 percent of the cattails and reed canarygrass showed any sign of injury.

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[1] Reprinted with permission from the Editor, Water Hauler's Bulletin, Fall/89 issue, Alberta Agriculture Centre, Lethbridge, Alberta T1J 4C7.

[2] Sonar is the registered trademark for Elanco Product's fluridone.

Fluridone inhibits the plant's ability to make food, says Bill McGregor, a research specialist with Elanco Products Company. Without that ability, the plant dies. Specifically, fluridone inhibits carotenoid synthesis. Carotenoids (yellow pigments) are an important part of the plant's photosynthetic (food making) system. These yellow pigments protect the plant's green pigments (chlorophyll) from photodegradation (decomposition by sunlight). When carotenoid synthesis is inhibited, the chlorophyll is exposed to photodegradation and is gradually destroyed.

As a plant's chlorophyll decreases, so does its capacity to produce carbohydrates through photosynthesis. The visual system of fluridone action is bleaching or the development of chlorosis on the terminal bud or growing points of the plant, adds McGregor.

In the United States, Sonar is applied to impounded water in irrigation canals when aquatic weeds are actively growing. The treated water must remain impounded for at least 7 days to allow for herbicide uptake by the weeds.

With Alberta's short season and continual water demand, it is not feasible to impound water within canals for 7 days during the period of active aquatic weed growth. As a result, investigators decided to try applying Sonar to a canal bottom after shutdown in the fall. The idea is that the herbicide — which remains active in the soil over several months — will move into the soil profile over the winter and spring and be available for rooted plant uptake during the following growing season.

Sonar 5P (5 percent fluridone pellet) was applied to the bottom of Lateral H1 of the Bow River Irrigation District on October 25, 1988. A total of 3.4 kg (approximately 7-1/2 pounds) of the pelleted formulation [0.17 kg (approximately 6 ounces) of fluridone] was applied to three plots each measuring 50 m × 5 m (approximately 164 feet × 16 feet) [this corresponds to an application rate of 45 kg Sonar 5P per ha (approximately 40 pounds per acre) or 2.25 kg fluridone per ha (approximately 2 pounds per acre)].

The pellets were applied to the canal bottom by use of a hand-held Whirlybird seeder/ fertilizer spreader. For each plot, the applicator walked down the center of the canal spreading 50 percent of the required amount of material [total of 1.125 kg (approximately 2-1/2 pounds) Sonar 5P per plot], then repeated the sequence applying the remaining 50 percent. The seeder delivered 0.9 kg (approximately 2 pounds) of product per minute over a swath of about 5 m (approximately 16 feet) which was adequate to cover the width of the canal bottom. A walking speed of 4.5 km/h (2.8 mi/h) was maintained while applying the material.

The weed growth in the canal consists of several types of submersed vegetation (water milfoil, pondweeds), as well as cattails and reed canarygrass. Water was reintroduced to the canal in May of 1989 and the effects of the herbicide treatment were evaluated after weed growth began in earnest in June.

In early July, effects of the Sonar treatment began to show up on the cattails.

Typical symptoms of Sonar injury include a bleaching of the green pigment so that treated plants turn white. This characteristic was apparent on about 10 percent of the cattail and reed canarygrass in the treated sections of the canal.

Unfortunately, the low level of effect observed in July did not increase through the rest of the summer nor did effects become evident on the submersed weed growth under water. It may be possible that inadequate herbicide remained in the soil over the winter and spring to cause significant control of the weed population.

Although the test was only partially successful, much was learned about the use of Sonar in irrigation canals. Information from this past year's efforts will be used to determine techniques that will improve the effectiveness of Sonar for aquatic weed control in irrigation canals. Even if Sonar does prove to be effective, it is a long road to obtaining government approval for the chemical's use here in Canada, says McGregor. It could be another 5 to 10 years before we have completed our research and obtained approvals, and only then could we commercially market Sonar, he adds.

For further information, please contact Robert Burland, Alberta Environment, Provincial Building, 200 - 5 Avenue South, Lethbridge, Alberta T1J 4C7; telephone (403) 382-4015.

FREEZING TEMPERATURES MEAN BROKEN AIR VALVES[1]

Protection Is the Key

Along with the autumn colors comes freezing temperatures and the ensuing problem of broken air valves on irrigation pipeline turnouts. The St. Mary River Irrigation District (SMRID) decided something had to be done to prevent this costly yearly expenditure.

In the fall of 1988, the SMRID began experimenting with various methods to find a solution. Ron Renwick, P. Eng., District Engineer, felt that some type of insulation may be the answer. "We tried wrapping the valves with insulation but this didn't work very well. Next we tried insulated jackets similar to those used in the oil fields but again they weren't the answer," says Renwick.

The engineering staff designed, built, and began experimenting with a steel canister insulated with 50-mm-thick (2-inch-) styrofoam, says Myles Kasun, irrigation technologist with the district. We chose two adjacent air valves protecting one with the insulated steel canister, the other valve was left exposed to the elements, says Kasun. Minimum temperature recordings were taken at both locations and were as follows:

	Minimum temperature °C	
Date	Insulated valve	Uninsulated valve
10-25-88	6	2
10-26-88	5	1
10-27-88	-3	-7
10-28-88	-4	-9
10-29-88	-7	-11
10-30-88	-3	-6
10-31-88	5	3
11-01-88	3	0
11-08-88	-3	-3
11-09-88	-4	-8
11-10-88	-5	-7
11-11-88	-2	-2
11-12-88	-1	-2
11-13-88	-3	-6
11-14-88	-4	-7
11-15-88	-11	-15

The insulated canister provided an average 3 °C protection. It is believed the insulation also reduces the time the valve is exposed to the low temperature thus allowing less time for freezing to occur. The uninsulated valve was damaged at a low temperature of −7 °C. The insulated valve was undamaged at −15 °C, says Kasun.

[1] Reprinted with permission from the Editor, Water Hauler's Bulletin, Fall/89 issue, Alberta Agriculture Centre, Lethbridge, Alberta T1J 4C7.

Technologist, Paul Ewashen, demonstrates how easily an insulated canister can be removed for valve inspection.

The insulated steel canister does work as evidenced again, this past fall, where the SMRID had approximately 12 valves freeze that weren't provided the canister protection. Not one protected valve froze however. The canisters are left on year round providing protection from early spring frosts as well, adds Renwick.

One small design change, says Renwick, was the addition of a small hole in the bottom of the canister to equalize air pressure allowing for proper valve operation.

At a cost of approximately $300 apiece, air valves are not cheap. The complete insulated steel canisters on the other hand cost approximately $120. Most welding shops can easily fabricate them and the insulation is available from Duncan Aluminun in Lethbridge, says Kasun.

For more information, please contact Myles Kasun, Irrigation Technologist, St. Mary River Irrigation District, PO Box 278, Lethbridge, Alberta T1J 3Y7; telephone (403) 328-4401.

UNDERSTANDING TRUCK TIRES[1]

Municipal fleets include automobiles, heavy equipment, and perhaps the real day-to-day workhorses of public works departments, trucks. Virtually any municipal operation will include a truck of some kind — small trucks for transportation and administration, dump trucks to deliver materials and tow trailers, and refuse packers for the collection of solid waste, to name just a few. But no truck, no matter how sophisticated, can deliver full service without the right tires. Understanding tire types, construction, materials, selection, maintenance, and retreading is critical for a municipal fleet manager.

There are two basic types of truck tires today: tube-type and tubeless. Tube-type tires rely on a tube — a separate inner chamber — to contain compressed air. Tubeless tires have an integral inner liner that holds the air; there is no separate chamber.

The tubeless tire has numerous advantages over the tube-type tire. Most of these advantages stem from a tubeless tire needing only two components, while a tube-type tire needs as many as six. Also, tubeless tires can run with a bigger rim and lower profile than tube-type tires.

Tubeless tires are easier to mount and have less chance of failure due to mismatched components and reduced inventory and labor costs. The better balanced assembly of tubeless tires gives a smoother ride and increased mileage. Also, weight saving of several pounds per tire reduced strain on the suspension, allows several hundred pounds more payload, and creates less rolling resistance to save fuel. Because tube-against-tire friction is eliminated and a bigger wheel with increased air flow is used, the tubeless tire runs cooler to improve mileage, economy, and casing life.

Finally, tubeless tires experience fewer flats and downtime. The tubeless liner clings to penetrating objects to slow air loss and there is less chance of losing air around the valve on a tubeless tire.

