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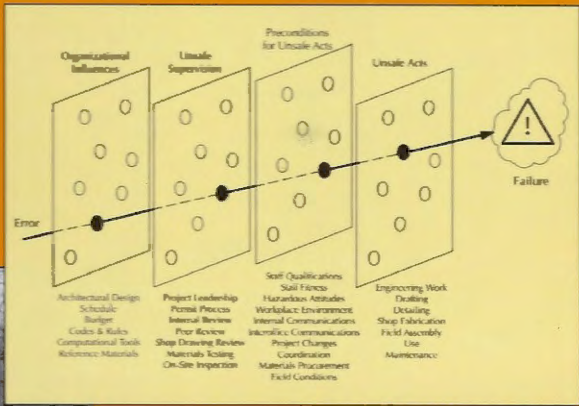

Human Error in Structural Engineering

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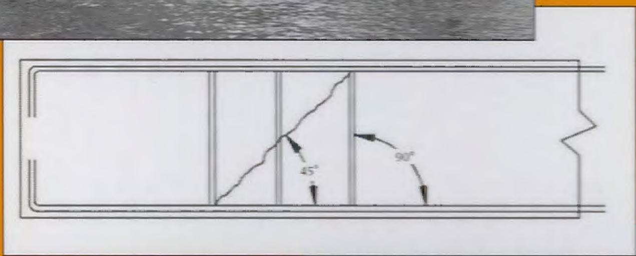

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A Change of Guard

It is now a little over seven years since I took over as chair of the editorial board of the *Civil Engineering Practice*. Serving as chair has been a great opportunity for me to work with a group of enthusiastic people who are dedicated to our journal and its objectives — *i.e.*, publishing relevant and practice-oriented articles covering civil engineering issues in general and significant civil engineering projects in and around the Boston area in particular. It is now time for a change in leadership mainly because I believe that such a change every few years will inject new perspective and new energy into this position in order to keep the journal vibrant, evolving and forward-looking.

I have been fortunate that during these years I had the support of many entities and individuals within the engineering community in Boston. BSCES leadership has been extremely supportive of *Civil Engineering Practice* and our mission. This support is especially significant because it requires a significant outlay of resources to publish the journal, even taking into account all the volunteer work that goes into it. I feel that one of the major accomplishments of my tenure as editorial board chair has been to keep the costs of the journal as low as possible and still maintain a high-quality publication. To this end we have embarked on two projects (in different stages of completion) in order to keep a lid on costs, to maintain our standard for high-quality content and to adapt to changing technologies: First, we have created a digital archive of all of the articles published by *Civil Engineering Practice* since its reincarnation in 1986, and we are now studying the best ways to present this digital archive for the benefit of our members and the engineering community as a whole. The other project is to research how to move *Civil Engineering Practice* more fully into the digital age and how we can offer the journal electronically to membership and the world.

Several members of the editorial board have been instrumental in soliciting articles, reviewing these articles and providing guidance for the future of the journal. While I am grateful to all the journal editorial board members for their contributions I would like to especially thank Brian Brenner, chair of our Editorial Subcommittee; Eduardo Gamez, chair of our Advertising Subcommittee; and Anni Autio and John Gaythwaite, members of the editorial board. Last but not least, we would not have *Civil Engineering Practice* in its present form if not for the efforts of Gian Lombardo, the journal's editor. I cannot thank Gian enough for his dedication and hard work.

Based on my recommendation and the support of the editorial board, James Lambrecht is taking over as the chair of the editorial board effective the fall of 2008. Jim is a geotechnical engineer with many years of experience working on large projects, formerly with the firm of Haley and Aldrich and now as a full-time faculty member with the Wentworth Institute of Technology.

He combines industrial experience with academic rigor and this combination makes him ideal for leading *Civil Engineering Practice* in the upcoming years. We are fortunate that Jim has accepted our invitation and we all look forward to working with him.

Ali Touran

Ali Touran
Chair, *Civil Engineering Practice* Editorial Board

An Innovative Rehabilitation Project at the Cobble Mountain Dam Outlet Works

An innovative plugging system provided cost savings and reduced risk on a rehabilitation project for a diversion tunnel high-pressure outlet works facility for a city water supply system.

NEILL J. HAMPTON & JAMES CONSTANTINO

Construction of Springfield Water and Sewer Commission's (SWSC) Cobble Mountain Reservoir Dam, located in Granville, Massachusetts, began in 1929 and was completed in 1931. Since then, as part of the Little River Water Supply System, the reservoir has served as the primary drinking water source for the City of Springfield, Massachusetts, and its surrounding communities. In 2001, a rehabilitation project was planned for the diversion tunnel high-pressure outlet works facility, which included the replacement of two existing water-operated 42- by 30-inch Larner-Johnson needle valves. The project also included the rehabilitation of the two existing 40-inch rotary (ball) style guard valves.

The earthen dam is 240 feet high, retains 22.5 billion gallons of water and is one of only a handful of dams constructed using the hydraulic fill method in the United States (see Figures 1 & 2). The Little River system has the capability of delivering 100 million gallons per day (mgd) of finished water to Springfield and its outer lying communities, serving a total population of approximately 250,000. Raw water is conveyed to the West Parish Filters Water Treatment Plant from Cobble Mountain Reservoir and the raw water conveyance system is comprised of the following key facilities:

- Broome gate house, power tunnel, surge tank and hydroelectric facility
- Diversion tunnel, gate house (see Figures 3, 4 & 5) and outlet works (see Figures 6 & 7).

The Broome gate house and power tunnel are

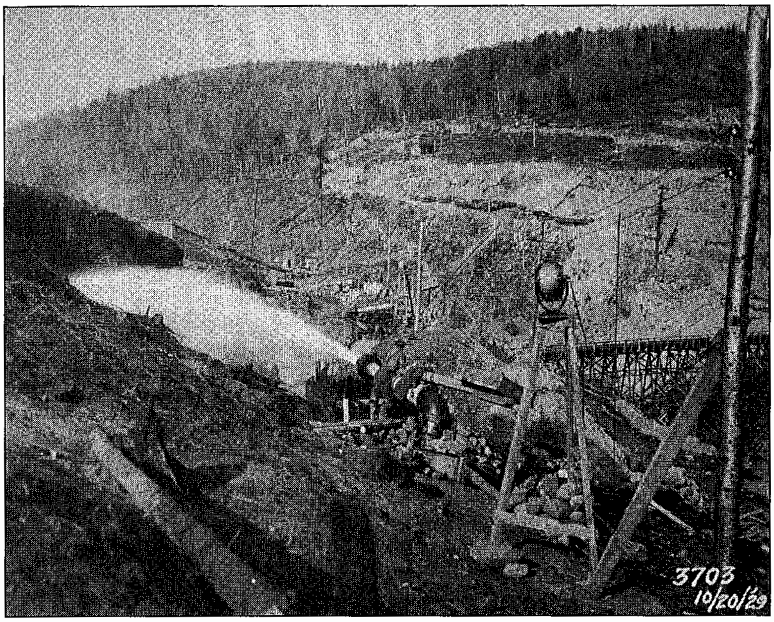


FIGURE 1. Mining giants and grizzlies operating at Cobble Mountain during sluicing operations in 1929.

the primary conveyance structures used to transfer raw water from Cobble Mountain Reservoir to the West Parish Filters Water Treatment Plant. During normal operations, raw water flows from the Broome gate house through the power tunnel to the hydroelectric facility, which contributes up to 33 kilowatts (kW) to the local grid. The tailrace from the hydroelectric facility discharges to the Little River, which flows to the Intake Reservoir. The Intake Reservoir outlet works conveys raw water to the water treatment plant.

The Cobble Mountain diversion tunnel was originally constructed during the late 1920s to divert the Little River flow around the dam construction site. The tunnel was blasted through solid rock at the base of Cobble Mountain, and has an 11.5-foot horseshoe shaped cross-section and concrete lining (see Figures 3 & 4). Upon completion of the dam, a shaft was bored 233 feet down from the dam access road to the tunnel below and a high-pressure outlet works facility was constructed approximately 350 feet into the tunnel from the reservoir (see Figure 6).

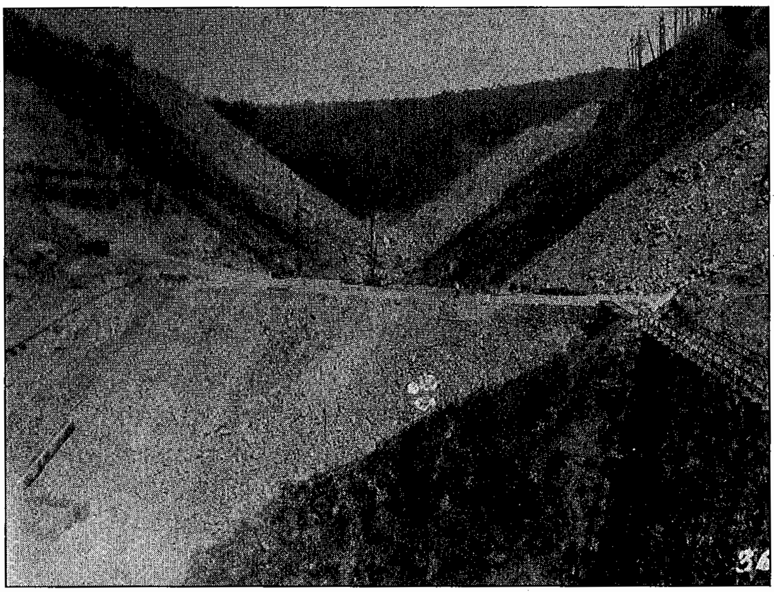


FIGURE 2. Cobble Mountain Dam at about half completion in 1929.

The outlet works includes a concrete bifurcation and twin 42- by 40-inch diameter, 40-foot-long, riveted steel-lined outlet conduits terminating in a gate chamber. A control chamber was constructed above the gate chamber where operators could safely operate the outlet valves.

Flow through the outlet works is regulated (throttled) through two 42- by 30-inch Larner-Johnson differential needle valves. The needle valves free-discharge into the diversion tunnel and direct raw water to the water treatment plant via the Little

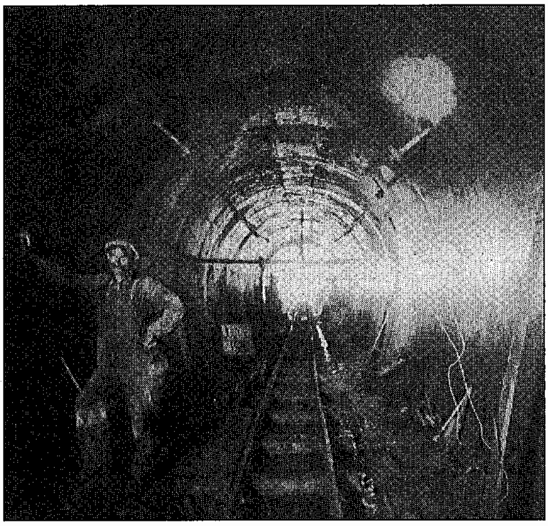


FIGURE 3. The diversion tunnel under construction.

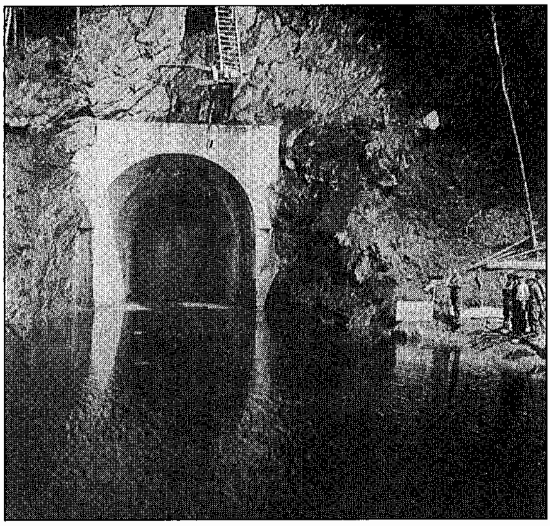


FIGURE 4. The diversion tunnel entrance portal.

River during maintenance outages at the hydroelectric facility. The outlet works' original discharge capacity was 480 mgd with both needle valves fully open and the reservoir water surface at the spillway crest elevation.

Each differential needle valve was isolated from the reservoir when not in use by 40-inch diameter rotary valves (see Figures 10 & 11). The diversion valves were operated from a control chamber constructed above the gate chamber.

Access to the outlet works facility is gained via the Diversion Gate House (see Figures 5 & 6). A forty-flight stairway was constructed within the access shaft from the gate house to the control chamber.

During the years after commissioning, the Larner-Johnson needle valves never functioned properly, and were plagued with mechanical problems. In the 1960s, the needle valves had a major mechanical overhaul (see Figure 12) but this and other attempts to improve reliability were met with limited success. This situation would not change from the 1960s through the 1990s (see Figure 13).

The diversion tunnel outlet works is the secondary raw water conveyance structure from Cobble Mountain Reservoir, so it was critical that the needle valves be fully operational and reliable. In addition, by the early 1990s the power tunnel, surge tank and hydroelectric facility were due for a comprehensive maintenance upgrade, which would require an extended shutdown. In the early 1990s, SWSC included an upgrade to the diversion tunnel facility in its long-term capital improvement plan. However, they had a surprise coming that would place diversion tunnel valve maintenance to the top of its priority list.

The $500,000 4-Inch Gate Valve

In the late 1980s, vandals broke into the

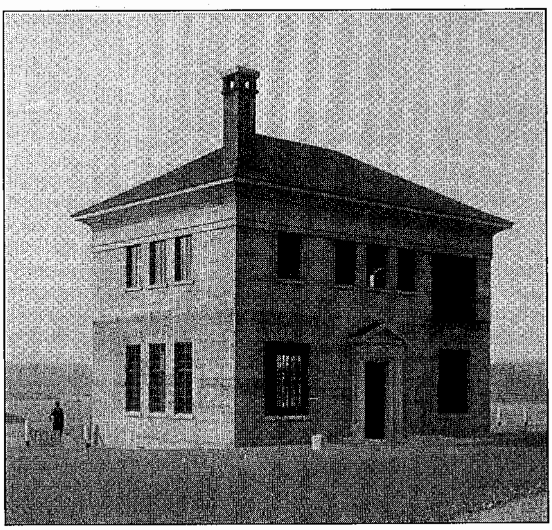


FIGURE 5. The Cobble Mountain Diversion Gate House (1931).

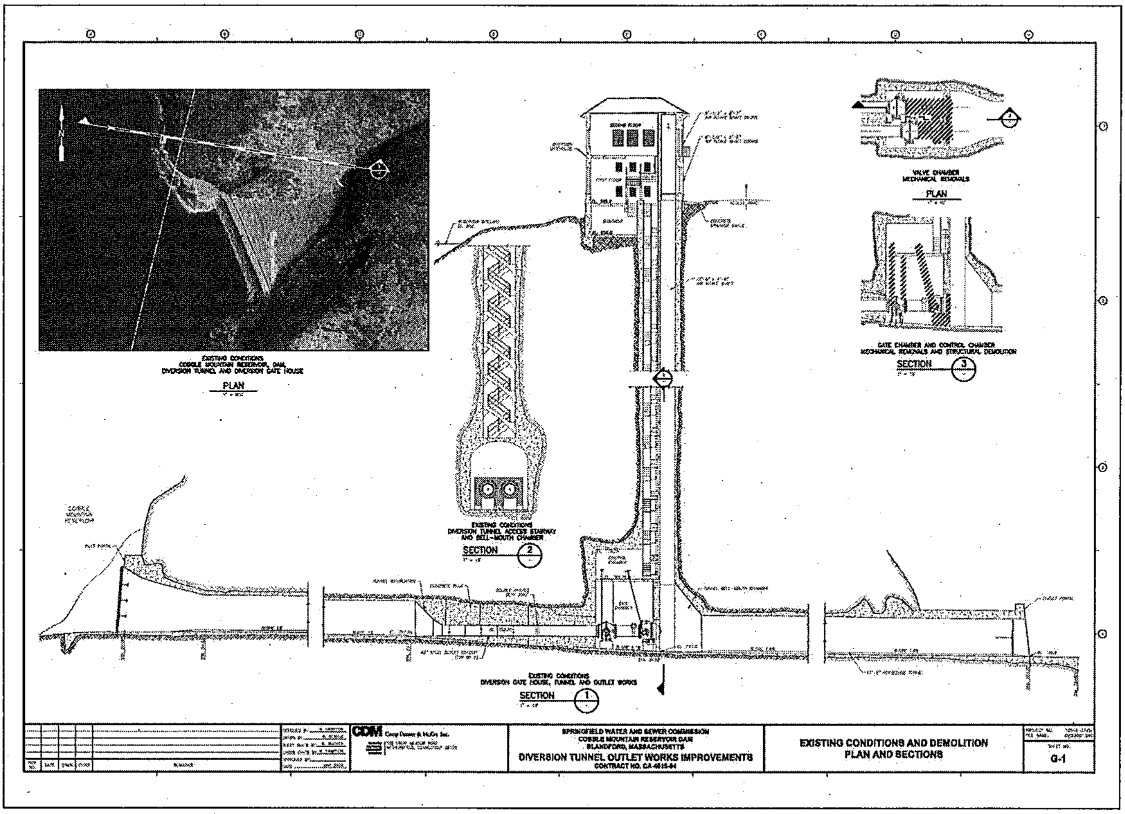


FIGURE 6. The diversion tunnel, gate chamber, access shaft and gate house.

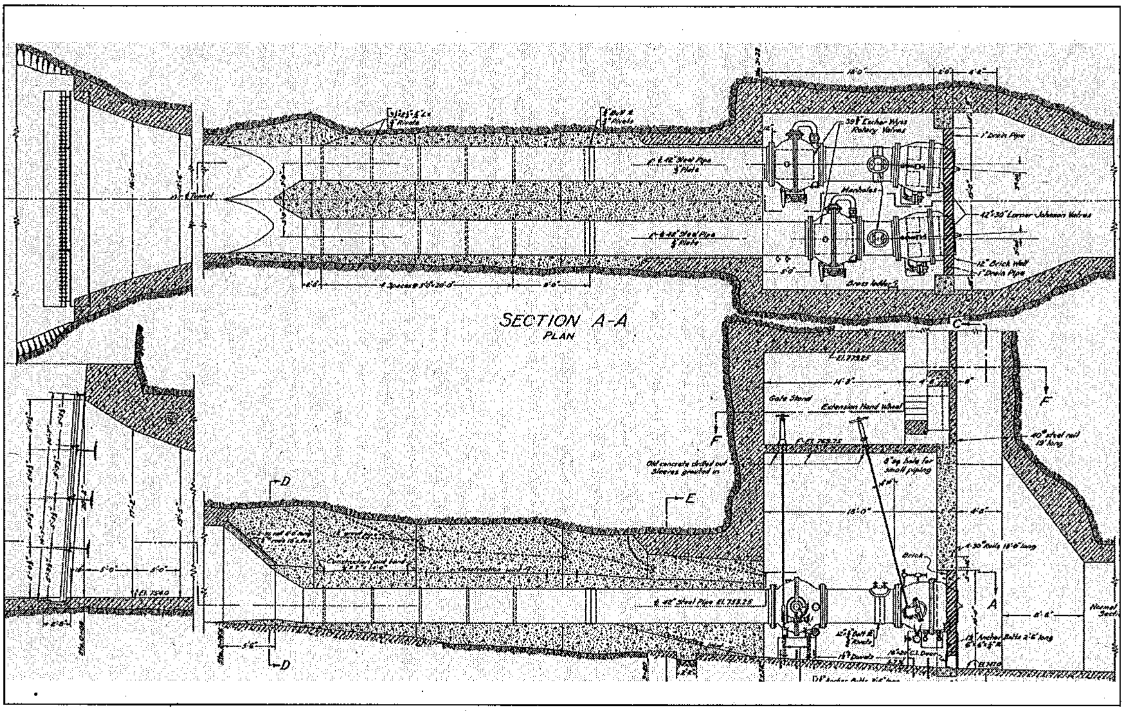


FIGURE 7. The diversion tunnel outlet works.

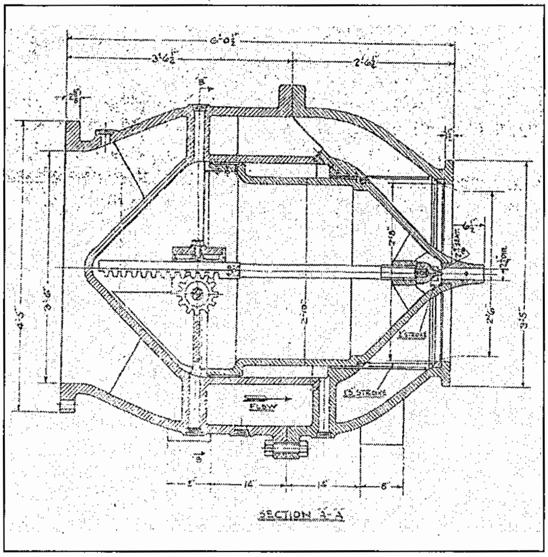


FIGURE 8. Needle valve cross-section.