Today's truck tires are available in two basic constructions: bias and radial. In bias construction, bias body ply cords run diagonally from bead to bead - each layer criss-crossing the other. These tires may also have narrow belt-like plies, called breakers, which lie under the tread. These breakers have cords that lie diagonally to the bead. In radial construction, radial body ply cords run straight across the tire from bead to bead. This construction permits easier flexing in the sidewall area. In addition, belt plies encircle the tire under the tread. These belts (there may be three or four) constrict the radial plies and add strength and rigidity. Generally, radial construction impacts the following:

- *Tread life.*-Sidewall flexing lets the tire hug the road while the belts keep it flat, reducing wear.

- *Fuel efficiency.*-Radial plies put up less rolling resistance.

- *Load.*-Radial construction can handle more load.

[1] Reprinted with permission from the Editor, Public Works, November 1989 issue.

- *Downtime.*-Radial belts protect against punctures for fewer flats.

- *Performance and comfort.*-Easy-flexing sidewalls and strong belts give road-hugging traction and a better ride.

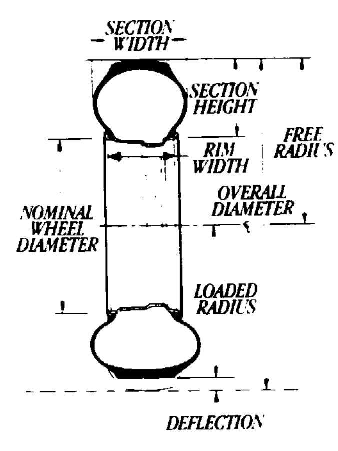
- *Retreading.*-Radial plies generate less heat than bias plies, and heat can shorten tire life, so radial casings have more retread potential.

Tire Selection

When selecting a tire, besides considering tire type and construction, you must also remember tire size, casing strength, life cycle cost, and tread design.

Truck tire sizes are indicated by a number molded into the sidewall. It is usually a two-part number giving the section width of the tire in inches followed by the rim or wheel diameter in inches. For example, 11-22.5 would mean that the tire section is 11 inches wide and the diameter of the rim or wheel is 22.5 inches. Equivalent tube-type and tubeless tires will have different size designations. A 9.00-20 tube-type tire is equal to a 10-22.5 tubeless. A 10.00-22 tube-type is equal to an 11-24.5 tubeless. This is because a tubeless tire has a lower profile and a larger rim than a tube-type tire.

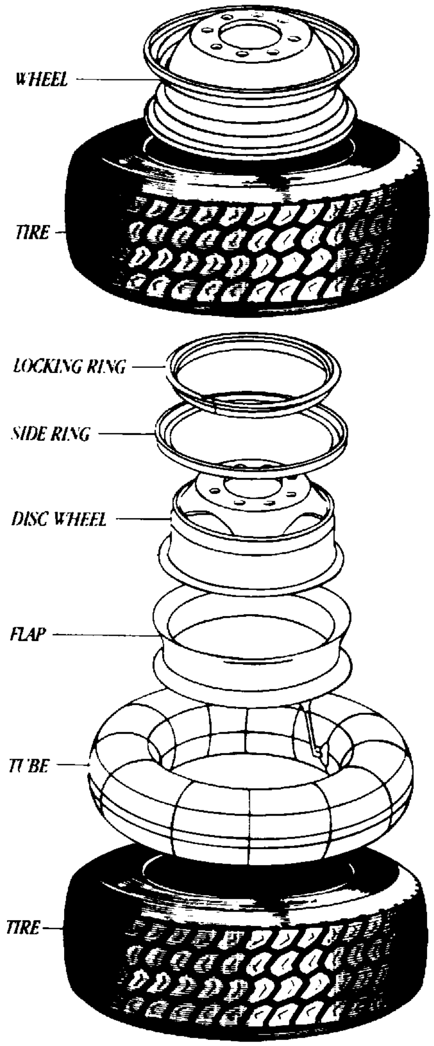
A rule of thumb for converting a tube-type size designation to a tubeless is to take the section width and remove all figures after the decimal point. Then add 1 to the section width and 2.5 to the rim diameter.



■ OTHER tire dimensions.

Low profile markings use a three-part designation. The first number is the cross section in millimeters, the second is the aspect ratio (tire height to width), and the third number is the rim size in inches. So in the low profile size designation 275/80R24.5, 275 is the section width in millimeters, 80 is the aspect ratio, R stands for radial, and 24.5 is the rim or wheel diameter in inches.

Tire casing strength is expressed as ply rating or load range. A ply rating is given as a number while load range is a letter. These designations indicate load-carrying capacity - the higher the ply rating, the more load the tire can carry. Actually, ply rating or load range relates to the tire's ability to hold air pressure and volume. It is the air that carries the load.



Tube-type tires have many more components than the tubeless tire.

The load-carrying ability of the tire varies with inflation pressure. The Rubber Manufacturers Association provides detailed load and inflation tables to help determine load limits of a tire at various inflation pressures.

The load per tire must not exceed the capacity of the tire, rim, wheel, or other vehicle components. The vehicle's capacity for load is given in its Gross Axle Weight Rating or Gross Vehicle Weight Rating. The tire load capacity is stamped on the sidewall of all truck tires manufactured after March 1975.

One way to prevent tire overload is to weigh each axle or tire when the vehicle is fully loaded. If an axle is weighed, the number of tires on the axle are divided into the weight to determine load per tire. If the maximum load-carrying capacity of the tire is below the scale weight, tires with greater capacity should be used.

When selecting a different size tire than original equipment, be sure the rims are suited for it and the vehicle geometry will accept it. Your local tire dealer can help you with other selection factors in selecting the right tire, such as speed, distance, traction, mileage, or road conditions.

Life cycle of the tire casing is important to any tire selection. To begin with, the casing is 80 percent of the tire cost. Because tires are a major expense in any truck fleet, it is crucial to get the maximum life out of them before they are scrapped. This means buying a tire with good retread potential. Also, make sure your supplier has the most up-to-date technology for repairing, inspecting, and retreading.

Proper Treads

It is important to match the original or retread design to the application. There are several basic categories of tread design, but those most applicable to public works applications include off-the-road, light truck, or specialty treads.

Off-the-road treads are built to handle severe traction problems and as such have aggressive lug designs. They are heavy-duty treads built with deep tread depths to ward off nails, glass, and other road hazards that might be encountered off the highway. Many of these treads are on-and-off-the-road treads that are designed to handle severe off-the-road use, but also short-haul, slow-speed highway use.

Light truck treads are built for light truck use — vehicles with tire rim diameters 17 inches or smaller. A variety of tread designs give many combinations of traction and mileage options.

Specialty treads can go from light-weight treads to massive lug treads designed to handle rocks.

Besides tread design and characteristics, be aware of tread depth (skid depth). The optimum tread depth varies by tire position and application. For example, an off-the-road tire, because of the hazards it may encounter, needs a deeper tread than a highway tire.

Preventive Maintenance

The most important factor in tire maintenance is proper inflation. More tires are lost to underinflation or overinflation than any other cause. The tire gauge and tread depth gauge are the most important tools in any tire program.

When a tire is overinflated, it is more rigid. The more rigid the tire, the less able it is to absorb road shock, and the more vulnerable it is to road hazards that can lead to cuts, snags, punctures, and body breaks. Overinflated tires also wear faster. Overinflation does not compensate for overloading a vehicle. No matter how much air is added, the tire's carrying capacity will not increase above the maximum load. The only way to effectively address the overloading issue is to use more tire, not more air.

If overinflation is serious, underinflation is hazardous. It is the most common condition because no tire or tube is completely impervious to air loss. Sooner or later, air will have to be added. The reason underinflation is devastating to a tire is heat. Abnormal deflection of an underinflated tire causes friction within tire components that elevates temperature. Heat is the primary cause of premature tire failure, and it does not take much of an increase to make a difference in the life cycle of a tire. Once the correct pressure for the vehicle, load, and speed are determined using load and inflation tables available from the Rubber Manufacturers Association, regular pressure checks should be done weekly when tires are cold using an accurate pressure gauge.

Among the best ways to begin a tire pressure maintenance program is to have a fleet survey done by your tire supplier. A thorough fleet survey will give you a wealth of information about your tires and a picture of conditions that need correction to improve performance. Additional valuable information can be obtained from a failed tire analysis and tire comparison tests.

Tires mounted in duals must be matched so that the maximum difference between the diameters of the tires does not exceed 1/4 inch, or the circumferential difference does not exceed 3/4 inch. Mismatching causes the tire with the larger diameter to carry a larger share of the load, resulting in overloading and possible damage. The smaller tire, lacking proper road contact, wears faster and irregularly. Also, provide enough space between the duals to keep the tires from rubbing against each other.

There are many vehicle factors that have a major effect on tire life, including the condition of the braking and suspension systems. But the vehicle factor that has the most direct effect on tread wear is axle alignment.

Proper axle alignment delivers three important benefits: reduced tread wear, better vehicle handling and control, and less fuel consumption. Alignment not only refers to the various angles of the steering axle geometry, but also to the tracking of axles on the vehicle.

Main alignment settings consist of toe, camber, caster, toe-out-on-turns, and tracking. Toe determines how parallel steer axle tires are. It is the difference in distance between a measurement taken between the front of the tires and one taken between the rear of the tires. With toe-in, the fronts of the wheels are closer together than the backs. Toe-out is the opposite. New vehicle wheels are set with a slight toe-in to eliminate the tendency of wheels to weave from side to side. Excessive toe-in causes rapid wear on the outside shoulders of the tires. Toe-out causes rapid wear on the inside shoulders.

Camber is the angle the wheel tilts from the vertical. Positive camber is an outward tilt of the wheel, negative is inward tilt. Excessive camber results in rapid shoulder wear.

Caster is the backward (positive) or forward (negative) tilt of the kingpin when viewed from the side. Insufficient caster reduces stability and can cause wander. Excessive caster increases steering effort and can cause shimmy.

Toe-out-on-turns is the difference in the arcs steering wheels make in a turn. This difference is necessary to prevent the inside tire from scrubbing around a turn. It is accomplished by setting the steering arm and tie rod so that an imaginary line drawn from each steering arm converges on the centerline of the rear axle. The setting is not adjustable.

As for tracking, rear wheels should follow the front wheels on a parallel line when driven straight ahead. Out-of-track rear wheels will drive the vehicle off course, causing penalties in tire wear, fuel consumption, suspension wear, driver fatigue, and ultimately, safety.

The Retreading Process

Rebuilding a tire tread consists of initial inspection, buffing, ultrasonic inspection, measuring, casing preparation, repair and rebelting if required or as required, tread application, curing, and final inspection.

Initial inspection.–This may be the most important part of the retread process. More than half the failures of retreaded tires can be traced to poor initial inspection. Inspection begins when the tire is placed on a mechanical spreader and a drop-light is used to make the inside visible to the inspector. As the casing is rotated, the inspector checks inside and outside with his hands and eyes.

Buffing.–Buffing is simply the removal of previous tread material and the shaping, sizing, and texturing of the casing surface to receive the new tread. The casing is mounted on a lathe-type machine, inflated, and rotated as a powered buffing rasp removes material. The casing is buffed as flat as possible so there will be good tread-to-road contact. It is buffed to a predetermined crown width, radius, and symmetrical profile. Bias tires are "vented." Small holes are laced in the bead and shoulder areas to reduce buildup of air pressure within the tire cords while curing.

Ultrasonic inspection.–For damage that cannot be seen, quality retreading shops today use sophisticated ultrasonic equipment to check for hidden flaws like pinhole leaks, separations, and other damage hidden from visual inspection.

Measuring.–The casing is carefully measured either with an automatic device on the buffer or with a steel tape. In the case of a mold cure operation, this is to determine the proper mold fit. In the case of a precure operation, it is used to determine the length and width of tread to be used.

Casing preparation.–Injuries remaining or uncovered after buffing are then repaired. This involves skiving or "buzz out" of the injury with a powered rasp. This is a crucial operation. All exposed cords are trimmed, finished, and coated with cement as soon as possible after buffing and skiving. This is to prevent oxidation of the material. Steel belt cord needs to be protected within 15 minutes.

Repair.–Basically, repairing involves removal, filling, and reinforcing the injury and surrounding area. There are four basic types of repair:

- *Nail hole repair.*–A nail hole is an injury 1/4 inch or less in diameter in the crown area or 1/16 inch or less in the sidewall that penetrates 50 percent or more of the plies. Any number of nail holes in the crown or sidewall of a tire can be repaired as long as the repair patches do not overlap. Nail holes in the bead area of a tire cannot be repaired.

- *Spot repair.*–A spot repair is the removal and replacement of rubber in an injury that is larger than a nail hole, but involves less than 25 percent of the actual body plies. Any number of spot repairs can be made as long as the repairs do not involve body ply damage in the bead area.

- *Reinforcement repair.*–A reinforcement repair is repair of an injury through 25 percent, but less than 75 percent of the body plies (less than 50 percent in California). Local service bias drive tires may have no more than two reinforcement repairs in each quarter-section of the tire. No portion of a repair patch can overlap another. Reinforcement repairs are limited to bias tires.

- *Section repair.*–A section repair is made when an injury extends through 75 percent (in California, 50 percent) or more of the body plies or completely through the casing in the tread and sidewall areas. Bias ply section repairs are limited to two for local service drive tires. Radial section repairs are not limited by number, but no portion of a repair patch can overlap another.

Rebelting.–A machine with a knife cuts under the belts and peels them off the casing. When the belts are removed, the casing is buffed, prepared, and cemented. Then two new belt assembly packages are added followed by standard retreading procedures. In the precure process, a restraining ring is used to retain the shape while curing.

Tread application.–After repairs are made, the casing is then sprayed with cement to enhance adhesion between the new tread material and the casing. When the cement dries to a tacky consistency, it is ready for the tread. At this stage, the new tread material is fitted to the casing. It must be the precise size and it must be centered on the casing.

In precure operations, the proper length of the precured tread is cut, and a layer of cushion gum is added to the back of the tread. The cushion gum is the bonding agent between tread and tire casing. That bond becomes the strongest part of the tire. It reduces the chances of tread separation and increases the reliability of the retread.

The tread is then applied to the casing so as to distribute the tread evenly over the tire circumference. The tread may be applied manually or with the help of a machine. The ends of the cut tread are spliced together, temporarily stapled to hold them in place during curing, and then the tread is "stitched" to the casing to eliminate trapped air.

In mold cure operations, uncured rubber is added to the prepared casing.

Curing.–In the precure process, the prepared casing is covered with a flexible envelope fitted with an air exhaust valve, a curing tube is placed inside, and the casing is then mounted on a curing rim. Some systems use a special curing ring that replaces the curing tube and curing rim. The casing is then placed in the curing chamber, which is pressurized and heated while air is evacuated from between the tread and envelope, locking the tread in place. After a specific time and temperature (usually 3 to 4 hours at about 210 °F) the tread becomes bonded to the casing so well that it becomes the strongest part of the tire.

In the mold cure process, the casing and curing tube are placed in a mold. The curing tube pressure forces the raw tread rubber into the hot mold for a specific period of time and temperature. The tread rubber cures with the mold pattern imprint.

Final inspection.-Final inspection is made while the tire is hot because separations and other flaws are easier to see. The inside is checked to make sure all patches are properly bonded. The DOT number is checked. The tire is then trimmed of rubber flashing or overflow, staples are removed, the tire is painted, and then tagged for delivery.

This article is based on a booklet entitled "Truck Tire Retreading — A Guide for Fleet Owners, Managers, and Specifiers, published by Bandag, Inc. Copies are available for $5 each from Bandag, Inc., PO Box 414965, Kansas City MO 64141.

Rx FOR DAM REPAIR[1]

by Erik M. Bernard[2]

At two concrete gravity dams with similar freeze/thaw damage, distinctly different repair approaches were taken.

As increasing numbers of dams reach or exceed their design lives, it is essential that utilities, municipalities, and dam owners and operators maintain them properly and design repairs to extend their service life. Avoiding renovation work may minimize utility rates or taxes in the near term, but the long-term implications of neglect are disastrous.

Bridgeport Hydraulic Co., an investor owned water utility serving a population of approximately 360,000 in three Connecticut counties, has undertaken a comprehensive program to identify and schedule repairs on 21 dams it owns and operates. Most are concrete gravity dams constructed during the early part of the century and now exhibiting some spalling. Each dam is inspected annually by a registered professional engineer. The majority also fall under the guidelines of the U.S. Army Corps of Engineers' national dam inspection program.

Several factors affect the renovation approach taken at any dam, including the size and geometry of the dam, the depth and extent of concrete deterioration, the extent and source of leakage, and the cause of deterioration. An essential part of our renovation program was to conduct investigatory work to determine both the cause of deterioration and a general repair approach prior to detailed design and construction.

Two recent projects, at the Easton and Means Brook Dams, demonstrate the importance of this practice. Both structures are concrete gravity dams, both were constructed during the same period and both exhibited similar freeze/thaw damage. However, when we took a closer look we found that distinctly different repair approaches were called for in each case.