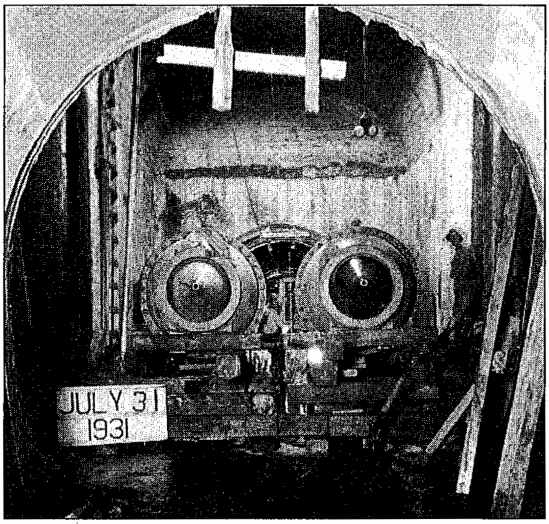


FIGURE 9. A view upstream in the diversion tunnel showing the needle valves installed in the gate chamber.

diversion tunnel gate chamber, and left the damaged access door open after their departure. The opened door allowed cold winter air to blow up the tunnel, causing a rupture in a 4-inch diameter auxiliary blow-off valve attached to rotary valve no. 2. The rupture was due to frozen reservoir water confined in the cast iron valve body. The valve sprang a major leak. In the original facility design, there was no provision for an isolation gate, bulkhead or stop-logs at the tunnel entrance.

The rotary valves are under the full reservoir head of 200 feet at all times and there is no way to drain and depressurize the valves for disassembly. The leak could not be stopped under dry conditions. This predicament turned a simple 4-inch valve replacement job into a much larger and more dan-

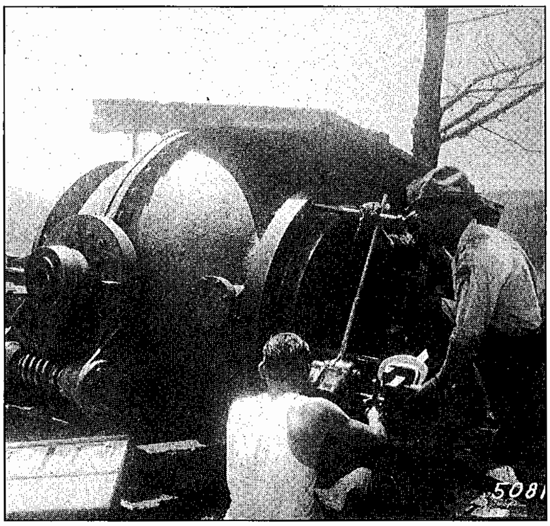


FIGURE 10. The rotary valves that were shipped from Zurich, Switzerland, to the Cobble Mountain site in 1931.

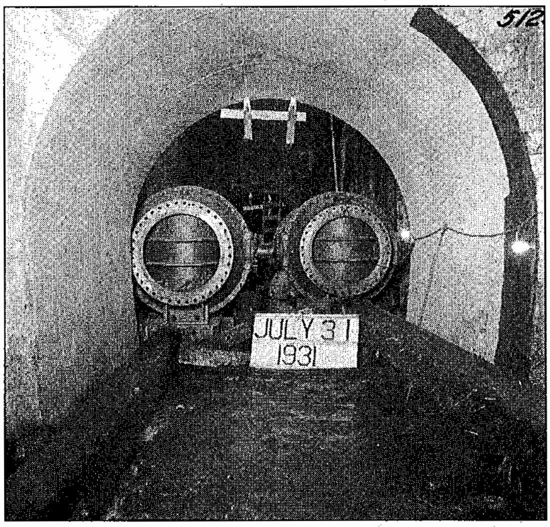


FIGURE 11. A view downstream in the diversion tunnel showing the rotary valves installed in the gate chamber.

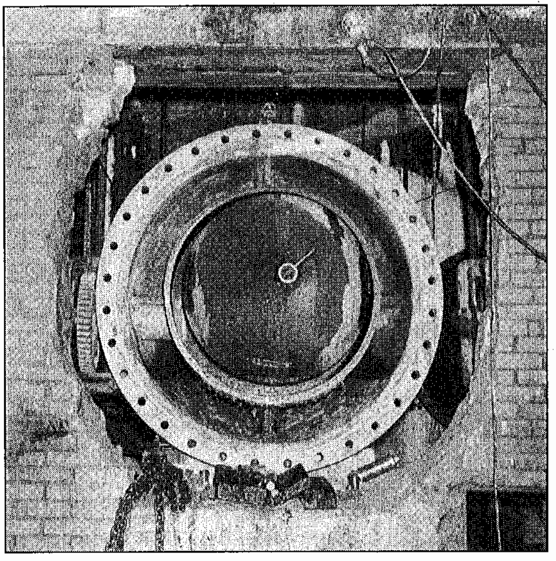


FIGURE 12. A view of needle valve no. 2 showing internals during a mechanical overhaul in the 1960s.

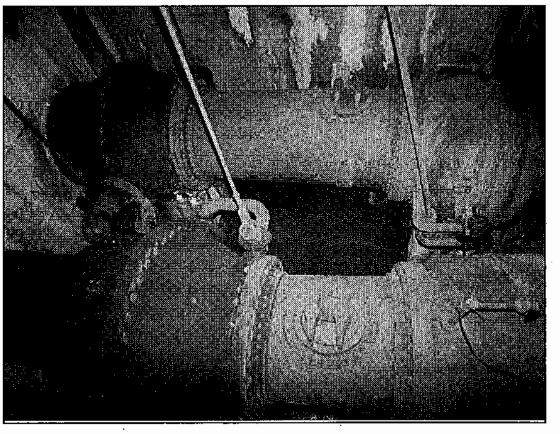


FIGURE 13. The diversion tunnel entrance portal.

gerous project. After considering several options, the SWSC preferred a plan for the replacement of the valve involving the following:

- Construction of a concrete bulkhead in the bell-mouth of the diversion tunnel, downstream of the gate chamber;
- Filling the gate chamber, control chamber, stairway and air shaft with reservoir water to balance the water pressure between the reservoir and diversion facility; and
- Replacement of the valve under submerged conditions by commercial divers.

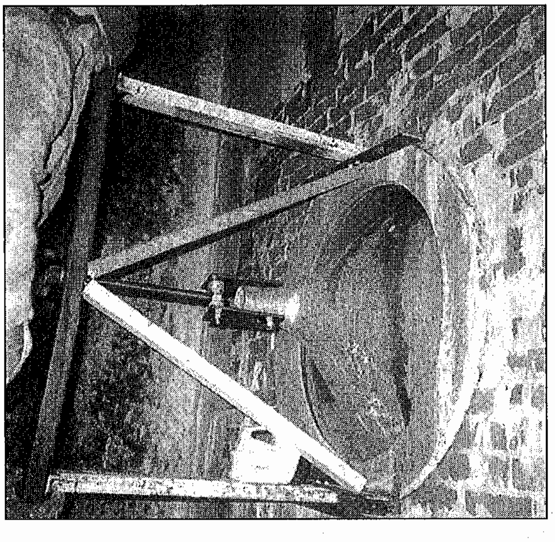


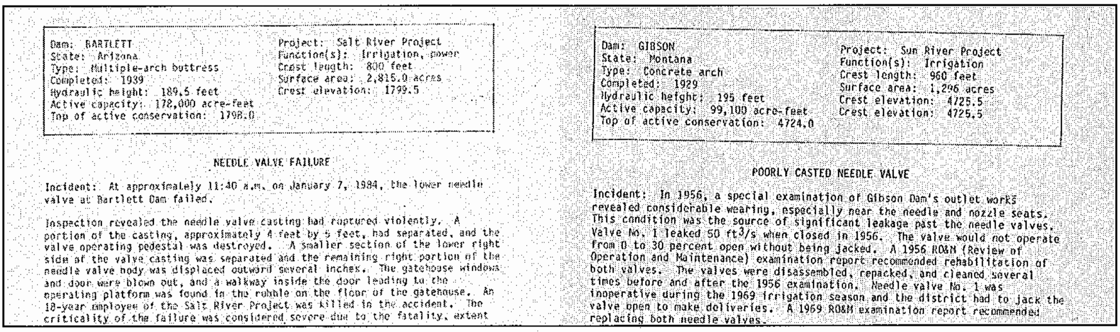
FIGURE 14. Closing the jammed needle valve.

This alternative was extremely dangerous and expensive (its cost was approximately $500,000). However, the SWSC had no other viable choice at the time, and could not avoid removal and replacement of the leaking valve.

Work began in early 1990. The concrete bulkhead was constructed, the facility was flooded and the divers were able to access the valves in the chamber. For every hour of work time in the chamber, each diver was required to decompress for seven hours to prevent the formation of nitrogen bubbles in their blood (the bends).

The cracked blow-off valve was removed, but the new valve purchased by the contractor could not be used as a replacement valve because the flange pattern on the Swiss-made Escher Wyss valve was drilled to British Standards. Given the circumstances, SWSC decided to remove the blow-off valves from both rotary valves, and replace them with custom fabricated blind flanges.

After installation of the blind flanges, the facility was drained and the concrete bulkhead was removed using a wire saw. This 4-inch gate valve problem cost SWSC more than $500,000, was an extremely risky operation



Dam: BARTLETT
State: Arizona
Type: Multiple-arch buttress
Completed: 1939
Hydraulic height: 189.5 feet
Active capacity: 178,000 acre-feet
Top of active conservation: 1798.0

Project: Salt River Project
Function(s): Irrigation, power
Crest length: 800 feet
Surface area: 2,815.0 acres
Crest elevation: 1799.5

NEEDLE VALVE FAILURE

Incident: At approximately 11:40 a.m. on January 7, 1984, the lower needle valve at Bartlett Dam failed.

Inspection revealed the needle valve casting had ruptured violently. A portion of the casting, approximately 4 feet by 5 feet, had separated, and the valve operating pedestal was destroyed. A smaller section of the lower right side of the valve casting was separated and the remaining right portion of the needle valve body was displaced outward several inches. The gatehouse windows and door were blown out, and a walkway inside the door leading to the operating platform was found in the rubble on the floor of the gatehouse. An 18-year employee of the Salt River Project was killed in the accident. The criticality of the failure was considered severe due to the fatality, extent

Dam: GIBSON
State: Montana
Type: Concrete arch
Completed: 1929
Hydraulic height: 195 feet
Active capacity: 99,100 acre-feet
Top of active conservation: 4724.0

Project: Sun River Project
Function(s): Irrigation
Crest length: 960 feet
Surface area: 1,296 acres
Crest elevation: 4725.5
Crest elevation: 4725.5

POORLY CASTED NEEDLE VALVE

Incident: In 1956, a special examination of Gibson Dam's outlet works revealed considerable wearing, especially near the needle and nozzle seats. This condition was the source of significant leakage past the needle valves. Valve No. 1 leaked 50 ft^3/s when closed in 1956. The valve would not operate from 0 to 30 percent open without being jacked. A 1956 RO&M (Review of Operation and Maintenance) examination report recommended rehabilitation of both valves. The valves were disassembled, repacked, and cleaned several times before and after the 1956 examination. Needle valve No. 1 was inoperative during the 1969 irrigation season and the district had to jack the valve open to make deliveries. A 1969 RO&M examination report recommended replacing both needle valves.

FIGURE 15. USBR reports of needle valve failures on the Salt River Project in Arizona.

and the contractor was not able to actually replace the blow-off valves due to the unusual mechanical design of the rotary valves and construction constraints within the facility. The incident also exemplifies the hazards and high expense associated with construction at this facility, and provided a preview of the challenges that would be faced on the future rehabilitation project.

In 2001, SWSC consulted with an engineering consultant to provide technical assistance in its efforts to rehabilitate and make operational and reliable the valves and appurtenances in the diversion tunnel gate chamber. Unfortunately, soon after the investigation began and an attempt was made to operate one of the needle valves, the needle became jammed in the open position. Millwrights were hired to free the needle and close the valve (see Figure 14). Soon after the initial inspection, it was clear to SWSC and the engineering consultant that the facility needed a major overhaul. During the previous years, the rotary and needle valves had fallen further into disrepair and required mechanical improvements or replacement.

The Needle Valve Problem

Free-discharge regulating valves and gates for high-pressure outlets at dams have seen many developmental variations during the past century with mixed success. Free-discharge end-of-the-line valves accomplish two difficult tasks:

- Throttling high flows; and,
- Energy dissipation, without damage from hydrodynamic cavitation.

Needle valves were originally developed by the United States Bureau of Reclamation (USBR) to regulate irrigation and stream maintenance flows from dams in the western United States. The Larner-Johnson valve uses the differential pressure between an inner pilot valve and the main valve flow passage to hydraulically operate the spider-mounted plunger style "needle" that throttles the flow through the main valve body. This type of valve became commercially available through a patented design by the I.P. Morris Company in the early 1900s. Their installations included 21-foot diameter valves installed at Niagara Falls, New York. The needle valves at the diversion tunnel outlet works had many mechanical malfunctions over the years. The valve on outlet no. 1 was completely inoperable.

After some research by the engineering consultant, in collaboration with specialists at the USBR, several incidents at dams around the country were identified where needle valves had catastrophic failures, causing the deaths of several workers and millions of dollars in damages to hydraulic structures (see Figure 15). The USBR has carried out a needle valve replacement program during the past two decades to mitigate the potential for future failures.

Based on these facts and the history of mechanical problems, the engineering consultant recommended complete replacement of the needle valves. The goals for replacement included selecting a new throttling valve that could meet or exceed the discharge capacity of the existing system (480 mgd), fit into the gate chamber without dimensional con-

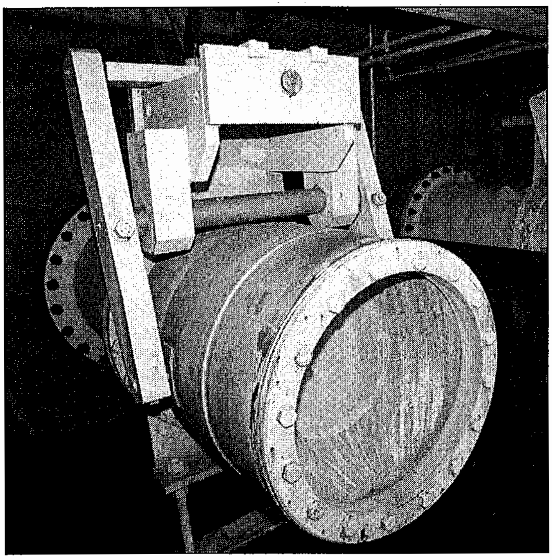


FIGURE 16. A fixed cone valve.

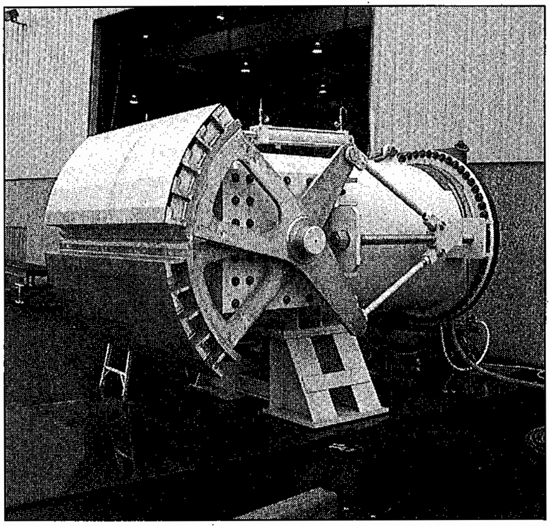


FIGURE 17. A clam shell gate.

flicts or security concerns and provide greater reliability for a project life of eighty years or more. Several replacement valve alternatives were considered including:

- Fixed Cone Valves (see Figure 16). These valves were originally developed by the USBR for use in free-discharge applications at high head dams. There are hundreds of installations of these valves at dams across the United States and abroad. During the past fifty years, the design had several mechanical problems, but they have been worked out of the models that are commercially available today. This valve discharges at a wide angle and would require an energy dissipating hood to direct flow into the diversion tunnel. This aspect of the valve's construction did not make it suitable for installation in the gate chamber. In addition, there were dimensional conflicts that prevented their installation in the diversion tunnel bell-mouth chamber. These valves would not be a good replacement for the needle valves and were not recommended.
- Clam Shell Gates (see Figure 17). These gates are a recent development by the USBR. There are a lot of moving parts on this type of gate, and there were dimensional conflicts in the gate chamber. Based on these facts, and its high price, this gate was not selected to replace the needle valves.
- Jet-Flow Gates (see Figure 18). These gates were originally developed by the USBR for installation at Shasta Dam in the early 1930s. They have gone through several design modifications over the years and the service records for most installations have been very good. The jet-flow gate has the ability to discharge large flows at heads up to 300 feet with smooth operation and very little (if any) vibration. A 30-inch diameter jet-flow gate would increase the capacity of each outlet from 240 to 300 mgd. The 30-inch gate has a short laying length, which had a good fit dimensionally in the gate chamber. Based on the long successful track record of this type of gate, and the project specific goals that were met, the engineering consultant recommended installation of 30-inch diameter jet-flow gates at Cobble Mountain.

Jet-Flow Gates

There are commercially available jet-flow gates from several valve and gate manufacturers. The engineering consultant and SWSC selected to include the design of the gate in the contract drawings. The twin 30-inch gate design was taken from a USBR design used successfully on many projects.

Each gate body is fabricated from A36 steel. The gate leaf is fabricated of Type 304L stainless steel. The gate is operated by a center-mounted, extended shaft 10-inch diameter hydraulic cylinder. Two dedicated 1,500 pounds per square inch (psi) hydraulic power units (HPU) were installed in the control chamber and featured triple redundancy. Two hydraulic pumps would be furnished with each HPU. If both pumps are operated, the gate will rise at 12 inches per minute. If one pump is out of service, the gate will rise using the other pump at 6 inches per minute. If both hydraulic pumps are out of service, the gate can be lifted manually using a hand pump. If the hand pump and the hydraulic pumps malfunction, the dedicated HPU for the other gate can be used to raise both gates. The gate throttles flow through a 30-inch diameter aluminum bronze conical nozzle. In the full open position, the gate leaf is raised 6 inches above the discharge jet.

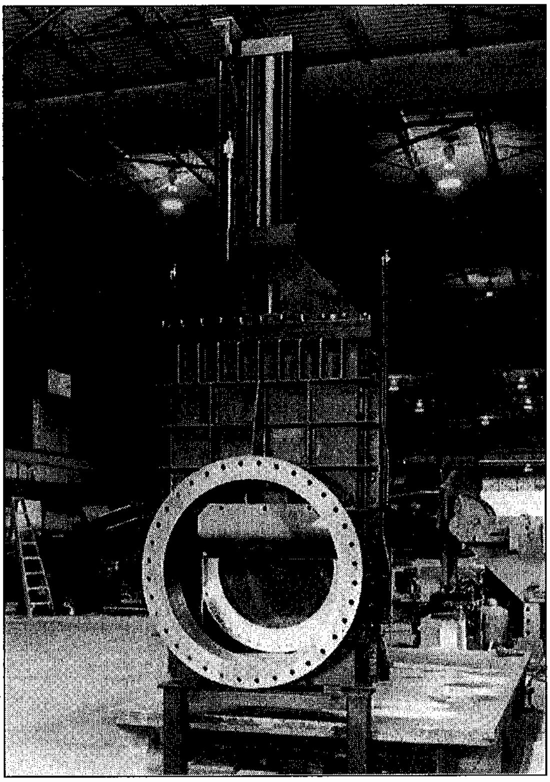


FIGURE 18. A jet-flow gate.

Rotary Valve Rehabilitation

The Cobble Mountain rotary valves were one of the first commercially available large-diameter rotary (ball) valves. Manufactured in the early 1900s, the rotary valve has many design features that are similar to modern AWWA ball valves. The most important of these features is a drip-tight shut-off for isolation of the throttling valves. These valves also exhibit exceptional hydraulic characteristics, which allow the valves to close safely if the throttling valve malfunctions.

The valve has three main components: a trunion-mounted rotating plug, and front and rear body castings. The body castings are steel, which was uncommon at that time. The standard valve model was furnished in cast iron, but the design engineers insisted that cast steel valves be furnished for Cobble Mountain. A bronze seat ring is mounted on the rotating ball, which seals against a bronze stationary seat mounted on the downstream nozzle of the valve. The seats are engaged utilizing the upstream reservoir pressure. The plug seat must be lifted prior to opening the main valve by balancing the pressure between the downstream (dry) side, and upstream side of the seat ring (accomplished by an auxiliary bypass valve and piping mounted on the side of the main valve body — see Figure 19).

The rotary valves cannot be isolated from the reservoir and remain under full reservoir pressure at all times. Isolation of the valves could only be accomplished by plugging the upstream conduit. Upstream plugging would add significant risk, complexity, cost and time to the project. There were no viable alternatives for complete replacement of the rotary valves within the allocated project budget, which was $1.5 million. Rehabilitation of the existing rotary valves was the only viable and cost-effective alternative, and it posed many design challenges.