Easton Dam

Built in 1926, Easton Dam is 1,040 feet long with a maximum height of 123 feet and a top width of 12 feet. The downstream batter is 0.7:1; the upstream batter 0.05:1. The spillway is 100 feet long with an ogee crest. A total of 16 vertical construction joints run through the dam.

In 1978, a Corps inspection found many areas of surface spalling, joint spalling, and general deterioration on the dam's crest. The crest overhang, or coping, also showed signs of deterioration and spalling along most of its length, causing railing anchors to

[1] Reprinted with permission from Civil Engineering, a monthly publication of ASCE (American Society of Civil Engineers), November 1989 issue.
[2] Erik M. Bernard is Manager of Engineering Services at Bridgeport Hydraulic Co., Bridgeport, Connecticut.

pull from the concrete. The downstream face was more severely deteriorated. It exhibited large areas of surface spalling, joint spalling, and efflorescence.

Minor seepage was evident at the vertical construction joints, where deterioration and loss of material was greatest. These problems were strictly esthetic at the time of inspection, but left unchecked they could eventually affect the structural integrity of the dam.

The consulting engineer, Spiegel and Zamencik, Inc., New Haven, Connecticut, found that seepage on the downstream face varied seasonally, with greater quantities of moisture in colder weather. This pattern held even during dry periods, so leakage through the structure rather than dampness from precipitation was the cause. We do not know if waterstops were originally installed in the vertical joints. If they were, they had deteriorated under repeated expansions and contractions.

Freezing and thawing of reservoir leakage on the downstream face, and of ponded water on the crest, was the major cause of the concrete deterioration and spalling. With freeze/thaw damage visible as deep as 6 inches in core samples, a considerable amount of concrete had to be removed prior to resurfacing.

Freezing and thawing of reservoir leakage on the downstream face and ponded water on the crest of Easton Dam caused deterioration and spalling.

After considering several installation methods to seal the vertical joints, we drilled 6-inch-diameter core holes through the vertical construction joint from the top of the dam and filled them with chemical grout. This is a relatively new technique, which the Corps developed and has used with very good results.

Divers from Wiswell, Inc., Southport, Connecticut, conducted an underwater inspection of the upstream face and joints. We found that concrete and joint deterioration was limited to the upper 25 feet of the dam. This is also the portion subjected to the greatest water temperature differential and fluctuating reservoir levels. Based on this information and the joint movement calculations, we decided to install joint waterstops 35 feet below the top of the dam.

We evaluated a number of alternative filler materials, through a literature search and contacts with grout manufacturers and installers and the Corps. A urethane foam-type grout exhibited the best properties for application in the Easton Dam. These two-compound chemical grouts expand to eight times their volume upon contact with water, forming a sponge-like material. Their adhesion, stretchability, and nonshrinkage properties also make urethane foam grouts attractive. The urethane foam grout waterstops were installed in the fall and winter of 1983 at a cost of $97,000.

Drill holes 6 inches in diameter and either 35 feet deep or 5 feet into bedrock, whichever condition was met first, were located 2 to 3 feet from the upstream face and centered over each of the 16 vertical construction joints by the diamond rotary method, which was performed by Structural Preservation Systems, Baltimore, Maryland. To ensure that the drill holes stayed on center and straddled the joint for the entire depth, down-the-hole closed circuit video equipment constantly monitored the relative location of the joint in the drill hole from above.

The Corps has successfully installed urethane foam grout by soaking burlap rags in the uncured compound and tamping them, one by one, into drill holes with a drilling rod on a line. Uniformity of grout product throughout the depth of the hole is important. Since the drill holes recommended for Easton Dam were longer and larger than any previously used with this grout, a more reliable method of application was needed.

Rather than plunging the burlap rags into place one by one, the soaked rags were stapled to sections of 2-inch-diameter perforated PVC pipe and reamed into place manually. Once a pipe section was plunged to just above the grade of the dam crest, it was coupled to the next section so that plunging could continue. A temporary joint sealant approved for potable use was applied under water to the surface of the joints on the upstream face to prevent the grout from escaping through the joint into the reservoir water.

This arrested 80 percent or more of the seepage reaching the downstream face. Feeling that further efforts would result in diminishing returns and that seepage could not be entirely controlled in any event, we elected to make allowances for continued minor seepage when designing repairs for the concrete surface.

Crest repairs were undertaken first, since crest renovation was independent of leakage. Resurfacing the entire dam in one project was impractical due to its size. Prior to detailed design, additional core samples were obtained to provide a more detailed description of the existing concrete. Results indicated the cause of concrete deterioration was cyclic freezing and thawing on non-air-entrained concrete.

Based on the depth of freeze/thaw damage discovered in the core samples from the crest, at least 3 inches of existing concrete had to be removed prior to resurfacing. This was done by hydrodemolition. This method is commonly used in bridge deck repairs, but we believe this was its first application in a dam renovation project.

Reinforcing steel was specified for the new concrete to control shrinkage cracks. Dowels were specified to better anchor the new surface to the existing concrete. To control curling, hook bars were placed at the ends of each monolithic dam section between vertical construction joints. Epoxy coated steel was used to prevent future corrosion damage.

The additional reinforcing steel required a 9-inch overlay to provide the required depth of cover. This meant the new dam would actually be higher than the old. However, stability calculations determined this would have no adverse effect.

Durability of the new concrete was a major consideration. To minimize future problems with spalling and freeze/thaw damage, a 5,000 lb/in^2 minimum 56-day strength concrete mix was specified. An elastomeric sealant was applied at the vertical construction joints to keep water from entering.

Several design improvements were incorporated into the new crest. The cap was designed to be slightly wider, with railings attached to the outside faces for still more usable width. Pitch was provided both away from the vertical construction joints and toward the downstream face to eliminate standing water. To maintain the crest width throughout and allow heavy equipment to pass — making future renovation projects easier — an overhang, or cantilever section, was installed where the upper gatehouse extends into the crest.

The crest renovation was completed for $700,000 in the fall of 1988. Work on the downstream face of the dam is currently on hold, pending further evaluation and monitoring of joint seepage and additional data on potential resurfacing materials and application techniques. With most of the seepage and associated deterioration arrested, there is no urgency to the repairs.

Means Brook Dam

Means Brook Dam was built in 1916 in Shelton, Connecticut. Its abutments and foundation were keyed into rock. At its top, the dam is 527 feet long with a maximum height of 59.5 feet. The crest is 10 feet wide and supports a 2.1-foot concrete wall, which was added in 1977 to increase spillway capacity. The maximum base width is 35 feet. The upstream face is concrete and is vertical in cross-sectional view. The downstream face is also vertical and is battered at 0.7:1 from the toe to a distance of approximately 24 feet from the top of the dam. There are six vertical construction joints.

In 1979, the Corps inspectors recommended further study of spalled concrete on the downstream face and top of the dam, and urged rehabilitation of the dam to its original condition. As with Easton Dam, spalling and efflorescence posed no immediate threat to the structural integrity of the dam.

Consulting engineer on the project was Harza Engineering Company of Chicago, Illinois. In 1986, Harza performed a concrete condition assessment and identified a repair approach.

An underwater inspection of the upstream surface found that the vertical construction joints contained a bitumastic filler material apparently used as a waterstop. Deterioration of the concrete edges at the vertical joints was no greater than that of the dam face on either side of the joint.

Precast facing panels will be installed at Means Brook Dam to reduce continued freeze/thaw damage to the original downstream face.

Deterioration was greatest in the top 10 feet — from normal pool elevation to maximum typical drawdown. The apparent cause was freeze/thaw cycling. In two areas, moderately greater deterioration was found in the form of shotcrete lenses from previous repair work, which were ready to spall. No horizontal joints were visible on the dam face.

Stability analyses were performed for the maximum section of the dam under probable maximum flood (PMF) loading and normal pool plus ice loading conditions. The results indicated that moderate tension developed in the heel under both conditions. Under cracked-section analysis, the normal pool plus ice condition remained stable, while the PMF condition theoretically caused failure of the dam.

Concrete core and rock interface samples tested for tensile strength indicated the dam could withstand 100 times the tension developed from the PMF. When the dam was checked for shear, the required values ranged from 12 to 74 lb/in^2. These are relatively small compared to the shear strength of the in-place concrete and the greater shear strength encountered in the bedrock.

Visual and petrographic analysis showed that Means Brook Dam had only been affected by exposure and weathering. This effect was superficial, caused by freeze/thaw action on saturated or near-saturated, non-air-entrained portland cement concrete. The original concrete was found to have a water to cement ratio of 0.71, by weight, characterized by bleed channels and separation under coarse aggregate particles, which contributed to the deterioration of the dam surface.

After review of several repair approaches, the consultants recommended installing precast facing panels on the downstream face as the best repair solution. In addition to cost advantage, precast panels were selected because of their ability to greatly reduce freeze/thaw cycling, provide a uniform and maintenance-free surface, and reduce the amount of deteriorated concrete to be removed. Design work was completed in January 1988.

Design of the downstream facing of the dam resulted in 20-foot by 4-foot by 4-inch precast panels, nominally reinforced for temperature and shrinkage. A 5,000 lb/in^2, 28-day concrete mix was chosen for the panels. Embedded metal guides were designed to guide the panels into W8x28 structural steel frames, which extended vertically from the crest to the earthen embankment on the downstream face. The second panel from the top in each vertical column of panels was designed to be removable, so lower panels could be slid out of place for maintenance.

A W10x49 base beam and a W8x28 intermediate support beam embedded into sound concrete on the downstream face will support each steel panel frame member. One result of settling on structural steel frames for panel support was that the panels could not follow the contour of the downstream face, resulting in an increased void between the panels and the dam.

Insulation between the panels and the dam to reduce freeze/thaw cycling consists of 2 inches of styrofoam affixed to the back of each panel. This will reduce heat transfer by an estimated 35 percent. Additional design features include access hatches to allow entry between the panels and the dam for inspection and temperature probes to measure the actual reduction in freeze/thaw cycling. The crest will be resurfaced by removing deteriorated concrete and installing a new layer of cast-in-place concrete, increasing the height of the dam by 4 inches.

RECLAMATION USE OF CONCRETE SEALING COMPOUNDS

by W. Glenn Smoak[1]

The Bureau of Reclamation has concrete structures located in some of the most severe environments in North America. Temperature variations of 50 to 80 °F are common on a daily basis. The majority of our concrete structures are involved with water resources such that these temperature extremes are then combined with frequent exposures to wetting and drying. Even though high-quality concrete materials are properly mixed and constructed, the resulting concrete will have a natural porosity that permits it to absorb 3 to 8 percent water, by weight. This water will expand upon freezing and can result in freeze-thaw deterioration of unprotected concrete. The use of air-entraining admixtures has significantly reduced this damage, but freeze-thaw damage still may occur when improper materials or construction techniques are used. Additionally, there are numerous Reclamation concrete structures that were constructed long before air-entraining admixtures were used. These structures are subject to rapid deterioration if exposed to cyclic freeze-thaw conditions while critically saturated.

There are other conditions such as the presence of soluble sulfates, corrosive environments, various types of cracking, and numerous poor construction practices that can result in accelerated concrete damage when combined with frequent wetting and drying.

Since the first use of concrete as a construction material, scientists and engineers have recognized the need to protect concrete from the problems caused or made worse by water penetrating its surfaces. Historically, mineral spirits systems, oils, paints, and various other coatings and additives, were applied to "waterproof" concrete. In most instances, the very best that can be said about these waterproofing systems is that they caused little harm.

Currently, we are experiencing extensive efforts by manufacturers and suppliers to promote materials for use in sealing concrete surfaces. Many of these materials are supposed to penetrate the concrete surface, react chemically with some component of the concrete to form a new protecting compound, and remain in the concrete for extremely long periods of time. Application of these materials is purported to be very easy, requiring little or no preparation, to wet or dry concrete. Unfortunately, the claims contain little or no technical information of value in making sound engineering judgments concerning the use of these materials. Frequently, the claims of one product conflict with the claims of other products.

The concrete construction industry has become aware of these new sealers and studies are underway to test, evaluate, and classify the new compounds. The American Concrete Institute (ACI) has recently proposed a definition for these materials as follows:

> "Concrete Sealing Compound — A liquid that is applied to the surface of hardened concrete to prevent or decrease the penetration of liquid or gaseous media — such as water, aggressive solutions, and carbon dioxide — during the service exposure; preferably applied after initial drying to facilitate its absorption into voids and cracks."

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This definition has not been officially adopted by ACI as of the date of this article, but the proposed ACI definition will be used in the remainder of this discussion of sealing compounds.

The Concrete and Structural Branch, Research and Laboratory Services Division, has begun a series of screening tests and studies to evaluate sealing compounds. The objective of this program is to gain sufficient knowledge of sealing compounds to enable us to confidently specify materials for use on Reclamation projects. These studies are being performed in three broad phases.

The first phase, which has been underway for over a year, consists primarily of a series of laboratory screening tests of broad generic types of sealing compounds. Currently, we are testing a siloxane, a silane, a metallic stearate, several high molecular weight acrylics, several epoxies, and several types of oils including mineral spirits and linseed oil. A simple set of tests is being performed to reflect the performance of the sealing compounds. These tests include:

1. Water absorption
2. Electrical conductivity
3. Freeze-thaw resistance
4. Water vapor transmission
5. Crack sealing capability
6. Outdoor exposure
7. Hydrophobicity

As a result of these studies, we have selected two of the sealing compounds, a high molecular weight acrylic and a high solids content siloxane, for field applications. These tests will continue.

The second phase of the study, which will be started early in FY91, is the comparative screening of individual sealing compounds within the generic classifications. The initial efforts will involve products from the siloxane, metallic stearate, and acrylics groups. In these tests, we will try to determine which of the proprietary products are best suited for Reclamation applications.

The third phase of the study, field testing and evaluation, was started about 2 years ago. An acrylic sealing compound has been applied to Kortes Dam, American Falls Dam, and Nambe Falls Dam. A high solids content siloxane has been used in a major application at Grand Coulee Dam and in a small application at Nambe Falls Dam. Both materials are performing well so far. This phase will be expanded as laboratory screening tests reveal promising sealing compounds and as fieldsites become available. Assistance is requested from Reclamation project offices in locating suitable sites for these tests.

The concrete sealing compound studies are being performed under the Concrete Materials Systems Technology for Construction and Rehabilitation Research Program, NM-049. The Principal Investigator for this program is the author of this article; and questions, suggestions, or discussion concerning the need for or use of sealing compounds on Reclamation projects may be referred to him.

The following article on concrete sealing compounds appears to reach contradictory conclusions concerning the value of (alkoxy) siloxane sealants from the previous article ("Reclamation Use of Concrete Sealing Compounds"). Reclamation studies indicate that this material is one of the more effective sealing compounds while Corps of Engineers' studies reported by Husbands and Causey indicate that the alkoxy siloxanes did not offer a high degree of protection to concrete. Mr. Glenn Smoak, Division of Research, Bureau of Reclamation, contacted Mr. Husbands by telephone to discuss these differences in performance and to try to resolve the controversy. During this discussion the test methods and the materials used in the tests performed by each organization were compared. It was determined that the alkoxy siloxanes tested by the Corps contained less active solids than the material tested by Reclamation. The average solids content of the material in the Corps tests was 4 percent, by mass, while the material tested at Reclamation contained 20 percent solids, by mass. It is believed that the differences in our respective test results are due to this difference in active solids content, and caution potential users of the alkoxy siloxane sealants to specify a solids content in the 10 to 20 percent range (by mass).

SURFACE TREATMENTS FOR CONCRETE[1]

by Tony B. Husbands and Fred E. Causey[2]

Freezing and thawing, penetration of salts, weathering, chemical attack, and erosion cause concrete surfaces to deteriorate. Surface treatment of the concrete with a material more resistant to these forces than concrete can slow or even eliminate the rate of deterioration. However, little guidance in the selection and application of these treatments is available.

One of the work units in the Repair, Evaluation, Maintenance, and Rehabilitation (REMR) Research Program was designated to evaluate surface treatments that would minimize concrete deterioration. Emphasis was on materials that can reduce or prevent damage to concrete from cycles of freezing and thawing, the major contributing cause of non-air-entrained concrete failure.

Materials

Surface treatment materials were separated according to viscosity, total solids, manufacturers' recommended use, and chemical composition. The materials were then classified as concrete sealers, concrete coatings, shotcrete, and thin overlays.

Concrete sealers are the preferred surface treatment to minimize or prevent damage from freezing and thawing. Sealers damp proof and slow or prevent the intrusion of salts into concrete. They are usually low in viscosity and total solids. The generic types tested were acrylics, epoxies, polyurethanes, silicates, silanes, silicones, siloxanes, stearates, and those hydrocarbons made from petroleum distillates or oils, excluding linseed oil.

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[1] Reprinted from the REMR Bulletin; an information exchange bulletin of the U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, Mississippi, October 1989 issue.
[2] Tony B. Husbands and Fred E. Causey are a research chemist and a research chemical engineer, respectively, in the Engineering Sciences Branch of the Concrete Technology Division, Structures Laboratory, Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, Mississippi 39180-6199.