The main valve body had considerable surface corrosion that would have to be examined to determine the extent of material loss. If the main valve body could be certified as fit for service, the only obstacle to its rehabilitation would be isolation of the main valve

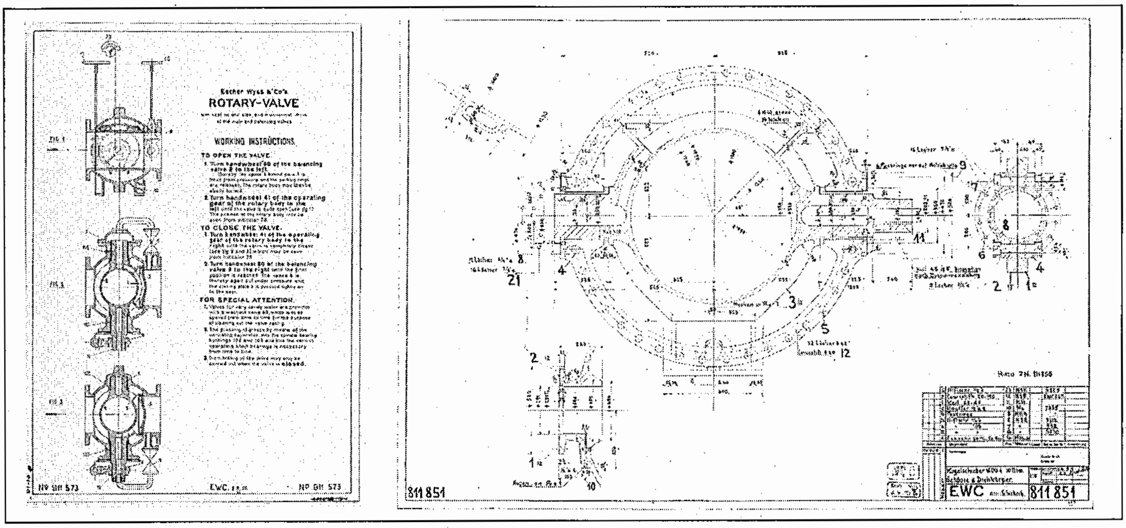


FIGURE 19. Excerpts from the original rotary valve shop drawings (1929).

from the reservoir for disassembly of the auxiliary components in the dry. The engineering consultant and a sub-consultant conducted wet magnetic particle and ultrasonic thickness testing on a 2-inch grid across both halves of the valve body (see Figure 20). Based on the results of this survey, it was determined that the valve body castings were fit for service, and would likely exceed the design life of eighty years.

FIGURE 20. Wet magnetic particle inspection of the steel rotary valve body castings.

The existing auxiliary bypass valves had severe corrosion and material loss requiring complete replacement (see Figure 21). The challenge was the location of the valve, which was always under system pressure and prevented removal of the valve in the dry because the upstream piping was under full reservoir pressure and could not be isolated. The bypass valves were 5 inches in diameter and had British Standard (BS) flanges installed on double-ended studs with British Standard Whitworth (BSW) threads. Ultrasonic thickness testing indicated a bypass pipe wall thickness of less than 0.125 inch. The pipe wall had deteriorated to a point where replacement was the only option.

The bypass piping would be replaced with TIG-welded Type 304L stainless steel pipe. The 5-inch bypass valves would be replaced by 4-inch CF-8 stainless steel gate valves. The existing BS flanges were thinner than modern ANSI flanges, so the existing flange studs would be

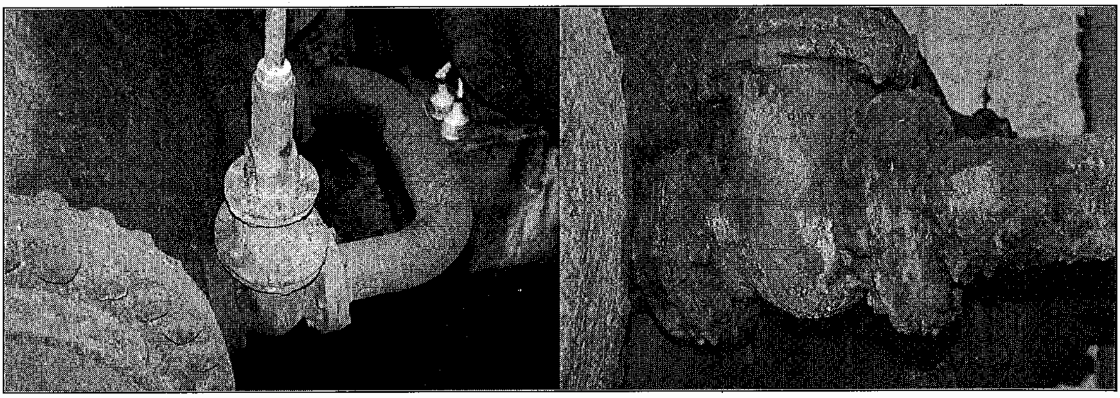


FIGURE 21. Rotary valve 5-inch auxiliary bypass valves showing extensive corrosion.

replaced with custom-designed 304 stainless steel double-ended studs having 55-degree BSW on one end, and 60-degree unified coarse threads on the other. The 4-inch diameter blow-off valves, originally installed on flange bosses located at the bottom of the rotary valves were removed in the early 1990s and replaced with blind flanges (see Figure 22). These valves are essential for flushing rust, silt and rock fragments that are deposited in the main valve body after operation. These valves would have to be replaced.

The existing drive-side shaft seal was leaking on both valves (see Figure 23). Since the valves were under pressure, there was no safe way of disassembling the valve to inspect the existing seal and stuffing box and determine the best replacement alternative.

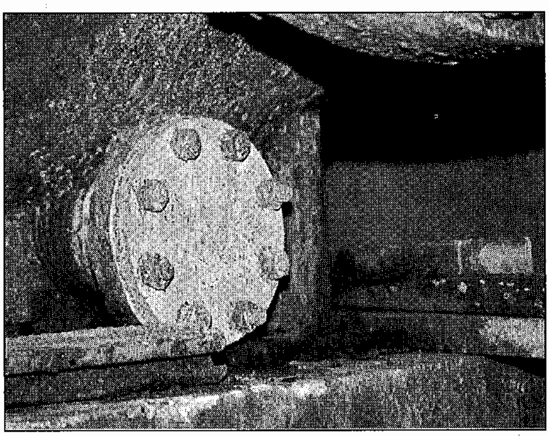


FIGURE 22. The rotary valve's 4-inch auxiliary body flush valve installation location showing the temporary blind flange.

This work would have to be included as a change order during construction after the valves were isolated from the reservoir and drained. A custom-designed shaft seal would be required. In general, the worm gear drive was in good condition and would only require cleaning, painting and replacement of some thrust bearings. The valves are operated by hand-wheels located on floor stands in the control chamber. These components and the intermediate piping between the rotary valves and needle valves would all be replaced.

The valve's manufacturer in Switzerland was able to furnish the original 1929 shop drawings. However, the drawing text was

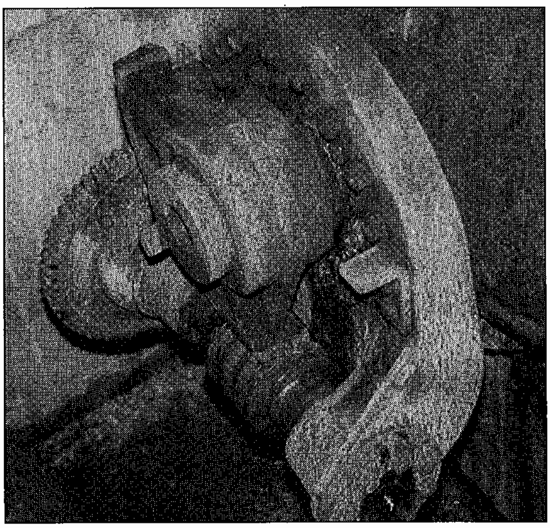


FIGURE 23. The rotary valve's worm gear drive showing the leaking shaft seal.

FIGURE 24. A typical wet-tap installation.

written in German and required translation. These documents would be extremely important in the design of the seal and other improvements, but the original manufacturer did not want to participate in the project beyond this assistance.

The Plug

A clear strategy for the rehabilitation of the rotary valves had been developed, but the problem of isolating the rotary valves for rehabilitation had not been solved. A plan was developed to install a bulkhead plug in the 40-inch diameter steel-lined conduit directly upstream of the rotary valves. Initially, the use of commercial divers to install the plug was explored and found to be a viable alternative. Unfortunately, this alternative was cost prohibitive at an estimated construction cost of $3 million, nearly twice the allocated budget for the project. A method of plugging the outlet from the gate chamber in-the-dry, at a cost within the project budget allocation, would have to be developed.

Plugging live pipelines has been accomplished successfully in the water and gas industry during the past hundred years at pressures up to 500 psi. Non-entry plugging equipment developed for the pipeline construction industry were reviewed by the engineering consultant to determine if a commercially available device could be used at Cobble Mountain.

Line-stopping and plugging of pipelines while they are still in service is typically accomplished using a combination of wet tapping and line-stopping equipment, which is commercially available from several manufacturers. The procedure involves the following steps:

1. Install a line-stop fitting and tapping valve on the pipeline.

2. Mount a wet-tap drilling machine to the tapping valve, transverse

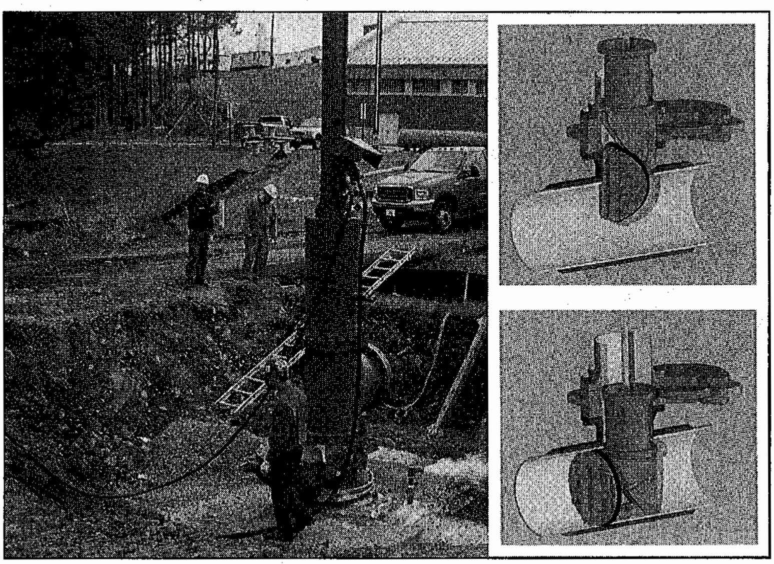


FIGURE 25. Line-stop installation, with an illustration of the internal mechanism.

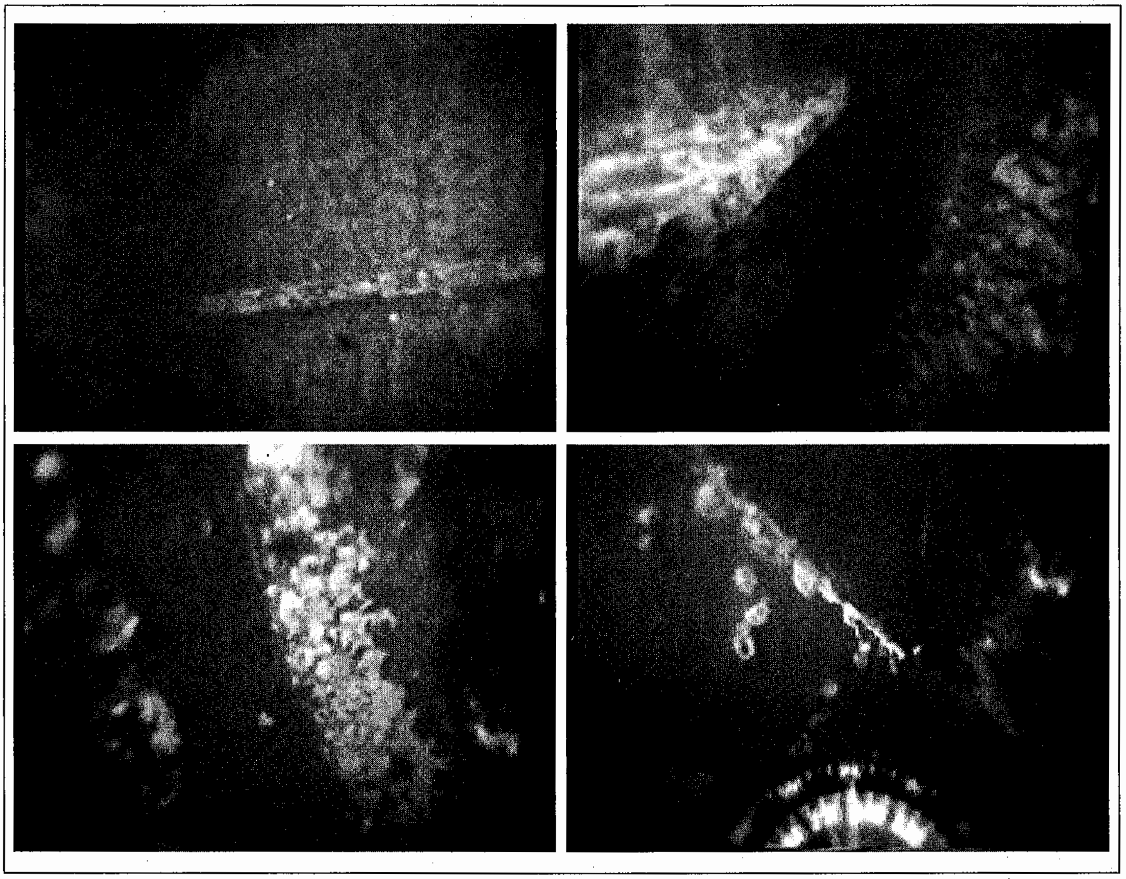


FIGURE 26. Video clips from a remotely operated submersible vehicle inspection: rotary valve ball (top left and right); steel conduit joints showing countersunk rivets at the flange (bottom left); and pipe seams (bottom right).

to the pipeline, and remove a coupon from the pipe wall (see Figure 24).

3. Remove the drilling machine and install a line-stop apparatus on the tapping valve.

4. Insert the line-stop through the tapping valve, into the pipeline and engage the plug seal (see Figure 25).

This procedure could not be used at Cobble Mountain because the upstream 42-inch steel conduit did not terminate far enough into the gate chamber to permit installation of the tapping sleeve. Also, even if it were possible to install the line-stop fitting, there would be serious safety and source water protection concerns associated with drilling into the side of the outlet conduit under 200 feet of head. If the line-stop fitting failed in any way, there would be no means of stopping the reservoir from draining.

After reviewing all of the commercially available technologies, discussing the problem with manufacturers and the USBR, it was clear to the design team that there was no device in existence that could accomplish the project goals. A new custom-designed device had to be developed that could deliver a plug through the rotary valve into the upstream conduit.

Before a plan could be developed, the design team had to find out if there were any tubercules on the conduit walls that would prevent a plug from being installed. The engineering consultant was able to obtain a video inspection, carried out using a remotely operated submersible vehicle, from the diving contractor who installed the blind flanges in the early 1990s (see Figure 26). This

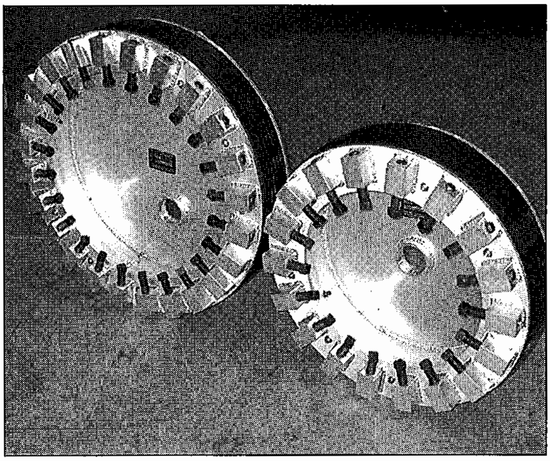


FIGURE 27. Fabricated aluminum pneumatic test plugs.

video inspection was clear enough to show very little corrosion to the valve plug and conduit wall; therefore, a plug could be installed.

The design team started with pneumatic bulkhead style pipeline plugs, which are used for pressure testing pipeline segments. These plugs are available in 40-inch diameter but would have to be specially designed by the plug manufacturer for the design pressure of 150 psi (see Figure 27).

The pneumatic plug would be custom built to the design requirements, but also needed a device capable of inserting the plug through the rotary valve. The engineering consultant developed a plug insertion device that could accomplish this task. The device is illustrated in Figures 28 through 35.

Figure 28 illustrates the device mounted to the closed rotary valve after the needle valves and intermediate piping are removed. The device would have a cylindrical housing that would retain the plug mounted on a shaft (see Figure 29). The shaft would be advanced through the valve and into the conduit by means of a threaded stem and stem nut actuated by a gear box and hand wheel.

To begin the insertion procedure, the existing bypass valve would be opened, and the pneumatic plug housing section filled with reservoir water to balance the pressure and raise the seat disc (see Figure 30). After the seat disc is raised, the main valve ball would be moved to the full-open position.

The design team confirmed from the 1920s shop drawings that the rotating valve ball had the same inside diameter as the connecting pipe, which would allow passage of the plug through the valve ball and into the conduit. After the plug was inserted and in position (see Figure 31), compressed gas would be used to inflate the elastomer seal ring on the periphery of the plug frame through a coiled

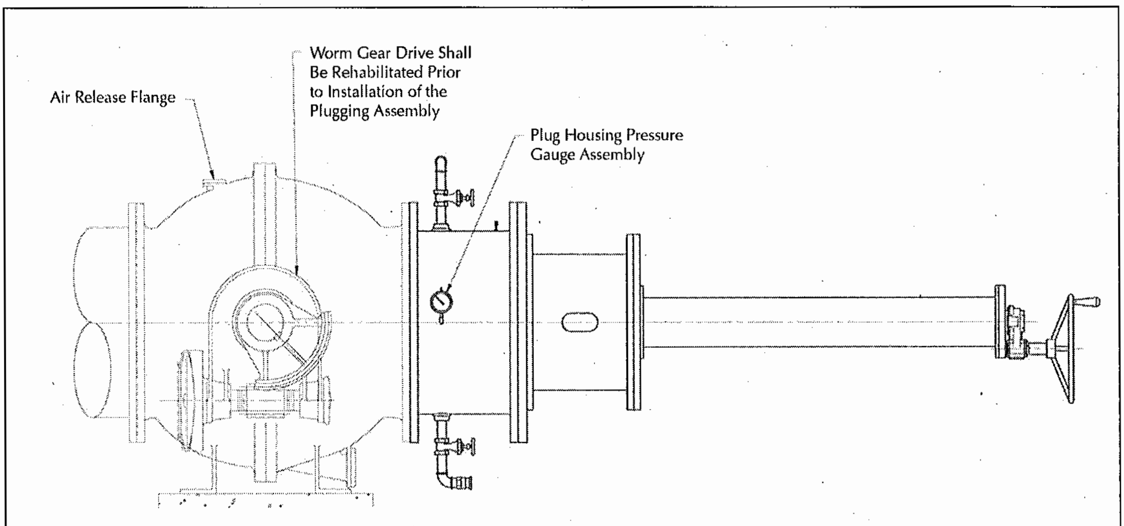


FIGURE 28. The insertion device mounted to the closed rotary valve.

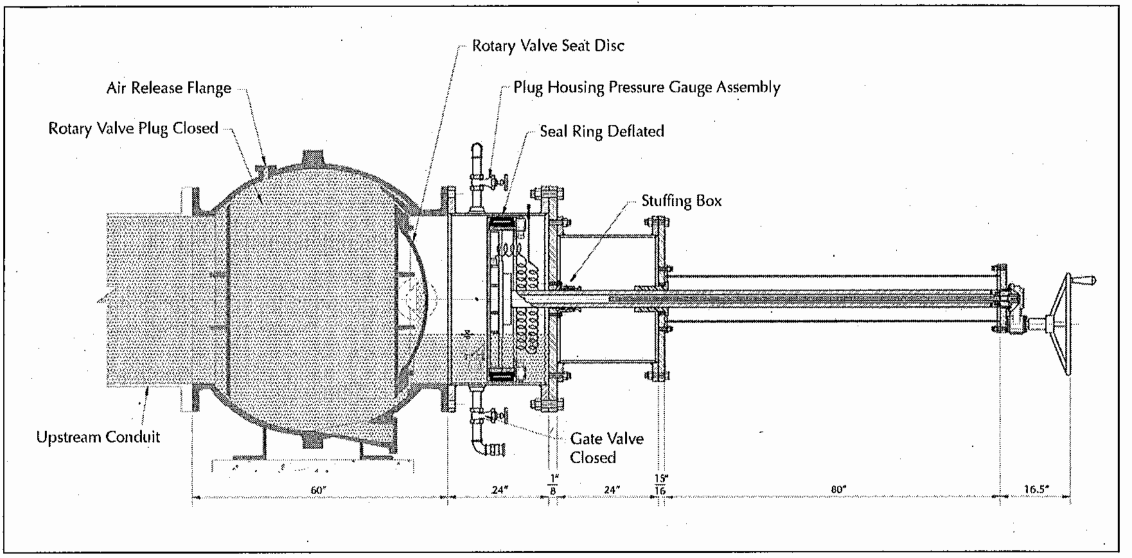


FIGURE 29. The device would have a cylindrical housing that would retain the plug mounted on a shaft.

retractable elastomer hose and bulkhead fitting. This step would provide the sealing mechanism for the system, but a thrust load of 150,000 pounds would be transferred to the shaft as the rotary valve is drained, so a separate means of mechanical restraint would be required. In the preliminary design, the insertion apparatus was removed and a blind flange was bolted to the stuffing box housing behind the shaft, restraining the plug and shaft after the valve is drained (see Figures 32, 33 & 34).