Field testing of a penetrating concrete sealer, Brandon Road Lock and Dam, Rock Island District, Illinois.

Concrete coatings are used for reducing ingress of water; for protecting against erosion, chemical attack, and weathering; and for resisting graffiti. They may be thin polymer systems used for sealing cracks in concrete by topical application. Some may be applied under water. One difficulty in using coatings is finding materials that can be used on cracked concrete surfaces because cracks migrate through most coatings.

Elastomeric coatings, crack sealants, and low viscosity polymer systems for sealing cracks by topical application were evaluated. Elastomeric coatings can bridge cracks and yield under any crack movement. Manufacturers of these types of coatings recommend routing out wide cracks and then either sealing the cracks before applying the coating or using fabrics under the coating.

Shotcrete was investigated as a coating for deteriorated concrete. Two problems may occur if a conventional shotcrete mixture is used: (1) poor resistance to freezing and thawing and (2) cracking of the material. Therefore, shotcrete with two different latex admixtures was evaluated as a possible solution to these problems.

The study also included a few commercial latex-modified mortars and a number of latex admixtures used to prepare latex-modified mortars. These materials are generally used as thin overlays on existing concrete structures.

Tests

A surface treatment must prevent water from entering non-air-entrained concrete to protect it from damage caused by freezing and thawing. A published water-absorption test (ASTM C 642-82)[3] was selected to screen materials for use in surface treatments. Tests used to evaluate the different materials included water vapor transmission (REMR report to be published), resistance to freezing and thawing (ASTM C 672-84)[3], bond strength to concrete (ASTM D 4541-84, ASTM C 882-87)[3], accelerated weathering

[3] American Society for Testing and Materials, 1988 Annual Book of ASTM Standards, Philadelphia, Pennsylvania.

(ASTM G 53-84)[4], total solids (ASTM D 1259)[3], resistance to abrasion (underwater abrasion test), viscosity (ASTM D 1824)[3], and dry-to-touch.

Test Results

Sealers.-Results of water-absorption (WA) and water-vapor transmission (WVT) tests for the large number of concrete sealers are reported by generic types in percent of the control (uncoated concrete cubes). The average WA for controls was 4.69 percent after 2 days soaking in water; early tests indicate that these specimens had almost reached water saturation after 8 hours. The average WVT for controls was 3.21 percent after 4 days.

Two possible causes of poor performance in the WA tests are low solids content of the sealer and the application rate. With one exception, the five acrylic sealers that performed poorly had solids content of less than 10 percent.

Most of the sealers that performed unsatisfactorily on the WA test were not tested for WVT. Of the 10 acrylic sealers tested, 7 had WVT values of 50 percent or higher. A search for a criterion for WVT in concrete was not successful. If the criterion, WA <15 percent and WVT >25 percent, had been used in the selection of sealers, very few of the 68 sealers tested would have qualified.

Some of the sealers that performed well on the WA test were selected for the accelerated weathering test (AWT). The results are shown in Table 1. The acrylic sealers were significantly affected by the AWT. They had been expected to perform much better as they are described as having good weatherability. The only explanation for the continued improvement of the two linseed oil-treated test blocks is that the oil continued to polymerize in the heat and UV light.

Table 1. - Accelerated Weathering Test Results

	Water absorption, % before testing (Material No.)				Water absorption, % 1,600 hours testing (Material No.)			
Generic type	1	2	3	4	1	2	3	4
Acrylic	0.55	0.61	0.72	0.87	2.56	3.12	4.00	3.94
Hydrocarbon	0.47	0.65	0.40		0.94	3.57	4.61	
Linseed oil*	4.50	1.57			0.54	0.88		
Polyurethane	0.34	0.53	0.22		0.87	1.44	0.87	
Silane	0.52	0.56	0.60	0.44	0.66	0.70	0.80	0.57
Silicone	0.44				0.47			
Siloxane	0.56	0.54	0.59		0.71	0.68	0.73	
Stearate	0.93	0.67			1.96	1.00		

* Material 1 is an emulsion and 2 is linseed oil in mineral spirits.

[4] 1987 Annual Book of ASTM Standards.

Tests results showing the resistance of concrete coated with sealers to deicing salts are shown in Table 2. Any rating above "2" was not considered satisfactory. All epoxy resins were effective in preventing scaling. The silanes and siloxanes did not perform as well as expected; the difference in the concrete mixtures used for the two tests could have been a factor.

Table 2. - Resistance to Scaling, Concrete Sealers

Generic type	No. tested	Visual rating of surface (No. materials for each rating) 0	1	2	3	4	5
Control	2					1	1
Acrylic	9	2	3	3		1	
Epoxy	4	4					
Hydrocarbon	3			1		1	1
Linseed oil	1			1			
Methyl methacrylate	1	1					
Polyurethane	5	3					2
Silane	5	1*			1	1	2
Siloxanes	6			1	3	1	1
Stearate	2				1		1

Rating 0-no scaling, 1-slight scaling, 2-slight to moderate, 3-moderate scaling, 4-moderate to severe, 5-severe

* Silane treated surface coated with acrylic sealer.

Coatings.-The WA and WVT tests used to evaluate the sealers were also used with the coatings. Most of the coatings effectively prevented water from entering concrete; exceptions included some of the acrylics, some of the cementitious, and one polyurethane. One reason for the high WA of these materials is that thicker coatings of the cementitious and acrylic mastics on the small cubes were not uniform, especially over the edges. A few pinholes were observed in some of the thicker water-based acrylics and a few of the polyurethane coatings.

The acrylic coatings had the highest WVT values. Most of the coatings bonded well to concrete. The soft elastomeric coatings, such as the acrylic mastics and silicone, had the lowest bond strength values. These coatings will not be satisfactory where wheeled traffic is expected.

Fourteen coatings were tested for resistance to scaling. All performed satisfactorily except for one cementitious coating and an acrylic coating. The cementitious coating began peeling off the surface after 8 cycles; the acrylic lost bond to the concrete surface.

Four polyurethane graffiti-resistant coatings were tested. They were effective sealers, and graffiti (enamel paint) was removed from them with ease. The coatings were not affected after graffiti had been applied and removed three times. Two-component coatings performed better in the AWT than single-component coatings.

Eight high-molecular-weight methacrylate (HMWM) monomers (viscosity 9 to 33 cp) and one low-viscosity epoxy resin (40 cp) were applied topically and then evaluated as crack sealants. High bond strengths were obtained for all materials, but the epoxy did not penetrate into thin cracks as well as the HMWM. Waterways Experiment Station (WES) worked with the U.S. Army Engineer District, Kansas City, in preparing specifications and guidance for sealing cracks in a bridge deck with HMWM and assisted Air Force personnel in sealing pavements that contained numerous cracks.

Shotcrete.–Latex-modified shotcrete was applied to concrete and plyboard to obtain panels for testing. Prisms cut from the panels were tested in accordance with ASTM C 666-84. The mixtures had good bond strength to concrete and durability to rapid freezing and thawing. Dilutions of latex to water ranging from 1:2 to 1:4 were found best for application. A defoamer had to be used with the acrylic latex. Satisfactory compressive and flexural strengths were obtained, with the flexural strength increasing with increases in the amount of latex used. A petrographic examination indicated that the latex actually entrained some air into the shotcrete mixtures. Polypropylene fibers added to some of the mixtures reduced cracking.

Overlays.–A number of commercial, prepackaged latex-modified mortars were tested for bond strength, concrete freeze-thaw durability, and flexural and compressive strengths. In addition, these mortars were tested as thin (13 mm) (1/2 inch) overlays for 1.2-m-long (4-foot-long) concrete panels. Three of the materials had satisfactory test results. Acrylic latex and styrene-butadiene were used to make two mortar mixtures which were evaluated by being immersed in water for 6 months. The styrene-butadiene latex mixture tested better.

Conclusions

The test data for surface materials indicate wide differences in the effectiveness of these materials for protecting concrete or minimizing concrete deterioration. However, results of these tests and other tests found in literature allowed the following criteria to be established as guidance in the selection of concrete sealers. Standard specifications have not been established.

Test	Requirement
Water absorption, percent of control 7 days	≤15
Water vapor transmission, percent of control 7 days	≥25
Accelerated weathering, percent difference after aging for 1,200 hours	≤0.50
Scaling resistance (ASTM C 672-84) 50 cycles, 4 percent $CaCl_2$ solution	Must have a visual rating of 1 or less

Of all the sealers tested, only one hydrocarbon, two siloxane, and two silane sealers met the criteria.