After the plug is installed, restrained and inflated, the rotary valve could then be drained and depressurized allowing safe disassembly of the auxiliary valve components (see Figure 35). The illustrations (Figures 28 to 35) were presented to engineering and operations staff at SWSC, and the system was approved for implementation. The engineering consultant worked with sub-contractors to determine the feasibility of fabricating the device, and to obtain an estimated cost. Based on a performance specification and illustrative

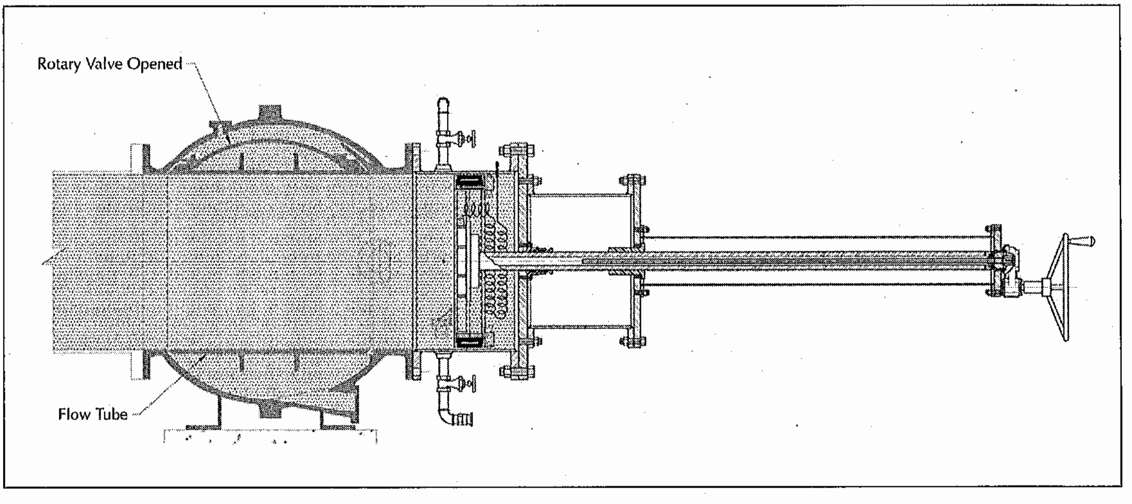


FIGURE 30. To begin the insertion procedure, the existing bypass valve would be opened.

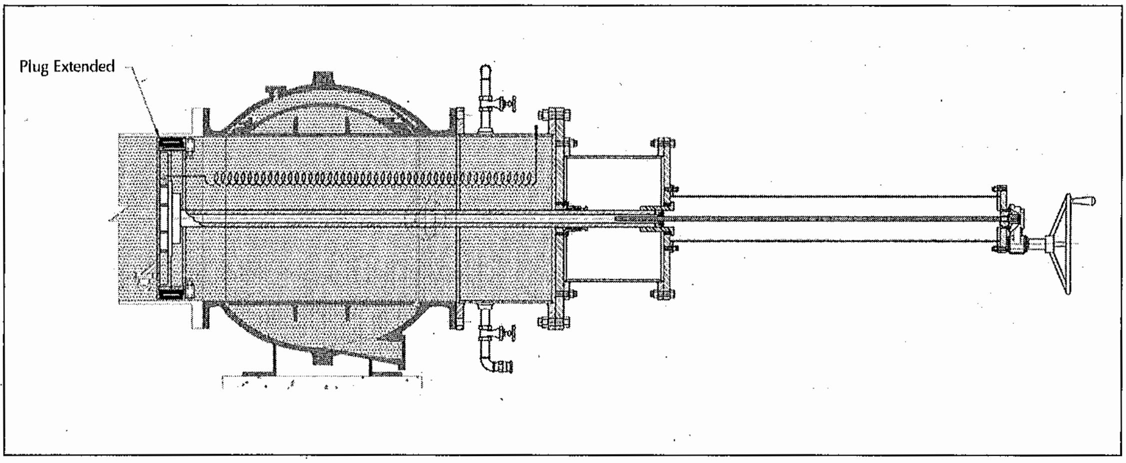


FIGURE 31. The plug would be inserted through the valve.

details of the insertion device included in the contract documents, it had an estimated cost of $100,000.

The Bulkhead

The non-entry plug concept was accepted by SWSC, but the Commissioners were concerned about the potential of a major accident in the gate chamber or catastrophic failure of the plugging system during construction that would drain the reservoir under an uncontrolled discharge. The SWSC wanted a redundant measure of protection.

In order to meet the SWSC's requirement to safeguard public health and safety, the reservoir and the environment, the engineering consultant recommended installing a temporary concrete bulkhead similar to what was installed as part of the blind flange installation in the 1990s. The temporary bulkhead would contain the discharge of a rupture and prevent a catastrophic uncontrolled discharge, but could increase worker's risk in the gate chamber if an accident or failure occurred during working hours.

The engineering consultant contacted the original bulkhead designer from the previous project in the 1990s to have the company review and reissue the proven design. The bulkhead consists of nine cast-in-place concrete beams installed in the bell-mouth of the diversion tunnel (see Figure 36). No work would be allowed in the gate chamber until the bulkhead construction was complete.

Construction Overview

The design was completed and requests for bids advertised in December 2004. The highest bid of $2.9 million was based on installing the plug by diving. The three lower bids were based on including the plugging system in the design. The lowest bid was $1.4 million and the second and third lowest bidders were within $200,000 of the low bid. It was clear that the plugging system saved SWSC approximately $1.6 million in construction costs.

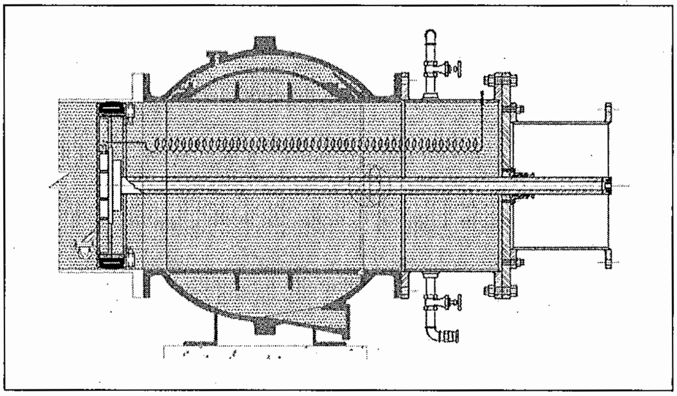


FIGURE 32. The insertion mechanism would be removed.

Construction of the bulkhead was a priority since no other work could be started in the gate chamber until it was completed (see Figure 37). Work began in June 2005. The diversion tunnel and gate house posed challenges to the contractor.

The only access to the gate chamber was gained through a 10- by 5-foot rectangular air shaft that was 233 feet deep (see Figure 38) or down an unlighted narrow stairway with forty flights of stairs (320 steps), and then down a brass ladder through a 24- by 24-inch access way from the control chamber to the gate chamber (see Figures 39 & 40). It was cool in the summer, but bone-chilling cold and damp during the other three seasons; there were bats hanging from the walls and occasionally flying around; walls were coated with slime and mold; and there were always musty odors present. The tunnel could be used initially but would be inaccessible after the bulkhead was constructed.

Use of the tunnel during bulkhead construction was constrained by its only access road, a 10-foot wide switchback trail

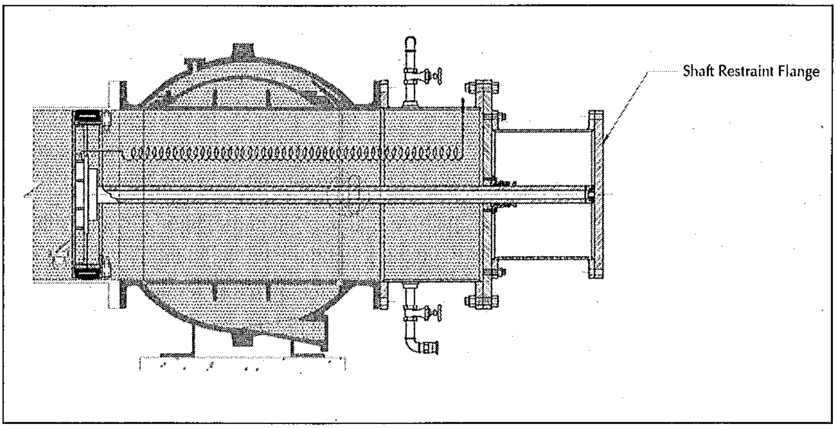


FIGURE 33. A plate would be installed to restrain the plug.

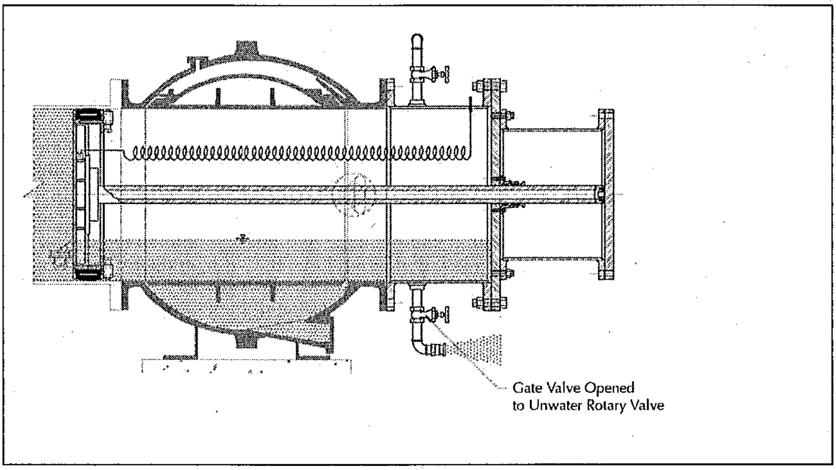


FIGURE 34. The plug would be inflated with nitrogen gas and the valve body drained.

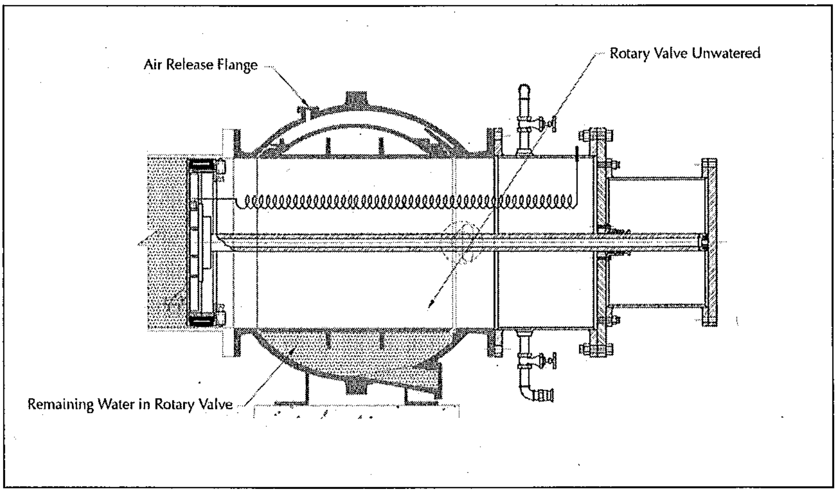


FIGURE 35. After the valve is drained, the auxiliary components can be replaced and the valve rehabilitated.

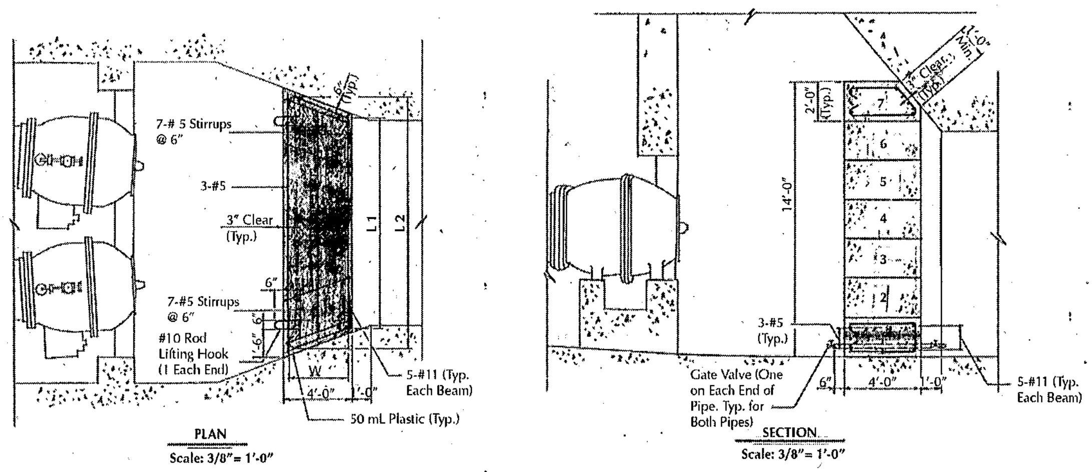


FIGURE 36. Concrete bulkhead plan and profile.

that wove its way down the Little River Gorge to the tunnel portal with 180-degree turns. Passage was difficult during the summer, but impossible during the winter. The air shaft was obviously the best alternative for access to the gate chamber.

The jet-flow gate HPUs posed an additional problem. The HPUs would be installed in the control chamber, and had to be lifted from the gate chamber in pieces through a 24- by 24-inch access-way.

The contractor installed a winch on the inside wall of the gate chamber with a capacity of 15,000 pounds and 225 feet of steel wire rope (see Figure 38). The winch was used to raise and lower materials and equipment to and from the bell-mouth chamber. A single cable was attached to the ceiling of the air shaft that would be used for an electrically operated lift-basket, providing easy access to the bell-mouth chamber by construction personnel and tools. A system of galvanized steel support beams were installed in the gate chamber and monorails were installed above each outlet (see Figure 41).

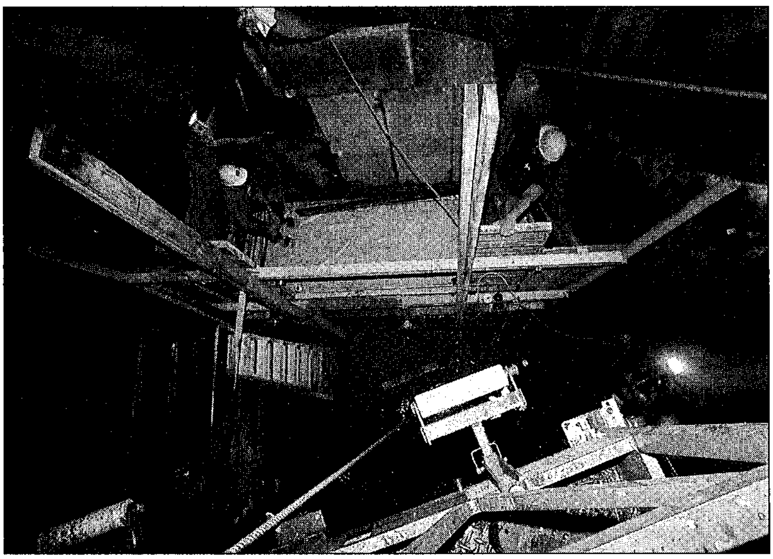


FIGURE 37. Bulkhead construction.

Equipment Fabrication

The contractor and fabricator responsible for the final design, and fabrication of the plug insertion device and the 30-inch jet-flow gates were both based in Massachusetts. The engineering consultant and SWSC project team were pleased that all of the equipment required for the project would be fabri-

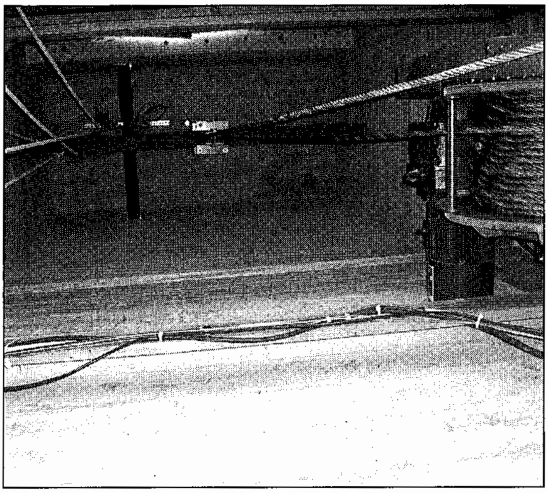


FIGURE 38. Air shaft winch installation.

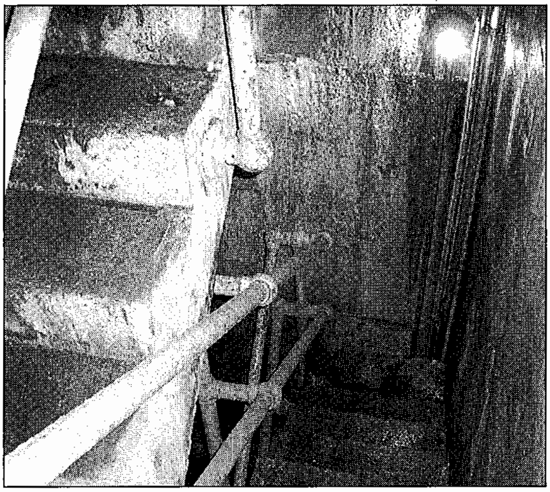


FIGURE 39. Access stairway.

cated locally in Massachusetts (see Figures 42 through 45).

Several improvements to the plug insertion device were developed by the fabricator during the final design and fabrication. Instead of using a blind flange to restrain the plug after inflation, two slots were machined into the insertion shaft to accept a keeper plate that would restrain the plug against a steel bracket mounted on the plug housing. In addition, at the request of project's contractor, the plug itself was designed with double elastomer seal rings back to back in order to provide redundancy and an extra measure of safety. The engineering consultant and SWSC were pleased with these improvements.

The plug frame was constructed of welded aluminum and mounted to the insertion device with a locknut on the threaded end of the shaft. The static pressure in the outlet is approximately 90 psi. The plug would have to be inflated to 50 psi above that pressure to create a seal. Nitrogen gas was selected to inflate both seal rings on the plug. Pressurized nitrogen in steel cylinders at 2,200 psi would provide additional response time in the case of a slow leak in one of the seal rings.

Shop inspections were conducted by personnel from the engineering consultant and SWSC at key milestones during fabrication, and a shop test was specified for both the

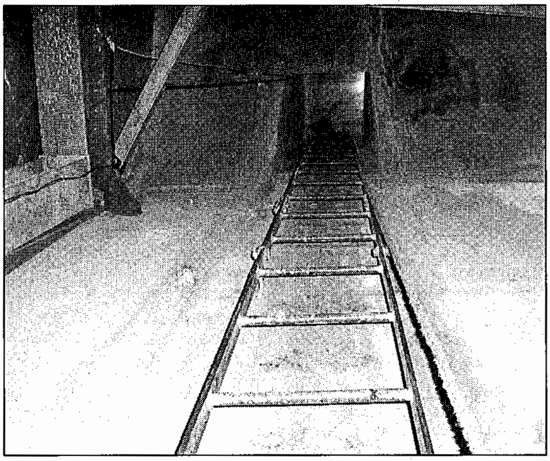


FIGURE 40. Gate chamber access ladder.

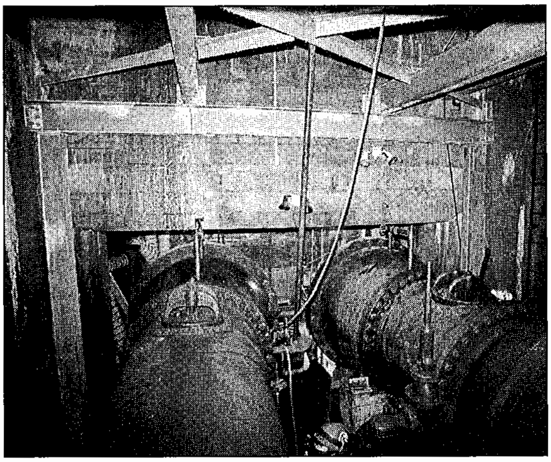


FIGURE 41. Gate chamber rigging equipment.

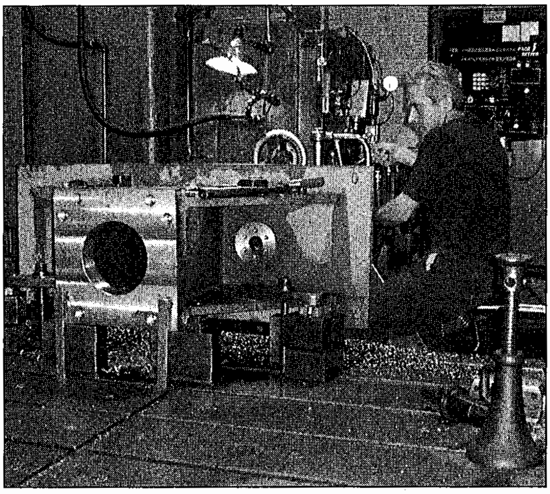


FIGURE 42. Jet-flow gate bonnet cover machining.

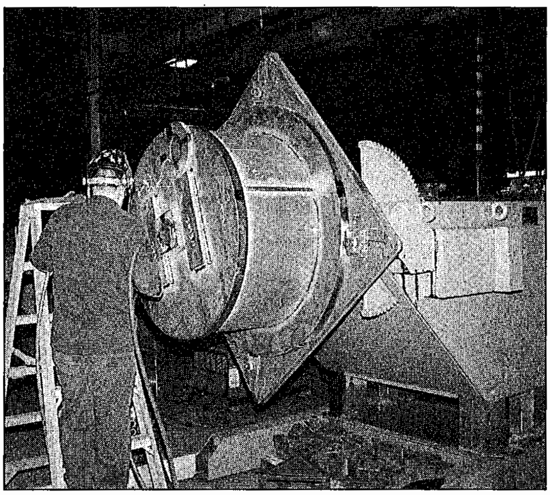


FIGURE 43. Plug insertion device during fabrication.

plugging system and jet-flow gate. The plugging system test was conducted on a test stand that simulated the conditions in the outlet (see Figure 46).

A custom-fabricated pipe spool with the same inside dimensions as the rotary valves and outlet conduit was set up on a test stand. Weld beads 0.375 inches in height were welded on the inside of the test pipe to simulate tuberculation and other obstructions on the inside of the rotary valve and outlet conduit. The plug was attached to the test stand, the pipe was filled with water and pressurized to the test pressure, and the plug was inserted into the pipe. As the plug was inserted, water had to be drained from the test stand to make up for the volume of the shaft.

The plug was inflated at the end of its insertion stroke, keeper plate installed and the water behind the plug drained to the atmosphere. A test pump maintained a pressure of 150 psi in front of the plug to simulate the reservoir plus safety factor, and the leakage was measured. The allowable leakage specified was 5 gallons per minute (gpm) since this flow was easily conveyed into the tunnel through the bulkhead drain pipe. Measured leakage was less than 5 gpm. The plug was retracted, the housing

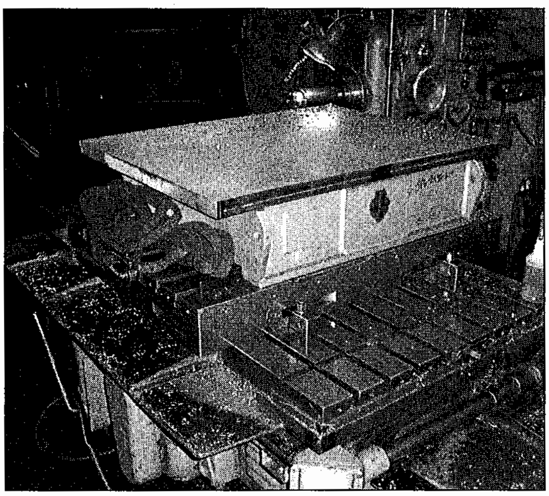


FIGURE 44. Jet-flow gate leaf machining.

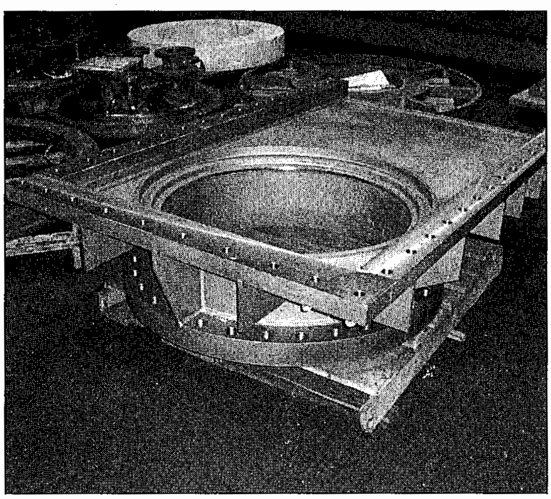


FIGURE 45. Jet-flow gate upstream body.

removed and the plug inspected for damage. The test was a complete success and the plugging system components were shipped to the site.

The jet-flow gate shop test included a hydrostatic leakage test and operational test to confirm smooth operation (see Figure 47). This test was conducted in August 2005 and was a complete success, with leakage measured at the specified limit. Since the jet-flow gate does not act as an isolation gate, the real purpose of the leakage test and functional test was to confirm that the dimensional tolerances were within those specified on the drawings. The test was successful and the gates were stored at the shop until the time of installation.

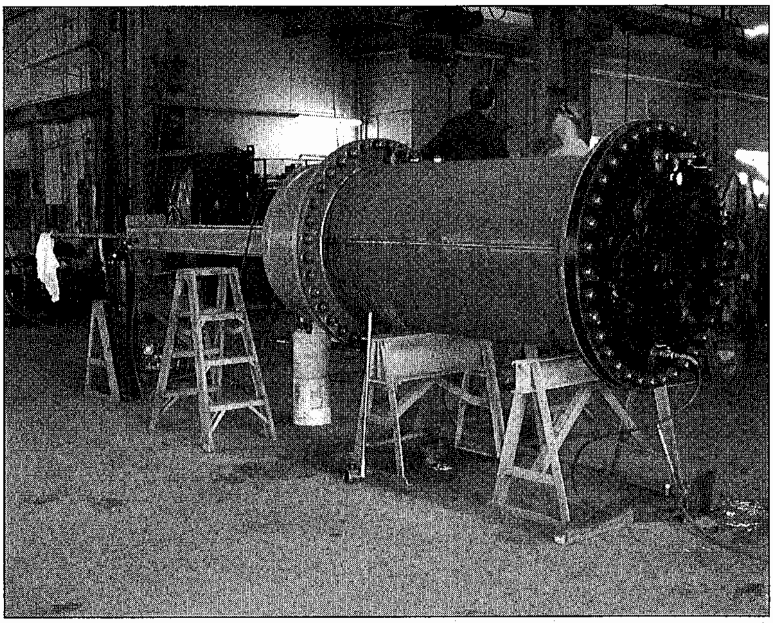


FIGURE 46. Plugging system test stand.

Rehabilitation & Installation Phase

After the bulkhead was completed and all components required for the project were on site (excluding the jet-flow gates), the contractor was allowed to commence demolition of the needle valves and intermediate piping. The gear drives on the rotary valves were disassembled and sent out for cleaning and painting. They would have to be rehabilitated and reinstalled prior to installation of the plugging system.

After demolition, the plugging system was lowered to the gate chamber, assembled and installed on the downstream flange of rotary valve no. 2 (see Figure 48). The plug was inserted through the valves without binding, rubbing or any other dimensional interference. The plug was inflated with nitrogen and there was zero leakage. Initially, it was anticipated that the plug would be installed for a short duration during disassembly and reassembly. The plug remained installed in each valve for over a month with zero leakage.

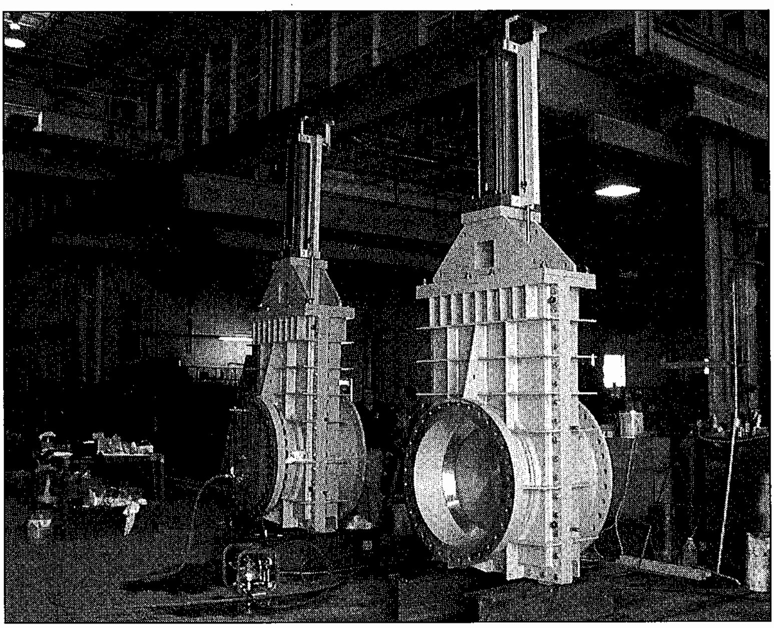


FIGURE 47. Jet-flow gate hydrostatic test during fabrication.

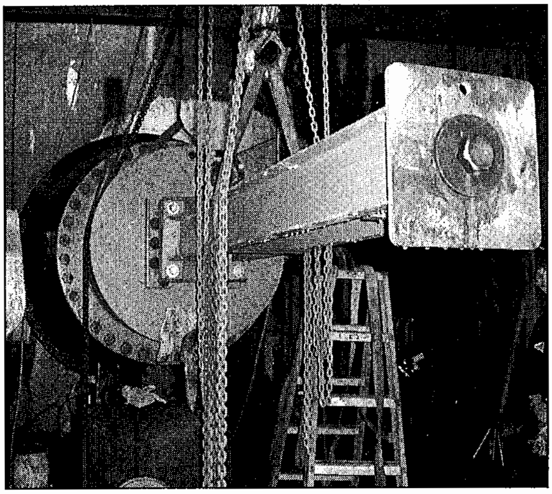


FIGURE 48. Plugging system installed on rotary valve no. 2.

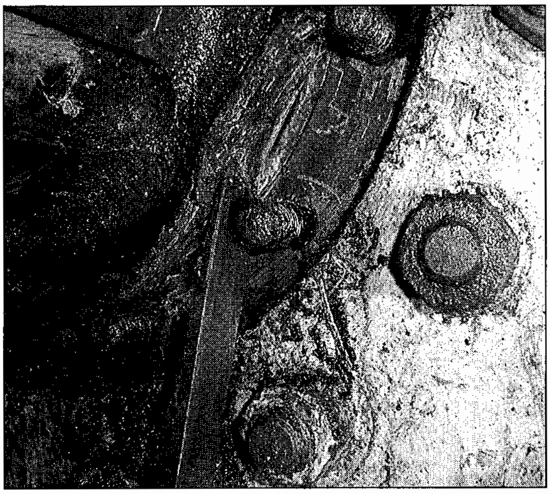


FIGURE 49. Rotary valve showing hat-leather seal and contractor ring.

With the rotary valves totally drained, the contractor was able to prepare the surface of the valves for a new finish coat of a high-performance epoxy coating. The bypass piping and valve, and the bottom flush valve blind flanges were removed, and the new stainless steel piping and valves were installed.

The seal gland was removed from the valve and the project team had its first look at the existing seal. The seal was constructed of leather, which was no surprise given the age of the valve. The configuration was a "hat-leather" flange seal that utilized a steel contractor ring and the hydrostatic pressure in the stuffing box to maintain the seal (see Figures 49 & 50). After removal of the existing seal,

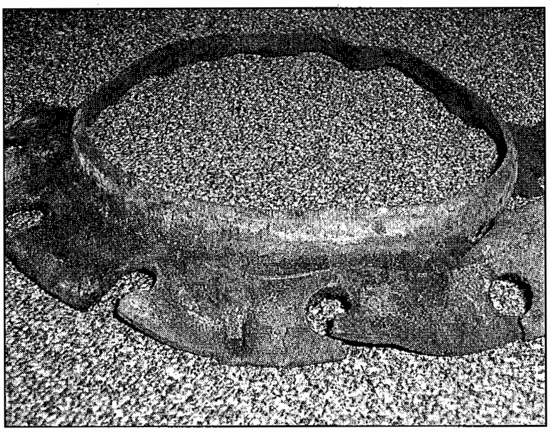


FIGURE 50. Hat-leather seal after removal.

dimensions of the stuffing box were taken and a new seal was designed by the engineering consultant for its replacement.

Unlike modern ball valves that have bodies constructed of four separate castings instead of two, the rotary valve stuffing box was split down the middle. Given the split and rough surface finish on the interior, the engineering consultant decided to design a new leather seal in a vee-packing configuration. The seal would consist of three packing rings with cast bronze male and female adapter rings (see Figure 51). The design was completed and a change order was issued for the new seal.

The leather seals were fabricated in Muskegon, Michigan. Custom-machined dies had to be fabricated to form the vee-packing rings due to the metric dimensions and large shaft diameter. The engineering consultant specified a petrolatum-impregnated leather for the rings, which would make them softer and more suitable for the surface finish in the stuffing box. The adapter rings were machined from bronze tubular castings in a local machine shop.

After the rehabilitation of the rotary valves was complete, the jet-flow gate and HPUs were delivered to the site for installation. Hydraulic piping was specified as Schedule 80 stainless steel with 3,000-pound socket welded fittings. The piping was TIG welded on site. By June 2006 the jet-flow gates and HPUs

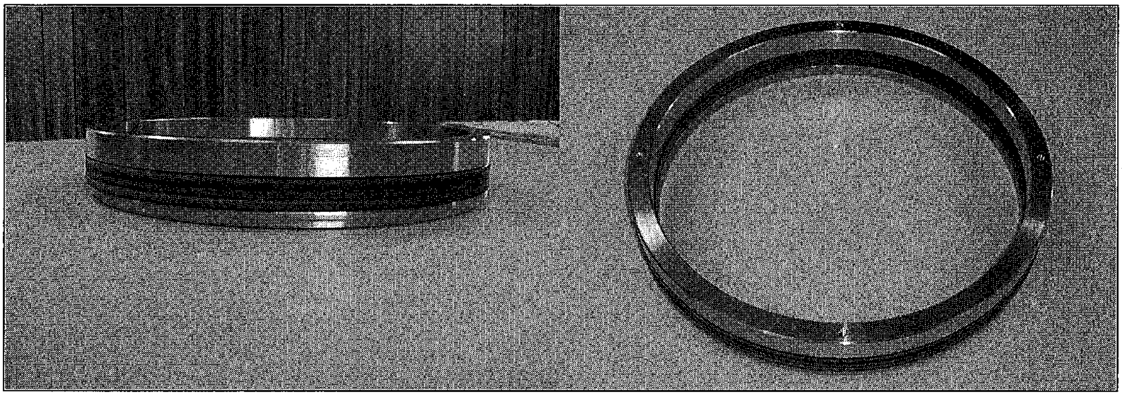


FIGURE 51. New leather vee-packing for the shaft seal.

were installed and ready for testing (see Figures 52 through 55).

Flow Test

In anticipation of the flow test, SWSC maintained the reservoir level at spillway crest elevation. At the maximum head, the jet-flow gates were expected to discharge a maximum flow at full open of 300 mgd (or 210,000 gpm) each. The exit velocity at the jet-flow gate would be approximately 75 feet per second. There was a major rainstorm on the designated test day but its occurrence did not impede the test.

The discharge flow from the gates into the tunnel creates an air demand that must be satisfied. This demand is known as insufflated air and the flow rates can be extremely high. The air demand is caused by the displacement of air by water droplets at high velocity and the aerodynamic drag caused by them. As the high-velocity spray generated by the flow increases, the air demand increases. This demand peaks at approximately 60 percent of gate stroke, and then drops off as the flow becomes a cylindrical jet and the spray diminishes. This phenomenon also occurred during needle valve operation, which is why the air shaft was constructed above the bell-mouth of the tunnel.

It is nearly impossible to accurately predict the air demand from the jet-flow gates but the project team needed to know the effect it would have on the static air pressure between the gate chamber and bell-mouth chamber,

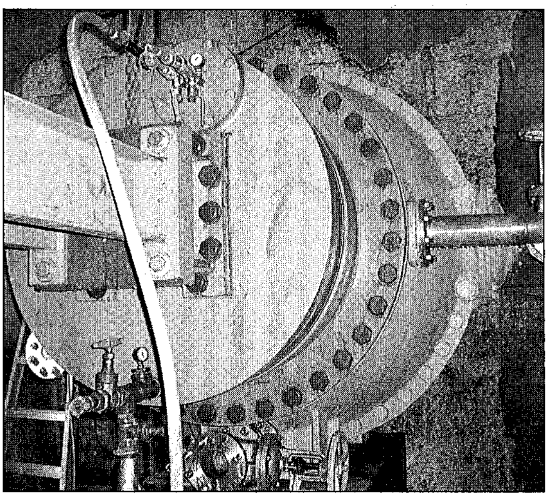


FIGURE 52. Rehabilitated rotary valve with plug.

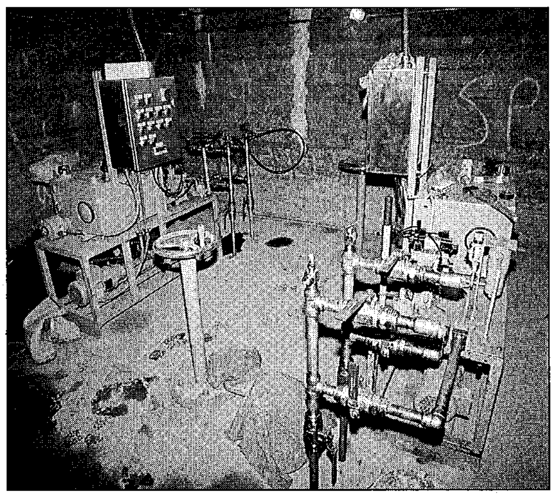


FIGURE 53. Jet-flow gate HPUs.

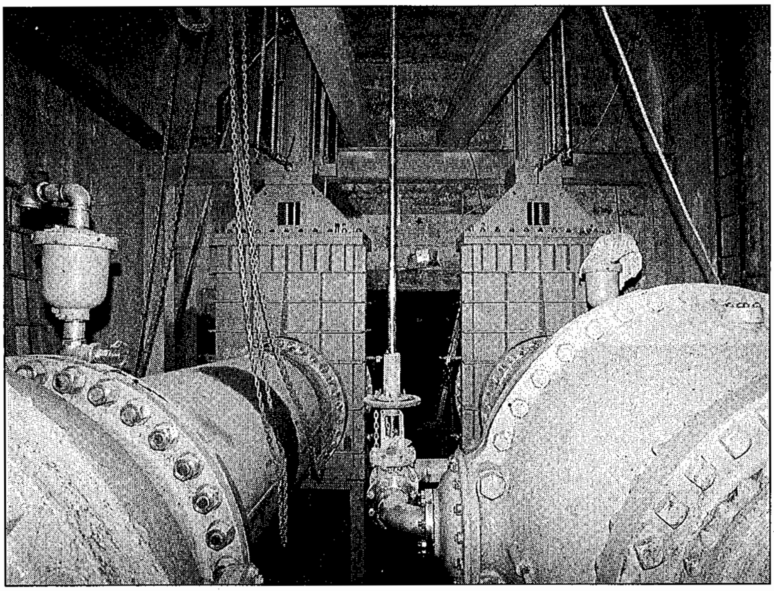


FIGURE 54. Jet-flow gates and rotary valves (view downstream).

which was separated by a new reinforced masonry wall. The wall was designed for a load of 16 pounds per square foot (psf), which equates to a differential static pressure of 3 inches-water-column. If the differential pressure exceeded this value, the wall could fall down. The design team considered installing a reinforced concrete wall instead of masonry, but this wall would limit access to the chamber for future maintenance without the use of heavy demolition equipment. It was better to quantify the differential pressure and provide means for vacuum relief.

A testing procedure was developed by the engineering consultant and instrumentation was installed to measure hydraulic and other parameters at various locations around the facility. A digital windspeed gauge with data logger was installed in the air shaft to measure air demand. A Magnahelic differential pressure gauge was installed between the tunnel bellmouth and gate chamber to measure differential pressure. A pressure transducer and data logger were installed upstream of the rotary valve on outlet no. 2, and a noise meter was installed in the gate chamber to determine whether hearing protection would be required in the future during operation. The upstream pressure would be used to calculate the flow at different gate positions using data provided from the USBR.

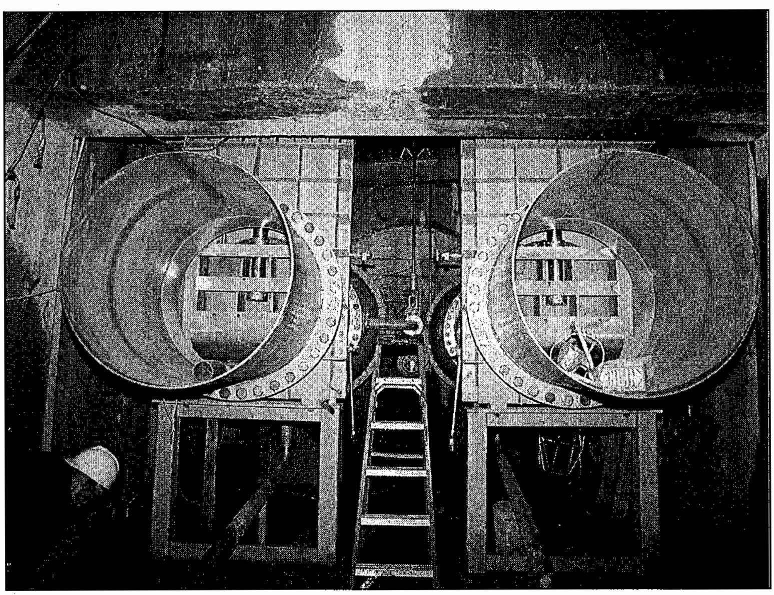


FIGURE 55. Jet-flow gates and discharge piping (view upstream).