Sealers for specific applications will require additional testing; for example, a sealer to be used in an area subject to abrasion will need to be tested for abrasion resistance. However, a sealer that will not be subjected to freezing and thawing will not have to be tested for freeze-thaw resistance.

Results of the study indicate:

- Siloxanes perform well as a generic-type concrete sealer, except in resisting cycles of freezing and thawing.

- Acrylic mastic coatings for treatment of cracked concrete tested well in the laboratory, but field application results are not available.

- Polyester resin coatings tested can be used effectively as abrasion-resistant coatings if applied to dry concrete.

- Only those polyurethanes recommended by manufacturers for sealing concrete surfaces subjected to vehicular traffic performed effectively in laboratory tests.

- The HMWM monomer systems can be used to seal cracks by topical application.

- Two cementitious coatings tested were found effective for waterproofing concrete, from both positive and negative sides, and may minimize concrete deterioration resulting from freezing and thawing. Others tested did not produce the desired results.

- The addition of latex admixtures improves the freeze-thaw durability of shotcrete.

- Polypropylene fibers appear to reduce cracking caused by drying and shrinkage in latex-modified shotcrete.

Testing of the freeze-thaw resistance of siloxanes and other types of sealers continues. Different application rates and concrete mixtures are being used in these tests.

For further information on this subject, contact Tony Husbands at (601) 634-3275 or Fred Causey at (601) 634-3590.

POLYURETHANE INJECTION STOPS WATER TUNNEL LEAKING[1]

by W. Glenn Smoak[2]

A valve at Pacheco Conduit inlet in California lets workers drain the conduit without draining the 5.3-mile-long tunnel that feeds it. During tests after construction in 1986, however, Bureau of Reclamation engineers found a 125 gal/min leak at the concrete valve structure expansion joint. The leak shown in figure 1 progressed up the joint as the conduit filled.

Figure 1. - Before repair, water leaks through an expansion joint in a California water tunnel project. The inset photo shows the area of the leak.

Engineers stopped the tests and partially drained the tunnel to get an inside look at the leaking joint. Although a waterstop had been installed at the joint (figure 2), inadequate concrete consolidation was permitting water to bypass the waterstop. Soundings and visual inspections also revealed two large voids, one at the joint invert and one at the top.

Removing and replacing the valve structure concrete would have been too costly. Repair by epoxy injection was rejected because liquid epoxy would penetrate the sponge rubber filler in the expansion joint, gluing the faces together. Then the repaired section would not be able to accommodate seasonal temperature change movements between the tunnel portal and valve structure. We needed a way to plug the leak while still allowing movement at the joint.

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[1] Reprinted with permission from the Managing Editor, Concrete Construction, 426 South Westgate, Addison, Illinois 60101, December 1988 issue.

[2] W. Glenn Smoak is a Research Civil Engineer employed by the Bureau of Reclamation, Concrete and Structural Branch, code D-3731, PO Box 25007, Denver, Colorado 80225.

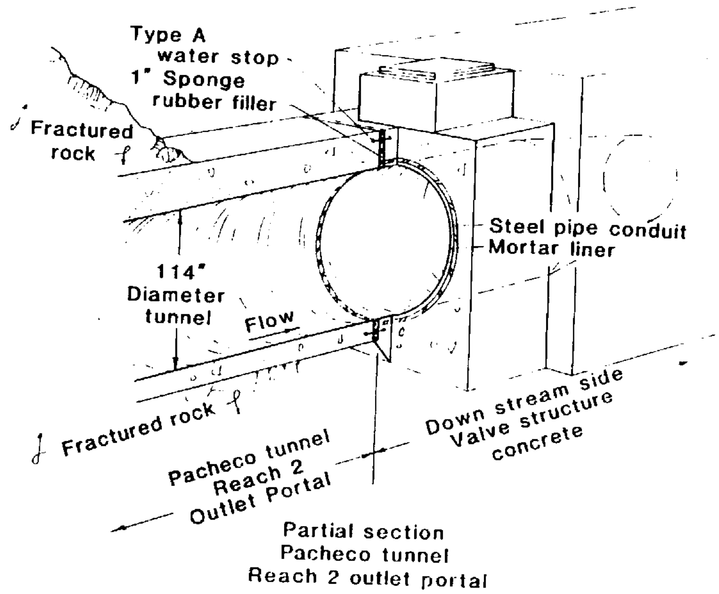

Figure 2. - Expansion joint between the Pacheco Tunnel and a steel pipe conduit includes a 1-inch sponge rubber filler and a waterstop around the joint circumference.

Injected Resin Turns to Foam

On several other projects, the Bureau of Reclamation had successfully repaired smaller leaks using hydrophilic polyurethane resin, injected like an epoxy. The urethane resin reacts with water to form an expansive foam that cures to a tough, flexible, cellular rubber.

If unrestrained, the reacted foam has a very low density and a volume 10 to 12 times that of the initial resin. Containing the expansion produces a higher density material better suited for plugging leaks in concrete structures. The Bureau prepared specifications and contracted with TeraLite, Inc., of San Jose, California, to do the work with a commercially available hydrophilic polyurethane resin.

Resin Injected Outside and Inside the Tunnel

The contractor began the repair by drilling a series of injection holes on 12-inch centers around the exterior of the joint. Holes started in both the tunnel portal concrete and the valve structure concrete were angled to intercept the joint at a depth of 8 to 12 inches (figure 3). Because the valve structure lay on bedrock, no exterior injection holes were drilled at the bottom.

Workers installed plastic injectors in the holes, then washed and flushed each hole with water. This showed which holes would accept resin and indicated relative flow rate. Also, the washing ensured that the joint contained enough water to react with the injected resin.

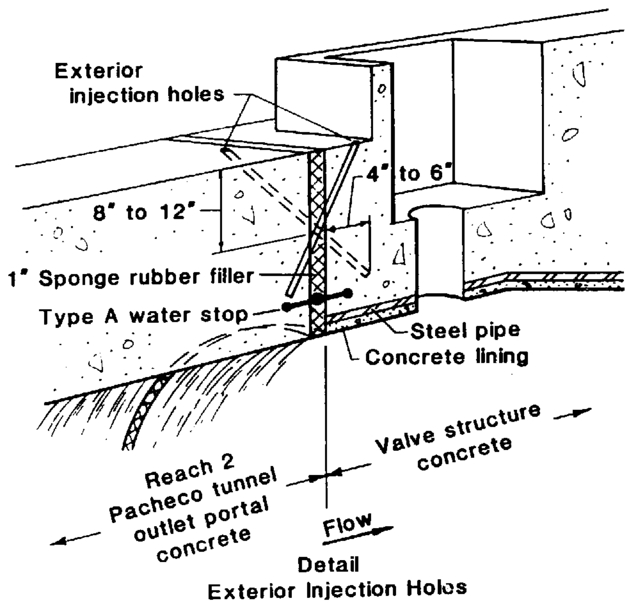

Figure 3. - Injection holes drilled at an angle from the outside intercept the joint at a depth of 8 to 12 inches. At the invert, only interior holes were drilled.

Using a positive displacement, airless pump, workers injected the resin (figure 4). During injection, the contractor monitored resin leakage from outside and inside the joint. When too much resin leaked out (figure 5), workers stopped injecting and forced resin-soaked oakum strips into the joint to contain the foam. After injecting all exterior ports, the contractor let the resin cure overnight.

Interior holes were also drilled and injected in the same way the exterior ones had been done (figure 6). The contractor drilled interior holes from the tunnel portal side of the joint, at the invert, and around the tunnel circumference. No holes were drilled through or into the mortar-lined steel pipe. About 25 gallons of urethane resin were injected into the expansion joint.

The day after interior joint injection, the tunnel was filled with water to test the repair. Water leaked at less than 0.5 gal/min. Workers drilled and injected 11 more holes with 4 more gallons of resin to stop this minor leakage. After removing excess foam from concrete, they patched the injection holes with expoxy mortar. Since the repair 2 years ago, no leaks have occurred.

Figure 4. – Workers inject urethane resin into the joint with a positive displacement, airless pump.

Figure 5. – Excessive resin leakage wastes material. When this happened, the contractor stopped injecting and forced resin-soaked oakum strips into the joint. This contained the foam.

Figure 6. – Workers also drilled and injected resin into holes inside the water tunnel. Resin injected from the outside of the tunnel portal extrudes from the joint.

SPOTLIGHT ON STONY GORGE DAM & RESERVOIR

Orland Project, California

The rapid development of the Central Valley in California began in 1849 after the discovery of gold. Cattle raising was the primary activity for the next decade; but at the same time, various forms of agriculture were being established.