The test was a complete success and was well documented. The maximum flow from both jet-flow gates was recorded at approximately 600 mgd, and the 11.5-foot diameter diversion tunnel was flowing full at the discharge portal, under pressure, at this flow rate (see Figure 56). The maximum airflow in the shaft with both gates fully open was 210,000 cubic feet per minute. The differential pressure between chambers was beyond measure, when both gates were opened simultaneously, and the gate chamber access door closed. The door was opened to allow sufficient air to flow down the access stairwell to relieve the vacuum. There was no vibration from the jet-flow gates during the test and noise levels were below OSHA limits at all times, even when the door was opened. The rotary valves created some periodic, loud cavitation that was clearly audible when the jet-flow gates were fully open, but it was not a cause of concern.

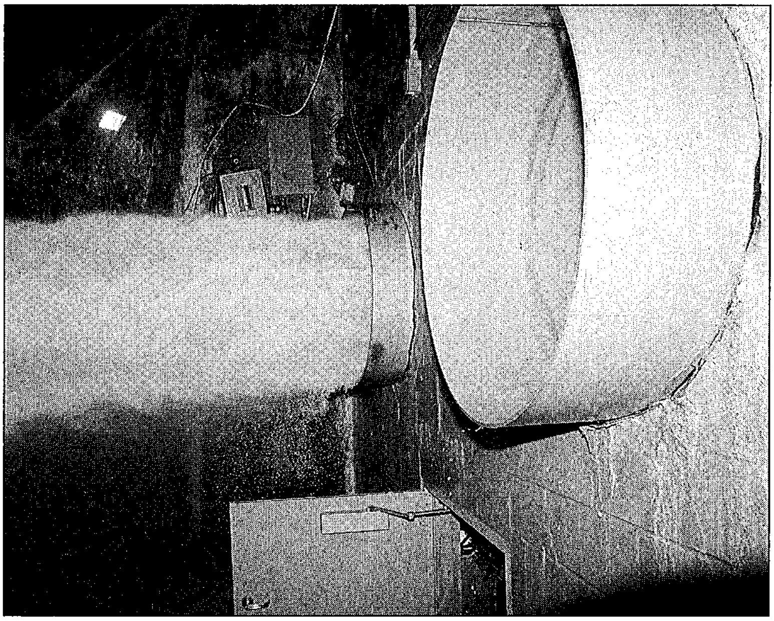


FIGURE 56. Discharge from jet-flow gate No. 2 fully opened.

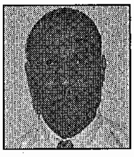
In order to minimize risk to the water supply and maintain safe working conditions, the construction schedule was extended by nine months beyond the completion date. The time extension had no major impact on the final cost of the project, which was \$1.44 million, with less than 3 percent net change orders.

Conclusion

SWSC, the engineering consultant and the contractor were very pleased with the success of this project, which:

- Included design solutions that showed resourcefulness in planning and execution;
- Pioneered the use of a unique non-entry pneumatic bulkhead plugging device for this application;
- Replaced potentially dangerous needle valves with jet-flow gates, the first installation of its type in New England;
- Returned control of the outlet works to the SWSC for safe and reliable service;
- Through innovation saved \$1.6 million dollars compared to the use of divers in a dangerous application, requiring decompression diving techniques; and
- Resulted in zero injuries in a high-risk confined space.

NOTES — *CDM provided engineering consulting and project management services. The Larner-Johnson differential needle valves were manufactured by the I.P. Morris Corporation of Philadelphia, Pennsylvania. The rotary valves were manufactured by Escher Wyss & Co., of Zürich, Switzerland, now a business unit in the Andritz Group of companies based in Graz, Austria. CDM and sub-consultant Conam Inspection Services conducted wet magnetic particle and ultrasonic thickness testing on a 2-inch grid across both halves of the valve body. CDM worked with Rodney Hunt Co. of Orange, Massachusetts, and Steel-Fab, Inc. of Fitchburg, Massachusetts, to determine the feasibility of fabricating the device capable of inserting the pnematic plug through the rotary valve, and to obtain an estimated cost. CDM contacted Seigmund Associates of Providence, Rhode Island, the original bulkhead designer from the previous project in the 1990s, to have it review and reissue the temporary bulkhead design. R.H. White Construction Co. was the contractor on the project. Steel-Fab, Inc., of Fitchburg, Massachusetts, was selected by R.H. White to perform the final design and fabrication of the plug insertion device and the jet-flow* *gates. The jet-flow gates were machined at Central Mass Machine in Holyoke, Massachusetts. The plug was designed and fabricated by Mechanical Research Co. in Manitowoc, Wisconsin. The leather seals were fabricated by C.W. Marsh Co. in Muskegon, Michigan.*



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Managing Human Error in Structural Engineering

"Be not ashamed of mistakes and thus make them crimes" — Confucius's ancient maxim suggests that avoiding future error is the higher form of justice.

DAVID P. BROSNAN

In a growing number of industries, managers are recognizing that what goes wrong is often a broadly predictable product of organizational information and personal judgment. In particular, the aviation profession invests heavily in the study of human behavior in high-pressure, technically complex environments. The findings of its systematic collection and analysis of data have led to remarkable improvements in control systems, information quality, personnel management, and most of all, public safety. The revolutionary thinking and methods that have emerged from that work have been adopted by engineers who control nuclear power plants, the captains of oceangoing vessels, firefighters and doctors and nurses in hospital operating rooms.

Human error explains many events other than airplane crashes. The nuclear reactor accidents at both Three Mile Island and Chernobyl stemmed from flawed human interaction with machines and with other people. A major flood in downtown Chicago in 1992 that caused a billion dollars of property damage was attributed to piles driven mistakenly through a tunnel. The explosion of a pesticide plant in Bhopal, India, was blamed on operational errors of the staff. Studies of American health care estimate that as many as 98,000 persons die every year because of mistaken diagnoses, incorrect medication and other types of medical errors.

Structural engineering can benefit from the application of human factors principles so that error in engineering design and construction work can be anticipated and managed in the interest of public safety. Knowing that most aircraft accidents happen because of flawed decision making, bad communications and poor teamwork has brought about a deeper appreciation of the limits of human understanding and human performance. The human beings who design and build structures possess all of the same limitations and shortcomings of those who fly aircraft. Like aviators, structural engineers owe the public their best efforts to control errors in their work.

From a human factors point of view, many sectors of the structural engineering profession have embraced extreme and mistaken positions about the role of people in structural

TABLE 1.
The Sources of Structural Failure According to the Beliefs of an "Expert Witness"

Causes of Failure	Characteristics
Negligence	Improper analysis or design; standards disregarded
Incompetence	Misunderstood or misapplied engineering principles or characteristics of systems & materials
Ignorance, Oversight	Design documents & safe construction practices not followed
Greed	Short-cuts for profit; intentionally ignoring codes & safety
Disorganization	Unclear lines of authority & poorly defined roles & responsibilities of parties
Miscommunication	Fragmented or bypassed lines of communications among parties
Misuse, Abuse, Neglect	Improper occupancy or maintenance of facility

From Ref. 3

accidents and failures. In professional literature one finds a fair amount of moral posturing.[1] For wider audiences, writings (like those of Henry Petroski) explain accidents and structural failures as the price society sometimes has to pay for technological progress.[2] Efforts to intimidate practitioners into good behavior have obscured the difference between error and ethical impropriety. The frequency of finger-pointing litigation in the construction business, and the public indifference to some types of danger like highway accidents, do not help. Table 1 shows the opinion of a structural engineer who provides expert testimony in court cases about the sources of accidents and failures.[3] Sanctimony has even crept into the pronouncements of important institutions. The National Academy of Engineering (NAE), for example, lavishes praise on its "moral exemplars" who called attention to mistakes at the risk of their reputations. But the Great Man Theory implicit in those retrospectives offers little guidance to most engineering practitioners. One of the NAE's most celebrated case studies, the Citicorp Tower in New York, has in fact raised even larger questions about day-to-day decision making in structural engineering.[4] Everyone makes mistakes; absent a deliberate act, structural failures should not be classified as moral failures. All investigations of structural failures need not degenerate into a rogue's march in which the "guilty" are publicly stripped of their dignity. From a human factors perspective, the more important questions to ask are:

- Why did a qualified individual happen to make this mistake?
- How can it be avoided in the future?

The correctness of engineering calculations represents only one part of the large scope of potential human error in construction work. To effectively control error, engineers also need to recognize the conditions that lead individuals and groups to take unintentional risks, embrace false priorities and promote flawed thinking. But it does not end with the engineer or the engineering firm. The rules governing technical decisions, the project environments in which structural engineers work, the clarity and usability of their reference materials, and the interfaces between engineers and computational tools also promote engineering error in significant and serious ways.

Basic Concepts of Error Management

The goal of error management is to classify the different types of errors that occur, and to use appropriate techniques to reduce the occur-

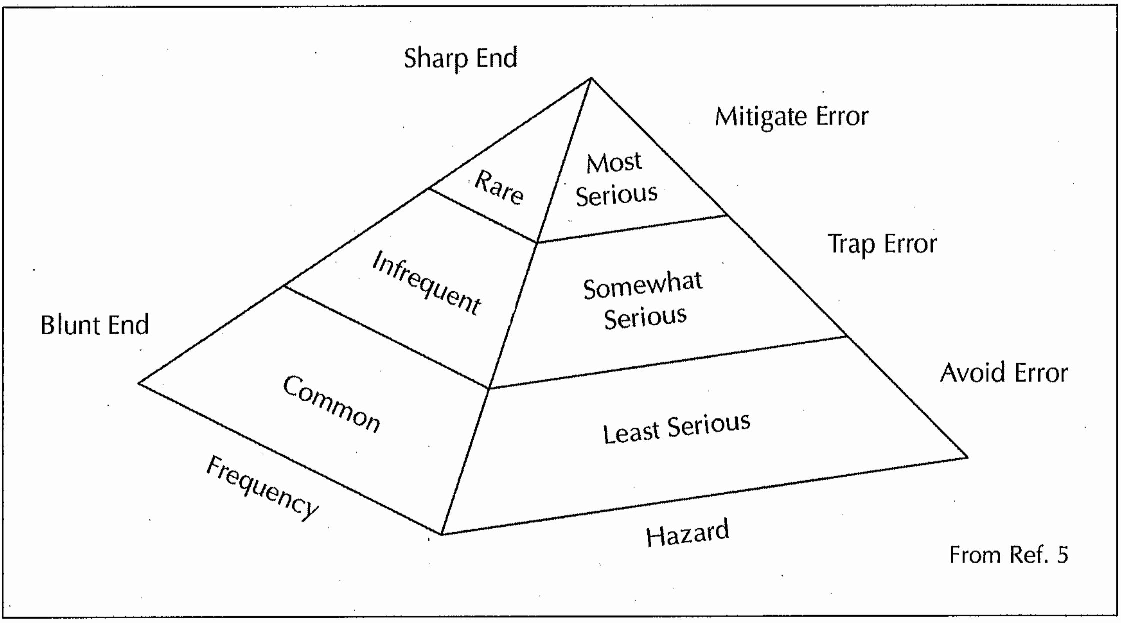

FIGURE 1. The pyramid model of error distribution.

rence and consequence of those errors. The pyramid model of error management, as illustrated in Figure 1, correlates the seriousness and the frequency of errors.[5] In this model, the most potentially dangerous errors are the least likely, and the most common errors are the least serious. In structural engineering, the most common errors should be addressed through education and training. More serious but less frequent errors would be caught with error trapping techniques such as peer review. The most dangerous errors — those that slip through the quality control procedures and appear at the sharp end of the pyramid — must be mitigated. Mitigation methods for structures generally require the establishment of four qualitative properties: proper configuration, continuity, ductility and redundancy. These properties, when present in a structure, assure that local failures do not progress into larger catastrophes.[6]

To make the pyramid model effective, reliable information about all sorts of engineering and construction errors must be available. At this time, sufficient data really do not exist to determine whether error among structural engineers is presently distributed according to the pyramid model. One study shows that 78 percent of structural failures have been traced to some form of human error.[7] (The rate of human error in the airline industry was almost identical before error management techniques were instituted.) Another study indicates that over 50 percent of structural failures involve some form of water.[8] Putting this limited information together, it appears that nearly four in ten structural failures can probably be attributed to errors of detailing rather than errors of computation. This interpretation tends to confirm the commonly held view of practitioners that an exclusively quantitative approach to engineering does not always suffice and must be informed by qualitative considerations. Still, the statistics presently at hand do imply that fruitful results can be achieved if error management is focused on the specific kinds of errors most responsible for failures.

The management methods of aviation offer some insight into what really may cause errors to occur. Investigators who study the errors made by military pilots have compiled a supervisory checklist of the factors believed to be related to them.[9] This checklist is set forth in Table 2. To a structural engineer, it will look manifestly different from the list of personal shortcomings suggested by the professional courtroom witness in Table 1. The work of

TABLE 2.
Human Factors Crucial to Accident Prevention Among Military Pilots

Causes of Accident	Characteristics
Sensory/Perceptual Factors	Degree of situational awareness
Medical/Physiological Factors	Health & fitness for the task
Knowledge/Skill Factors	Training & proficiency
Personality & Safety Attitude	Understanding & controlling risky behavior
Judgment-Risk Decision Factors	Knowing acceptable criteria for decision making
Communications/Coordination	Procedures, workload & information
Design/System Factors	Human-machine interaction
Supervisory Factors	Leadership & work environment

From Ref. 9

practicing engineers, by such reasoning, should be viewed as part of an interactive system, not as discrete choices between good and evil. The factors known to produce accidents further suggest that errors may be predictable when unfavorable circumstances compound one another. For example, a mildly ill engineer working on a tight deadline with ambiguous reference material would certainly be more likely to produce a mistake.

The thinking about safety prevalent in the aviation profession's training, literature and practice represents a very advanced and enlightened approach to error and its relationship to human factors. While some authors of engineering texts continue to cite Hammurabi's Code as an example of quality control procedures[10,11] and focus on a small number of large disasters, other industries and professional groups have learned from human factors principles to make significant improvements in every day practice. Engineering organizations such as the Institute of Electrical and Electronics Engineers, the Society of Automotive Engineers, the Society of Naval Architects and Marine Engineers and the Industrial Designers Society of America have all promoted the use of human factors techniques. Some business organizations have done likewise. One aircraft manufacturer states:

"Because technology continues to evolve faster than the ability to predict how humans will interact with it, the industry can no longer depend as much on experience and intuition to guide decisions related to human performance."[12]

Individual Error

An essential prerequisite for minimizing error is knowledge of human abilities, behavior and thinking in the work environment. Individuals produce three kinds of errors: knowledge-based error, skill-based error and judgment-based error. As individuals progress through different levels of skill, training and experience, they develop different attitudes toward their work and different beliefs about their own proficiency. Flawed thinking, poor skills and bad judgment are possible at all levels of education and responsibility. Table 3, based on military human factors guidelines for flight officers, shows the dangers inherent at each level of advancement.[13] The most skilled and most knowledgeable people are not always the safest.

Both military and civil aviation training have emphasized the need for sound decision making in high-pressure situations when the stakes are particularly high. In civil aviation, prospective pilots are required to know the types of flawed thinking that lead to accidents.

TABLE 3.
Risky Behavior in Military Pilots at Different Career Stages

Type of Individual	Problem	Traits	Danger
Rookie	Lack of self-confidence	Skills still developing Limited procedural knowledge Easily intimidated	Poor situational awareness Fails to speak up Does not know the rules
"Top Gun"	Overconfident	Works alone Thrives on challenges Pushes the envelope Finishes tasks "at any cost"	Poor perception of risk Poor teamwork False priorities Disregards the rules
Senior Staff	Puts judgement before procedure	"Been there, done that" Intimidates junior staff Overemphasizes experience Pulls rank	Rusty technical skills Fails to recognize own limits Bends the rules
Overstressed	Personal/family or financial issues	Distracted, depressed, hostile, or preoccupied Not focused on task	Poor situational awareness Violates the rules
Poor Performer	Poor basic skills	Never improves Easily overloaded	High rate of technical error Dependency on colleagues Misinterprets the rules

From Ref. 13

These are known as the Five Hazardous Attitudes, shown in Table 4.[14] When pilot error has been cited by investigating authorities as the cause of an aircraft accident, one or more of the hazardous attitudes is invariably blamed. Recognition of such attitudes is emphasized in pilot training because knowledge-based error and skill-based error do not produce as large an accident rate. Military aviation units convene "human factors boards" to examine pilot on-the-job performance and decision making. Engineers may conclude from their own experience that the Five Hazardous Attitudes are not confined to aviation.

But human personality and frame of mind are not the only individual characteristics that contribute to human error. Human beings have limits to their perception and performance. People make errors in repetitive tasks at a far higher rate than machines or computers do. Humans get tired and fatigued. They can be distracted and deceived. One airplane manufacturer's view of pilot error states that "errors typically occur when the crew does not perceive a problem." The manufacturer therefore promotes the use of "visual and tactile cues" in its aircraft instrument panels to "reinforce situational awareness."[12] Even as human error may be the most important cause of accident and failure, sound human judgment may be the best safeguard against it. One researcher's work shows that both medical doctors and airline pilots consistently overestimate the quality of their thinking and work when on duty for long periods of time.[5] The system of working young medical interns for long shifts has recently been attacked as a major source of error in hospitals.[15]

Human nature plays a bigger role in technical decision making than most engineers are

TABLE 4.
The Five Hazardous Attitudes of Civil Aviation

Hazardous Attitude	Mistaken Line of Thinking
Anti-Authority	The rules don't apply to me or to my special situation.
Impulsiveness	Hurry up! Get it over with! Do it now!
Invulnerability	Only incompetent, greedy or ignorant people make mistakes.
Macho	Let me show you the amazing things I can do.
Resignation	It doesn't matter what I do. I don't make a difference.

From Ref. 14

willing to admit. But it should come as no surprise to anyone that when presented with unusually difficult project parameters, most reasonable people will make simplifying assumptions that break them into manageable pieces. When handed a long list of things to do, they will use their judgment to prioritize it. When told to adopt unfamiliar techniques in their work, most will try to avoid them in favor of what they already know and trust. When given conflicting information, they will try to reconcile it. When put under time pressure, most people will try to meet the deadline. All of these common reactions to ordinary situations involve weighing alternatives, making judgments and taking small risks. The more difficulty, tedium, unfamiliarity, conflict and time pressure introduced to the engineer's assignment, the more error one should expect.

This more highly developed way of thinking about the role of individuals in failure and error differs from clinical accounts of structural failure that focus on the mechanics of collapse without closely considering the human influences on the entire process of design and construction. It also casts doubt on the views of expert witnesses and forensic engineers who appear to demonstrate an unjustified degree of certitude when they use words like ignorance, incompetence and greed. However, advanced thinking will not excuse bad results. Rather, the study and management of human error, unlike the belated declarations now prevalent in structural engineering, offer the engineering firm reasonable tools with which to better understand and improve human performance.

Collective Error

Organizations, teams and less formally constituted groups of co-workers contribute to error in different ways than individuals. Organizations provide their members with essential information that may be incorrect, incomplete or out of date. They establish rules and policies that may be confusing or difficult to apply to real situations. Organizations are often responsible for their members' training and are usually responsible for their supervision. They set budgets, schedules and organizational priorities — all of which may limit choices and affect judgment. They purchase tools and equipment that may be awkward or unreliable. They provide a work environment that may be uncomfortable or distracting. Most important, organizations choose the leaders who, good or bad, set a visible example for their subordinates.

One might think that a group of similarly qualified people could put more eyeballs on a technical problem and therefore find more mistakes than an individual. The Wikipedia online information resource, for example, is based on this very idea.[16] But groups can take on their own odd character and frequently become less than the sum of their parts. Groups make mistakes of a much greater magnitude than individuals do. One of the ironies of history is that the worst blunders seem to

TABLE 5.
Evidence of Flawed Organizational Decision Making & Culture

Trait	Characteristics
Illusion of invulnerability	Overly optimistic; encourages risk taking
Collective rationalization	Dismisses practical objections
Unquestioned morality	Dismisses ethical objections
Stereotyped views	Equates disagreement with ignorance
Direct pressure	Challenges dissenters as disloyal
Self-censorship	Participants suppress their own objections
Illusion of unanimity	Equates silence with agreement
Self-appointed thought police	Squelches discussion of unpleasant realities

From Ref. 18

come from groups of intelligent, principled and tough-minded people who honestly believed they were doing the right things for the right reasons.[17] The typical problem is not unethical behavior among the members, but insularity. Groups frequently consist of people having knowledge, skills, experience and beliefs that are very much alike. Such a condition could easily exist in an engineering firm, where all present might be likely to agree with one another about technical issues, if not about politics or music. "Groupthink" insists on its mistaken preconceptions and offers doctrinaire answers to difficult questions, often at the expense of balanced judgment. Table 5 outlines the fallacies and pressures that constitute groupthink.[18]

David Beaty, a British aviator who was one of the first to recognize the role of human factors in accidents, wrote of group dynamics: "We are herd animals, and if we want to keep our position or status, we do what the herd wants."[19] Groups can go to extraordinary lengths to avoid confrontations among their members. The avoidance of confrontation sometimes becomes the all-important, if unspoken, goal of a meeting, leading individuals to acquiesce in decisions with which they would otherwise disagree. A strange phenomenon, known among psychologists as the Abilene Paradox, occurs when the outcome of a group decision is the single course of action which every person in the group privately opposes. In some cases, the members of a group are intimidated by a leader with a very strong personality. But sometimes, also, groups simply adhere to false priorities like "quick meetings" and "no arguments" that improperly override real priorities requiring difficult questions and unpopular positions.[20]

In this light, one might consider differently the efforts of some professional organizations to more fully standardize structural engineering education. Proposed minium professional degree requirements stipulate plenty of classroom instruction but only minimal independent scholarship and research.[21] If everyone has exactly the same educational background, engineers might well come up with identical numerical results but also be too willing to agree with their colleagues, particularly if employers reward them for conformity of thought.

The culture of an organization and the quality of project leadership also can introduce major blind spots and even arrogance with respect to safety and error. Some managers find internal discussion and disagreement about technical issues to be a good thing that stimulates thinking; others consider it a sign of insubordination and disrespect. One can find engineers from all over the world in

TABLE 6.
Classification of Problem-Solving Errors in Technical Work

Type of Error	Characteristics	Dangers
Bounded Rationality	Oversimplifies complex issues	Disregards information
Imperfect Rationality	Relies only on past experience	Does not apply basic principles
Reluctant Rationality	Jumps to conclusions	Fails to explore all possibilities

From Ref. 25

professional offices. Their different ethnic origins, religions, traditions and educational systems can sometimes bump into one another. For example, American males are considered most likely among national groups to openly question instructions or assignments.[22] Members of other national groups may be inclined to show (or expect) more deference. Neither situation should be viewed as entirely good or bad. Feedback and obedience are not engineering methods, but the interactions between people can have a real effect on an engineering product. The aviation profession, after studying accidents attributed to poor relations between pilots, now encourages the captains of airliners to take heed of their junior colleagues when they dispute information or directions.

Engineering Error

Engineering errors are often found not in the mathematics, but in the judgments that make the work unrepresentative of the actual system under analysis. Researchers studying error in engineering work discovered that technical proficiency quickly becomes irrelevant when the engineer does not properly choose or model the correct problem to solve. They also found that engineers improperly try to fit old familiar solutions to new and different technical problems; some even fail to consider the basic boundary conditions.[23] Such short cuts become even more dangerous when the old solutions are mistakenly thought to be successful. Some engineers rule out alternative solutions to problems far too quickly and arbitrarily. Such traits are well known to those who study human error, but recently a government-funded researcher suggested that structural engineers could check their engineering work by looking at the drawings of other projects.[24] Many engineers may recall being counseled by their superiors to make their work consistent with previous designs issued to the same client.

Further observations of engineering and technical employees have helped to classify the kinds of engineering work that generate more errors than others. The technical problems that seem to bring forth mistakes and poor judgment have one or more of the following characteristics:[25]

- Problems involving more than two or three different design variables.
- Problems with strong cues that suggest the wrong solution.
- A wrong solution has "successfully" been used before for a similar problem.
- The choice of an appropriate solution requires a novel approach.

The reactions of engineers to these various types of problem-solving difficulties have been divided into three categories, as shown in Table 6, which describes technical errors in comparison to an ideal of "rational" thinking and action.

Engineers often juggle several projects at a time, and sometimes they pressure themselves into performing several tasks at the same time. Multi-tasking is a known cause of error that reduces situational awareness in all tasks being done simultaneously. The pitfalls of multi-tasking first became known in the publishing industry among proofreaders.[19] Publishers have long understood that proofreaders must make a choice when reading a document. If they look at the text closely for errors of punctuation and grammar, they will have no clue about the plot. If they read to follow the characters and story, then they will miss the mistakes in the text. These findings have implications for the structural engineering peer reviewers who "proofread" construction documents. Quantitative and qualitative checks must be run separately. Those who pore over voluminous calculations looking for quantitative errors will inevitably miss the qualitative mistakes, and vice versa.

Error Propagation

Today, most examinations of engineering error happen after a structural failure or other unsettling discovery. Even peer reviews ordinarily occur after the construction documents are essentially complete. While such reviews can be useful in isolating the errors, they do little to explain why the errors happen, the contributing causes or how they can be prevented. This lack of context has been a difficult problem even in aviation, where pilots or mechanics are often the last people in a long chain of decision making over which they have minimal control. One aircraft manufacturer has lamented that "the only data guaranteed to be collected is [sic] that related to accidents and major incidents. . . it is difficult to obtain insightful data in an aviation system that focuses on accountability. . . . We must overcome this 'blame' culture and encourage all members of our operations to be forthcoming."[26]

Without an intentionally malevolent actor, it usually takes a series of errors or omissions to generate an actual failure. The passage of an undetected error through many layers of authority and safeguards is described using the "Swiss Cheese Model" developed by Professor James Reason. In the Swiss cheese model, each slice of cheese represents either a step in a sequential process, or a layer of responsibility. The holes in the slices of cheese represent systemic flaws, individual shortcomings or "blind spots" that offer opportunities for error. In order to achieve the conditions in which failures occur, a direct path must be made through all the slices of cheese between the flaw and the failure.[27]

In building construction, relatively few steps of the process fall under the direct control of a structural engineer. The objective of error management in engineering offices should be to avoid both large holes and clusters of small holes. This rationale accepts the possibility that the structural engineer's slices of cheese will always have some holes (or in other words, that errors will inevitably occur). The Swiss cheese model addresses the ability of errors to continue moving through a system.[28] Figure 2 illustrates the Swiss cheese model for the process of designing and building a structure. Because structural engineers' authority is staggered among various stages of the process, they have the uncommon ability to prevent the mistakes and oversights of others from passing through their own slices of cheese.

Both individual and organizational mistakes contribute to the propagation of error through a multi-layered or sequential process. Most mistakes initiated by individuals are categorized as "active" errors. Active errors occur in decision making, computation and drafting of documents, choices between alternatives, detailing of connections, in-shop fabrication and in-field assembly of structures. Examples of active errors include an incorrectly sized or labeled structural element, an improper application of building code provisions, an improperly welded connection or a form-release agent sprayed on steel reinforcing bars in a concrete wall.

Errors occurring as a result of organizational shortcomings are known as "latent" errors. Latent errors do not require the active participation of an individual. They exist because of poor management, unclear technical standards, conflicting or rapidly changing project criteria, bad human-machine interaction and poor teamwork. Examples of latent errors are an unrealistic deadline for completion of construction documents, flaws in engineering software or an overly optimistic budget for construction work. Latent errors frequently occur early in the sequence of events and,

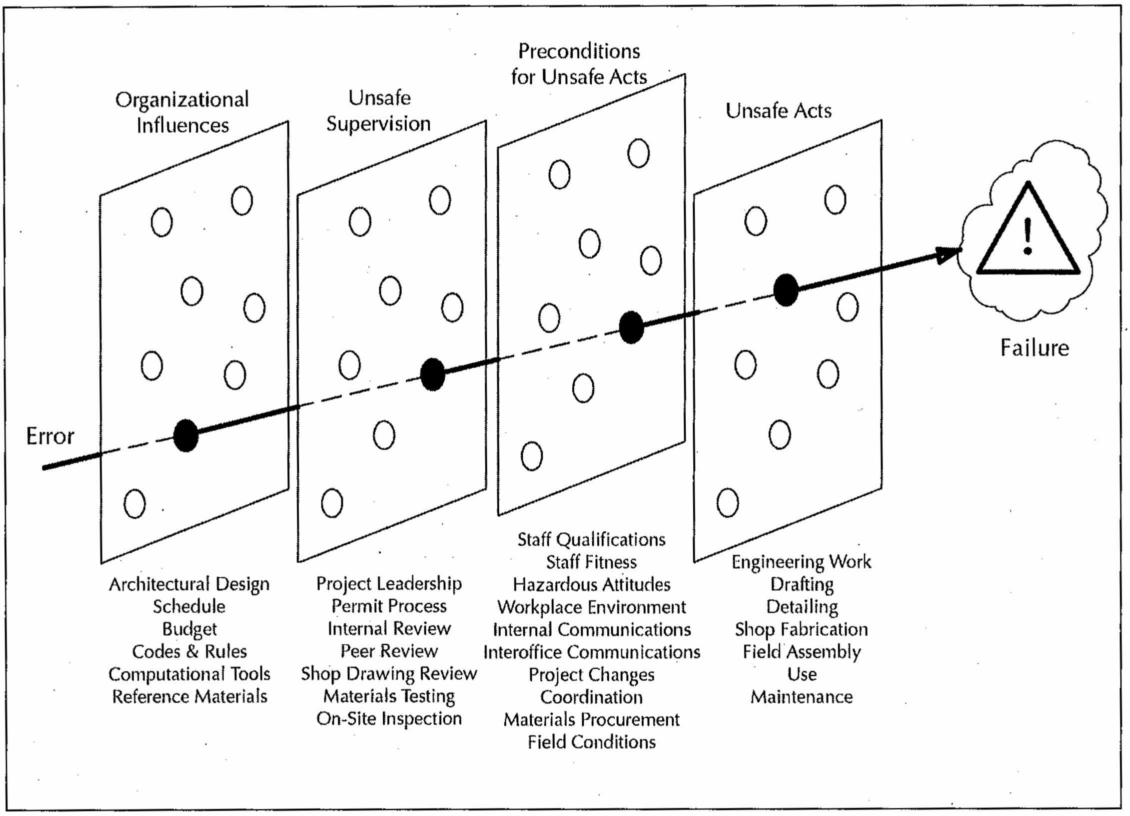

FIGURE 2. The Swiss cheese model of error propagation as applied to structural engineering.

therefore, can exclude the possibility of correction by those most affected, leaving them nothing but bad choices at the end of the sequence. Experience teaches that those who help bring about latent errors usually enjoy immunity from punishment, while those who make active errors more typically pay the price.

In order to recognize and act on both the errors and the propagation of errors through the Swiss cheese model, the Human Factors Analysis and Classification System (HFACS) has been developed by the U.S. Navy.[29] The HFACS considers four categories of performance, as shown in Table 7. Unsafe acts are the most active types of errors, while organizational influences are considered the most latent. HFACS is a tool that helps systematically analyze information about safety problems and generate recommendations. But such a tool requires a body of data. The aviation industry, followed by others, has encouraged its employees to report even minor incidents and "no harm/no foul" mistakes in order to gather such data for analysis.

Recommended Improvements for Successful Error Management

The structural engineering profession should consider the adoption of a more modern and enlightened approach to the management of error. Arbitrary after-the-fact punishment for mistakes cannot be the sole means of regulating human behavior. Evidence from other professions and industries shows that it is neither effective nor justifiable in work that involves uncertainty or complexity. Opinions of an engineer's personal character or ethical grounding should not be tied to one's performance at an engineering task. Quality control procedures for engineering work should address the kinds of faulty information, thinking and workplace conditions that engender human error. The valuable work done for

TABLE 7.
Human Factors Analysis & Classification System Used in Aviation, Medicine & Fire Fighting

Error Classification	Error Type	Examples
Unsafe Acts	Active	Bad execution of skill-based tasks Correct execution of wrong procedure
Preconditions for Unsafe Acts	Active or Latent	Poor situational awareness, illness, fatigue, fixation Poor communication with other team members
Unsafe Supervision	Latent	Inadequate training Inadequate oversight, guidance, pace-setting Failure to measure & track performance
Organizational Influences	Latent	Ambiguous direction or documentation Failure to provide safe methods & procedures Lack of appropriate tools

From Ref. 29

other industries should no longer be ignored in favor of intuitive management techniques, hero worship and primitive reactions to avoidable events. Improvements are clearly needed in four key areas:

- technical documents;
- the engineer-computer interface;
- engineering management; and,
- the collection of data about engineering and construction error.

Structural engineers can accurately think of themselves as consumers of technical information. They have every right to expect high standards of clarity and usability from publishers of building codes and other complex technical manuals and documents. Technical information sources should consider a system of uniform heuristics for their publications, understanding that those products are reference materials and their users hardly ever read them in full. Nothing prevents the color printing processes used in engineering magazines and mass mailings from being used also for the benefit of public safety by giving proper emphasis to important provisions and making information easier to find. Fundamental clarity and usability of engineering standards are not optional. They should not be subordinated to institutional false priorities like the advancement of new design methodologies or units of measurement used infrequently at best.[30]

Organizations that publish public safety regulations should avoid frequent and repeated revisions to technical standards, their methodology, their nomenclature or their governing philosophy. One can make a valid argument that the confusion caused by those changes may represent as great a danger to the public as the original unchanged code provisions. Governing bodies should seek instead to produce regulations that are written for the needs of the actual readership. When major changes are believed to be warranted, technical journals and publications should be used to inform practitioners and invite profession-wide discussion before deciding on their adoption. The experience of working engineers remains one the most essential safeguards for the safety of the public. The technical rules of engineering practice need not be altered so often and so completely that hard-won years of experience can be made worthless.

The profession likewise lacks uniform standards for the engineer/computer interface. This deficiency is becoming a larger problem as software companies now can alter their source code virtually at will over the Internet. In some self-updating software arrangements, actual results obtained from the same user inputs can vary from one month to the next. In other lines of work, such variations are not tolerated. Surgeons have a standard set of instruments with which to operate, and aviators have standardized control panels. Structural engineers should have analysis and design software that provides full and meaningful information in a concise and useful way. Users, and not software vendors, should define the minimum requirements of usability, output format and error avoidance. Wide variations exist among products in the presentation and format of results, and engineering software companies accept no legal responsibility for the quality of their products.[30]

Engineers in management should not confuse an entrepreneurial impulse to take bold risks and make quick decisions with sound engineering practice. Academic course work in engineering management should emphasize the importance of management decisions in preventing accidents. Engineers and their superiors should be aware of the five hazardous attitudes and learn to recognize symptoms of flawed thinking and false priorities. Engineering managers should also learn to control the risky personalities and situations that can produce unsatisfactory results. Teamwork within the engineering office and reasonable workloads on the staff should be emphasized over the desires of management to redeploy personnel among projects. Those who check engineering work must remember that structural systems must always satisfy fundamental principles of engineering. While prior work can be instructive, the only measure of acceptability should not be a comparison to previous designs.

An important success of the human factors approach to commercial aviation safety has been the emergence of the Aviation Safety Reporting System (ASRS). This confidential, voluntary and non-punitive program permits any person at any level of responsibility in the industry to report errors, weaknesses and problems. The ASRS encourages self-reporting of incidents that do not result in accidents or constitute violations of the law. Aviation industry workers have such confidence in the system that they make 50,000 reports, often about themselves, each year. Regularly published summaries are made available to the aviation community. There is presently a database of over 500,000 publicly accessible reports compiled since 1976.[31] In 2005, the International Association of Fire Chiefs instituted the National Fire Fighter Near-Miss Reporting System, modeled on the ASRS and employing principles of HFACS. This system has already generated recommendations for reducing fire fighter fatalities.[32] Given the importance of constructed facilities and the high rate of injury in construction work, the design and construction industry could benefit enormously from a similar non-punitive reporting system.

Conclusion

The human error that brings about most structural failures is far less sinister and far more pervasive than most engineers believe. Engineering firms need more than after-the-fact assignments of blame to human shortcomings; they also need the tools with which to recognize and confront the mistakes that people really make. Improved management of human error in structural engineering should be regarded as a necessary way to assure the sound performance of structural works and the safety of the public. Among practitioners, consideration and implementation of human factors methods also offer the opportunity for discussion of profoundly important issues. Such a profession-wide debate should examine the quality of technical information now in circulation, the effectiveness of organizational management, the interactions between engineers and computers, and the need to collect useful data about error and accidents from a broader spectrum of design and construction sources. Other industries have demonstrated that real progress in reducing error comes from a solid understanding of human limitations, not from attorneys or insurance companies. They have shown that the path to safer engineering practice is wide enough for both technical rigor and professional dignity.

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Application of Modified Compression Field Theory for Shear Design in the AASHTO LRFD Code

There are benefits and drawbacks to using the new AASHTO bridge design method to calculate the shear capacity of concrete instead of the traditional design method.

JASON VARNEY

Over the years, engineers have performed a great deal of research and experimentation in an attempt to accurately qualitatively and quantitatively describe the behavior and failure mechanisms of concrete in shear. Unlike materials such as steel, the non-homogeneity and inelasticity of concrete as a building material makes this behavior very difficult to quantify, and modern concrete design methods are continually being revised and reworked in order to better represent the true behavior of the material when subjected to shear forces. The American Association of State Highway and Transportation Officials (AASHTO) load and resistance factor design (LRFD) bridge design code has incorporated a new approach for analyzing and designing for shear in concrete bridges (issued as *AASHTO LRFD Bridge Design Specifications, Fourth Edition*[1]). This new approach is based on the modified compression field theory (MCFT).

Although the exact behavior of concrete will likely never be fully resolved, newer, more involved theories (such as MCFT) are being adopted within the engineering world in order to more accurately model the true shear strength of concrete. It remains to be seen whether these emerging theories provide more advantages in the economics of concrete design than disadvantages in their calculation.

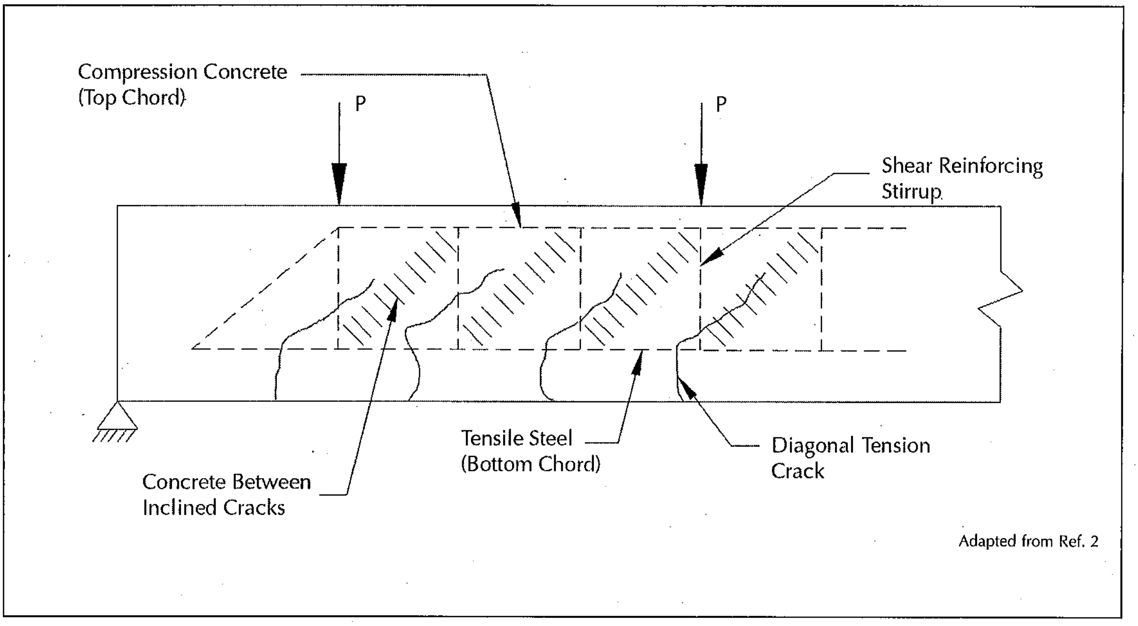

FIGURE 1. Truss model of axial shear forces in a reinforced concrete beam.

Traditional Method Derivation

Before adoption of the new MCFT approach to the AASHTO LRFD bridge design code, engineers used the "traditional" method to design reinforced concrete bridges for shear. This traditional approach to concrete shear analysis incorporates the assumption that the nominal shear capacity of a reinforced concrete section, V_n, is composed of two components:

- the resistance provided by the concrete, V_c; and,
- the resistance provided by the steel reinforcement stirrups, V_s.

The contributing forces can be summed up with a simplified truss model in which axial shear is resolved into compression and tension struts, with the concrete taking compression and the steel stirrups taking tension. This truss analogy can be seen in Figure 1, along with a schematic of beam components and dimensions in Figure 2.

The variables in Figure 2 are defined as follows:

s = spacing of shear stirrups
s_1 = crack length
α = stirrup angle
β = crack angle
C = compressive force resultant
T = tensile force resultant
d = distance from top face to tensile steel

From the given geometry, it can be determined that:

$$V_s = T_s{}^*sin(\alpha) \quad \textbf{(1)}$$

where:
T_s = the force resultant of all web stirrups across the diagonal crack, and
α = the angle of the shear stirrups relative to the horizontal.

If n is equal to the number of stirrup spacings within crack length s_1, then:

$$s_1 = n^*s = jd^*[cot(\alpha) + cot(\beta)] \quad \textbf{(2)}$$

where:
β = the angle of the shear crack.