As early as 1909, preliminary surveys had been made on the Stony Gorge Dam site. Supplemental surveys made in 1918 confirmed that an additional water supply for the project might be expected at this site. A vote of the water users, following a dry season in 1924 when there was considerable agricultural loss, resulted in unanimous approval of the dam. The dam was started in 1926 and finished in October 1928.

Stony Gorge Dam is located on Stony Creek about 30 miles southwest of Orland, California. The dam is an Ambursen-type dam with a crest length of 868 feet, a crest width of 9.75 feet, and a structural height of 139 feet. It is comprised of 46 bays of slab-and-buttress construction, with buttresses spaced on 18-foot centers. At each abutment, the dam terminates in a short, massive gravity section. The face slabs vary from 15 inches thick at the top of the dam to 50 inches thick at a depth of 120 feet.

The flat-slab buttress-type dam was developed and patented by Nils F. Ambursen, a mechanical engineer. One of the reasons for selecting the Ambursen-type dam was because the noncontinuous slabs, acting as simple beams between adjacent buttresses, were considered best adapted to accommodate possible minor foundation movements. A tight and relatively minor fault crosses the damsite along the north bank of the creek channel.

The reservoir, which has a storage capacity of 50,000 acre-feet, regulates flows along the lower reaches of Stony Creek, and stores surplus water for irrigation purposes. Releases from the reservoir travel 22 miles down Stony Creek to the project's diversion points.

A Safety of Dams modification completed in 1985 consisted of constructing a concrete protective slab on the right abutment, removing the parapet wall on the crest of the dam, constructing a new counterfort wall on the left side of the crest, and modifying the spillway gatehouse.

Instrumentation at the dam consists of collimation points and levels of the buttresses to determine horizontal deflections, and water level measurements in six foundation drill holes.

The spillway is located to the left of the two center bays and left of the fault line. It occupies six bays of the dam and is divided into three equal openings. Discharges are controlled by three 30- by 30-foot structural steel caterpillar or crawler overflow type gates. These gates move down the 45° slope of the upstream face of the dam to open. There is an ogee or reverse curve face slab on the downstream side of the spillway terminating in a concrete apron extending 50 feet below the dam. A gatehouse over the spillway gates contains gate hoisting machinery and a traveling crane.

The outlet works is located between buttresses 35 and 37, occupying the two bays which are to the right of the center bays of the dam and to the right or north side of the fault. Two 3.5- by 3.5-foot high-pressure guard gates are embedded near the upstream ends of the outlet pipes. The gatehouse between buttresses at the underside of the slab contains the high-pressure guard gates and operating equipment. The outlet works discharge is controlled by two 42-inch-diameter fixed-cone valves, one on each of the two 50-inch-diameter riveted steel outlet pipes. A valve house between buttresses at the downstream toe of the dam contains the fixed-cone valves (which replaced the needle valves in 1985) and operating equipment.

The steel outlet pipes were modified as penstocks for a two-unit powerplant which went on-line in April 1986. The powerplant, which is owned and operated by the city of Santa Clara, consists of two Francis turbines rated at 110 feet and two synchronous generators rated at 2.5 megawatts. The city of Santa Clara and Resource Management International, Inc., are jointly responsible for the maintenance of the outlet pipes and fixed-cone valves. With the addition of the powerplant, the old balanced needle valves were replaced with fixed-cone valves. The fixed-cone valves can be either operated manually from the valve house or can be set to operate automatically in conjunction with powerplant discharge to meet downstream water requirements. When the control panel is in the automatic mode, a penstock shutdown, created by closure of the counterbalanced butterfly valves in the powerplant, will open the fixed-cone valves to maintain the required downstream releases.

The principal crops in the area are irrigated pasture, wheat, alfalfa hay, sorghum, olives, nuts, and citrus fruits. Dairying is an important business due to the mild climate, good market, and feed conditions. The hills and mountains west of the project are used extensively for grazing of sheep and cattle.

Stony Gorge Reservoir area provides campgrounds, including trailer space, picnicking areas, swimming, boating, and fishing, primarily for bluegill and largemouth bass. Management of the recreation facilities is under the jurisdiction of the Bureau of Reclamation.

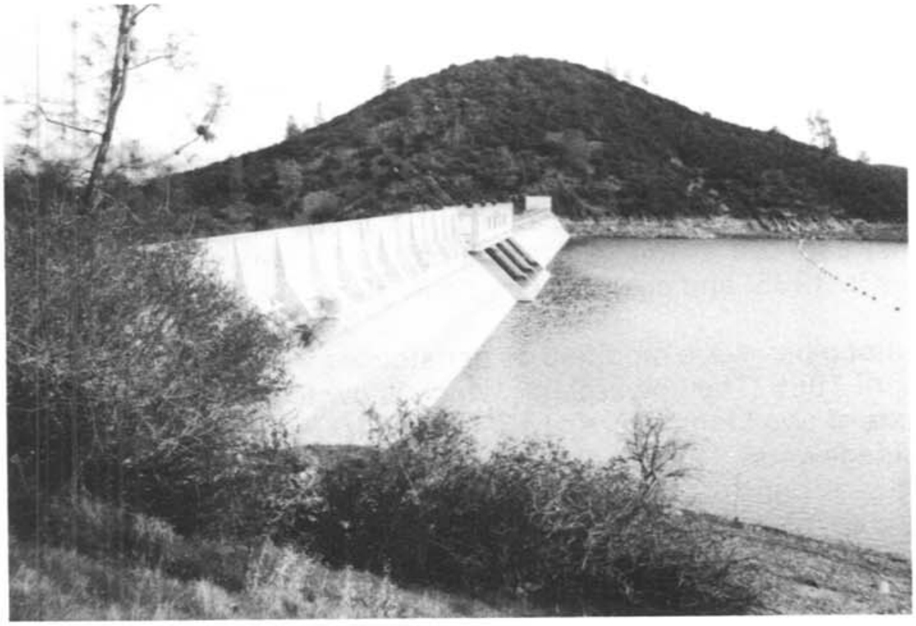

Figure 1. – Upstream view of Stony Gorge Dam from the left abutment showing the counterfort wall to the left of the spillway gatehouse. Photo by Joan Goodwin. 10/26/89

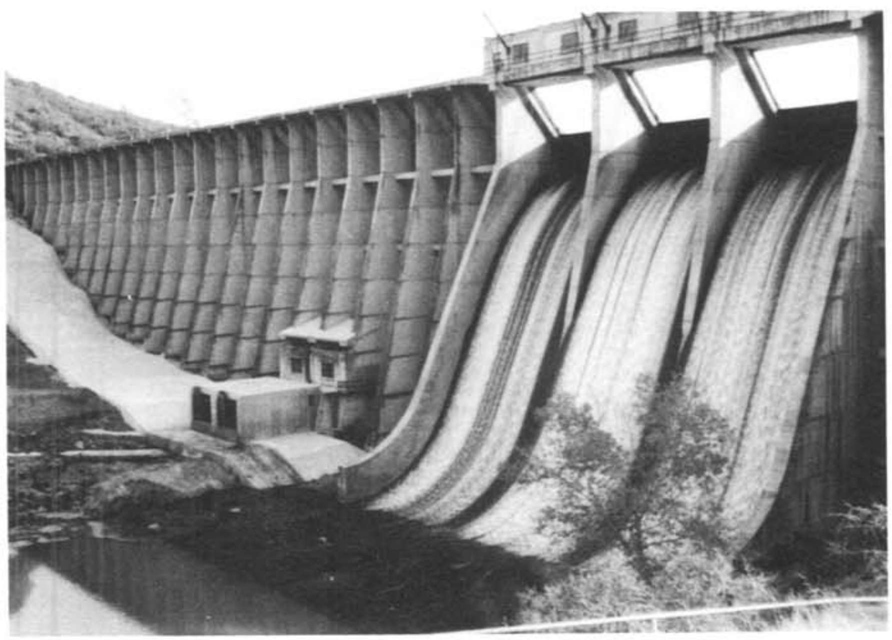

Figure 2. – Downstream view of Stony Gorge Dam showing the overtopping protection at the base of the dam. Photo by Joan Goodwin. 10/26/89

Figure 3. - City of Santa Clara powerplant immediately downstream of Stony Gorge Dam. Photo by Bill Bouley. 10/26/89

Figure 4. - Discharge from right fixed-cone valve flowing from the energy dissipation chamber. Photo by Bill Bouley. 10/26/89.

Mission of the Bureau of Reclamation

The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.

The purpose of this Bulletin is to serve as a medium of exchanging operation and maintenance information. Its success depends upon your help in obtaining and submitting new and useful O&M ideas.

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Prospective material should be submitted through your Bureau of Reclamation Regional office.