Dividing the stirrup force by the crack length, T_s/s_1, and approximating j equal to 1, substitutions to Equation 2 can be made in order to obtain:

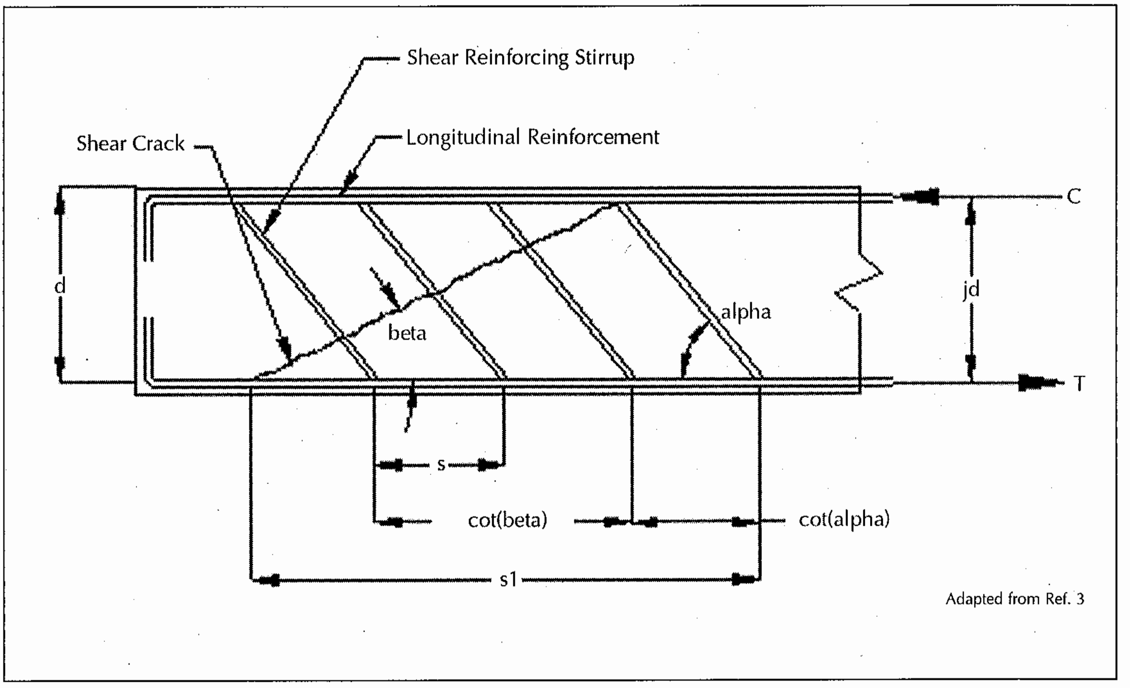

FIGURE 2. Schematic of a cracked shear-reinforced concrete beam.

$$(V_s/sin(\alpha))/(d*(cot(\alpha)+cot(\beta))) \quad (3)$$

Given n stirrups over crack length s_1, the total force resisted by the stirrups must equal T_S, or:

$$T_s = n*A_v*f_y \quad (4)$$

where:
A_v = the area of a single shear stirrup, and
f_y = the yield strength of the steel.

With these relationships, the following can be inferred:

$$n*A_V = T_s/f_y = (V_s*n*s/sin(\alpha))/(d*(cot(\alpha)+cot(\beta))*f_y) \quad (5)$$

By rearranging Equation 5, it can be restated as:

$$V_s = (A_v*f_y*d/s)*(cot(\alpha)+cot(\beta)) \quad (6)$$

The traditional model makes two simplifying assumptions as shown in Figure 3. First, the traditional approach assumes that shear cracks form at an angle, β, of 45 degrees to horizontal. Second, the stirrups are assumed to be vertical; therefore, α is equal to 90 degrees. If β is equal to 45 degrees:

$$V_s = (A_v*f_yd/s)*(sin(\alpha))*(1+cot(\alpha))$$
$$= A_v*f_y*d/s)*(sin(\alpha)+cos(\alpha)) \quad (7)$$

Equation 7 can be simplified further by setting α to 90 degrees, and

$$V_s = (A_v*f_y*d/s) \quad (8)$$
$$s = (A_v*f_y*d)/V_s = (A_v*f_y*d)/(V_u/\varphi-V_c) \quad (9)$$

where:
V_u/φ = the required resistance; and,
$V_c = (2\sqrt{f'_c})*b_w*d$ (concrete shear strength determined by experimentation).

Using $(2\sqrt{f'_c})*b_w*d$ to determine concrete shear strength, it is possible to determine if a concrete beam or slab is strong enough to resist shear without reinforcement. If it is not, then the equation for V_s, the capacity provided by the steel stirrup reinforcement, comes into play. The design approach is to select a shear stirrup size, A_v, and solve for the spacing of the stirrups, s.

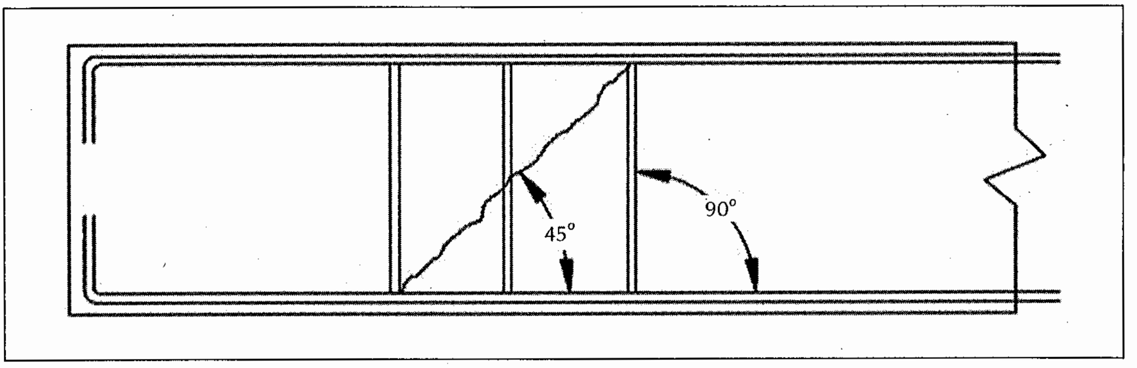

FIGURE 3. Traditional design schematic with simplifying assumptions.

As is evident through this derivation, the traditional model for shear design in concrete assumes that no tensile stress is transferred across the shear cracks and all tensile forces are taken by the tensile steel and vertical stirrups. While these assumptions greatly simplify the process of designing for shear in concrete bridge design, their validity has been challenged in recent years.

Modified Compression Field Theory Derivation

Recent experimentation has determined that the simplifying assumptions presented in the traditional method of concrete shear design are very conservative. The shear crack angle is variable. Also, counter-intuitively, it has been proven possible for tensile stresses to be transferred across these shear cracks, which happens due to the non-homogenous nature of concrete. The aggregate tends to "lock up" as the planes of the crack slip in opposite directions. As long as the axial strain of the member (ε_x) is kept to a minimum, the section remains whole and tension is, in fact, transferred across the diagonal shear cracks. This minimal axial strain is obtained by ensuring that the crack size is limited, allowing the aggregate particles to interlock and carry tension. Figure 4 depicts the pre-cracked condition of concrete, with the principal stresses f_1 and f_2 of the stress element equal to each other (a); the idealized concrete beam present in the traditional model of shear design, where θ is equal to 45 and f_1 is non-existent (b); and the idealized concrete beam present in the modified compression field theory, where θ is less than 45 and f_1 is not equal to 0 (c). Figure 5 shows the resulting schematic of forces and stresses in a cracked beam, according to MCFT.

Summing the vertical forces in Figure 5a yields:

$$\Sigma F_v = V = f_2 b_v d_v \cos(\theta)\sin(\theta) + f_1 b_v d_v \sin(\theta)\cos(\theta) \quad \textbf{(10)}$$

where:

f_1 = the average tensile stress across the concrete and d_v is the effective shear width, taken as the distance between the resultants of the tensile and compressive forces due to flexure.

If v is equal to the average shear stress across concrete, then:

$$v = V/(b_v d_v) \quad \textbf{(11)}$$

$$v b_v d_v = f_2 b_v d_v \cos(\theta)\sin(\theta) + f_1 b_v d_v \sin(\theta)\cos(\theta) = (f_2+f_1)\sin(\theta)\cos(\theta) \quad \textbf{(12)}$$

Solving for f_2:

$$f_2 = (v/\sin(\theta)\cos(\theta)) - f_1 \quad \textbf{(13)}$$

If the force in the stirrups is considered, summing the vertical forces in Figure 5b yields:

$$\Sigma F_v = A_v{}^*f_v = f_2{}^*s{}^*b_v{}^*\sin^2(\theta) - f_1{}^*s{}^*b_v{}^*\cos^2(\theta) \quad \textbf{(14)}$$

By substituting and simplifying, the following relationship can be obtained:

$$V = (A_v f_v d_v/s)cot(\theta)+f_1 b_v d_v cot(\theta) \quad \textbf{(15)}$$

This result can be compared to the relationship given by the traditional shear derivation:

$$V = (A_v f_v d_v/s)cot(\theta) \quad \textbf{(16)}$$

where:
$cot(\theta) = cot(45) = 1$

The difference between the two theories is the $f_1 b_v d_v cot(\theta)$ term, which represents the force due to tension across the shear cracks.

Tension can exist in the modified compression field only if slippage across the cracks is limited. Figure 6 illustrates the tensile forces across a crack, where (a) shows a beam web cracked by shear, (b) shows the average stresses between cracks and (c) shows the local stresses at a crack. The local value of shear stress at the crack, v_{ci}, can be determined by:

$$v_{ci} \leqslant (0.0683\sqrt{f'_c})/(0.3+24w/a_{max}+0.63) \quad \textbf{(17)}$$

where:
v_{ci} = the local value of shear stress at the crack,
a_{max} = the maximum aggregate size, and
w = the crack width.

Reverting to Equation 15, f_1 can be taken to be equal to the average tension stress (see Figures 5 & 6), thus:

$$f_1 = v_{ci}tan(\theta) \leqslant (0.0683\sqrt{f'_c})tan(\theta)/(0.3+24w/a_{max}+0.63) \quad \textbf{(18)}$$

AASHTO prefers to express Equation 18 as:

$$f_1 = v_{ci}tan(\theta) \leqslant (\mathbf{2.16*0.0316}\sqrt{f'_c})tan(\theta)/(0.3+24w/a_{max}+0.63) \quad \textbf{(19)}$$

where:
0.0316 = a conversion factor from psi to ksi (for f'_c) to accommodate AASHTO's preference for f'_c in ksi, in contrast to the ACI approach for f'_c to be in psi.

Thus, the expression for the nominal shear

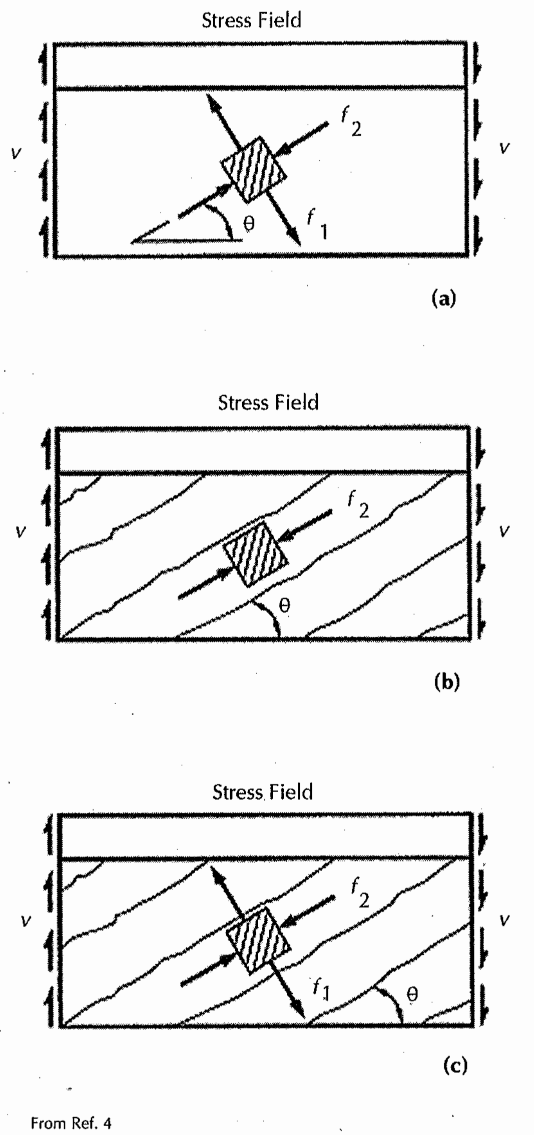

FIGURE 4. Comparison of stress fields in a concrete beam subjected to shear.

strength of a reinforced concrete beam becomes:

$$V = (A_v f_v d_v/s)cot(\theta)+(2.16*0.0316\sqrt{f'_c})/[(0.3+24w/a_{max}+0.63)b_v d_v] \quad \textbf{(20)}$$

If a new variable, β, is defined as $2.16\sqrt{f'_c}(0.3+24w/a_{max}+0.63)$, Equation 20 simplifies to its final form:

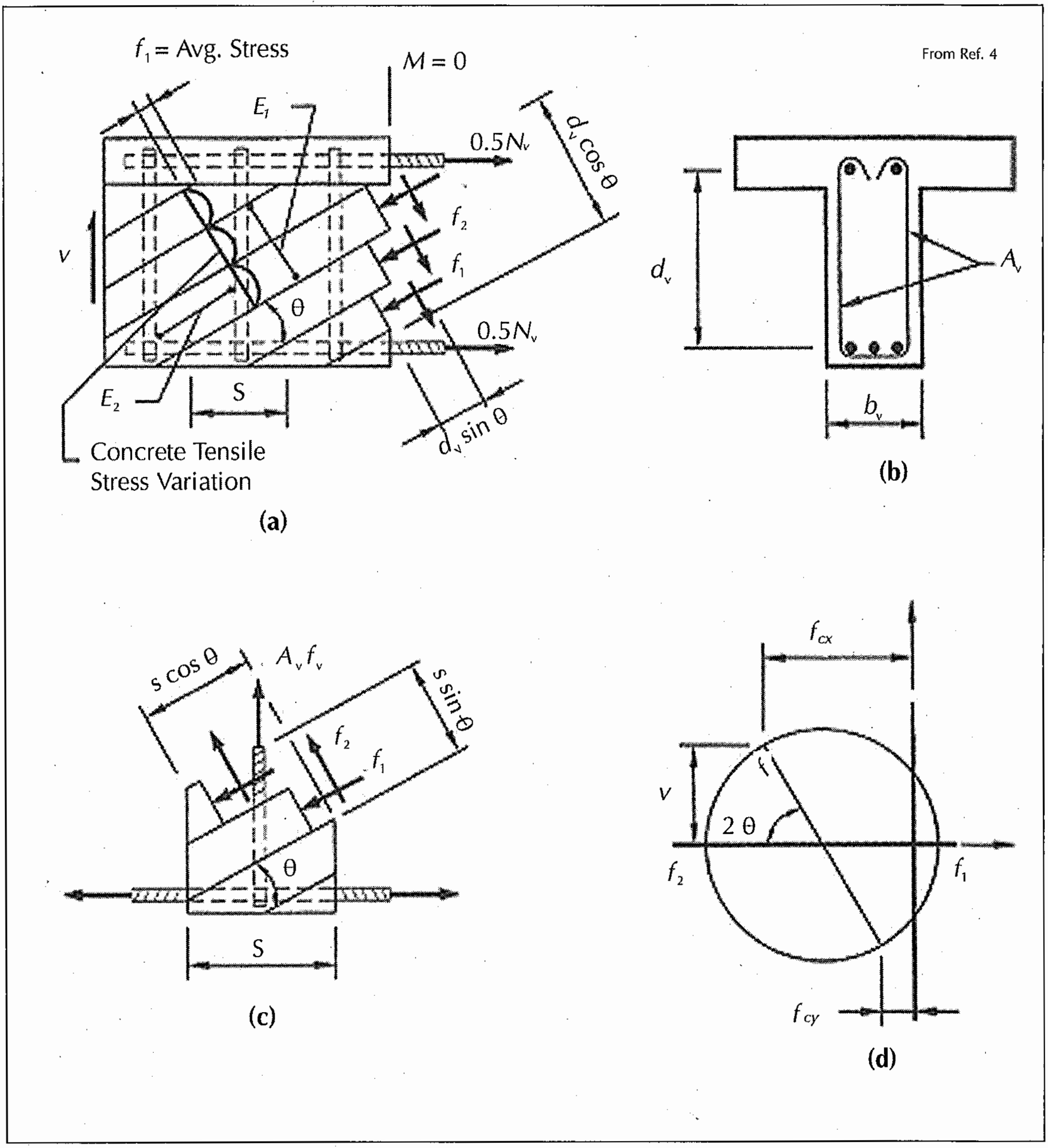

FIGURE 5. Schematic of forces and stresses in a concrete beam subjected to shear.

$$V = (A_v f_v d_v / s)\cot(\theta) + 0.0316\beta\sqrt{f'_c}(b_v d_v) \quad \textbf{(21)}$$

Note that for θ equal to 45 degrees and β equal to 2, this expression becomes the same as the traditional ACI method of shear calculation.

Numerical Comparison of Traditional & MCFT Methods

Values of V_s, the required shear resistance of the added steel stirrups, and s, the required spacing of stirrups to provide this resistance, were calculated for the example shown in Figure 7 using both the traditional model and the MCFT method. Results of these calculations are summarized in Table 1.

The problem statement assumes the following:

V_u = 157 kip
M_u = 220 kip-ft

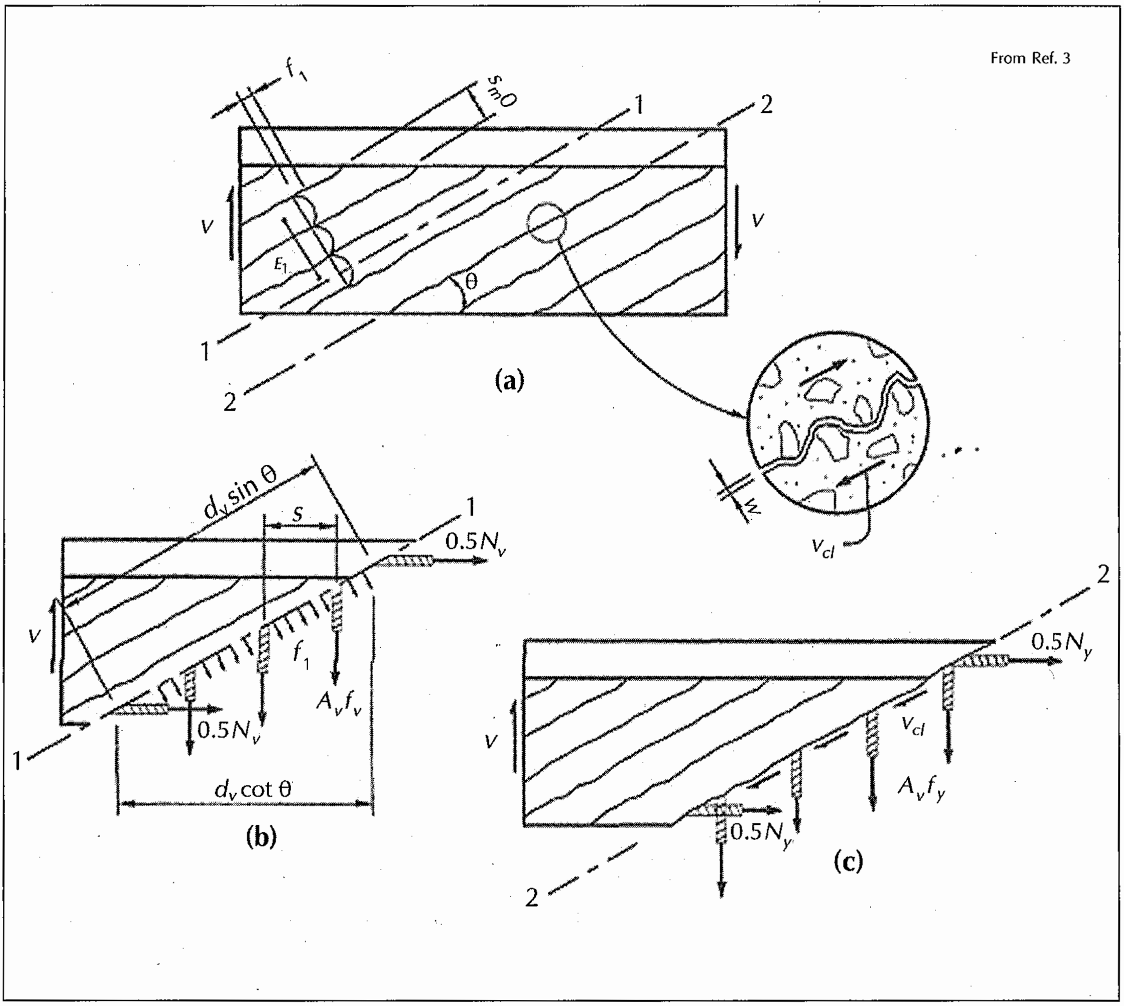

FIGURE 6. Tensile forces across a crack.

$f'_c = 4{,}500$ psi
$f_y = 60{,}000$ psi

Interestingly, the required shear resistance of the concrete, V_s, is 8.3 percent greater using MCFT. In effect, the modified "cost-saving" method counter-intuitively delivers a concrete beam section that has a lower nominal shear resistance. However, the shear strength of the concrete is not the governing factor in this calculation.

Regarding the required stirrup spacing (the real result of interest), a much more distinct difference between the two methods can be observed. The modified compression field theory model predicts a required stirrup spacing of 7.73 inches, 25.7 percent greater than the traditional model prediction of 6.15 inches. Although this larger spacing may not seem possible because the modified method requires a larger shear resistance, the treatment of the crack angle (β in the traditional method and θ in the modified method) in each derivation should be recalled. Both calculations have a *cot(crack angle)* multiplier at the end of the calculation. However, unlike the traditional method, which assumes a conservative crack angle of 45 degrees for simplification to make *cot(45)* equal to 1, MCFT assumes a variable crack angle.

This variable crack angle (always taken to be less than 45 degrees) turns the *cot(β)* term into a multiplier, rather than a disappearing

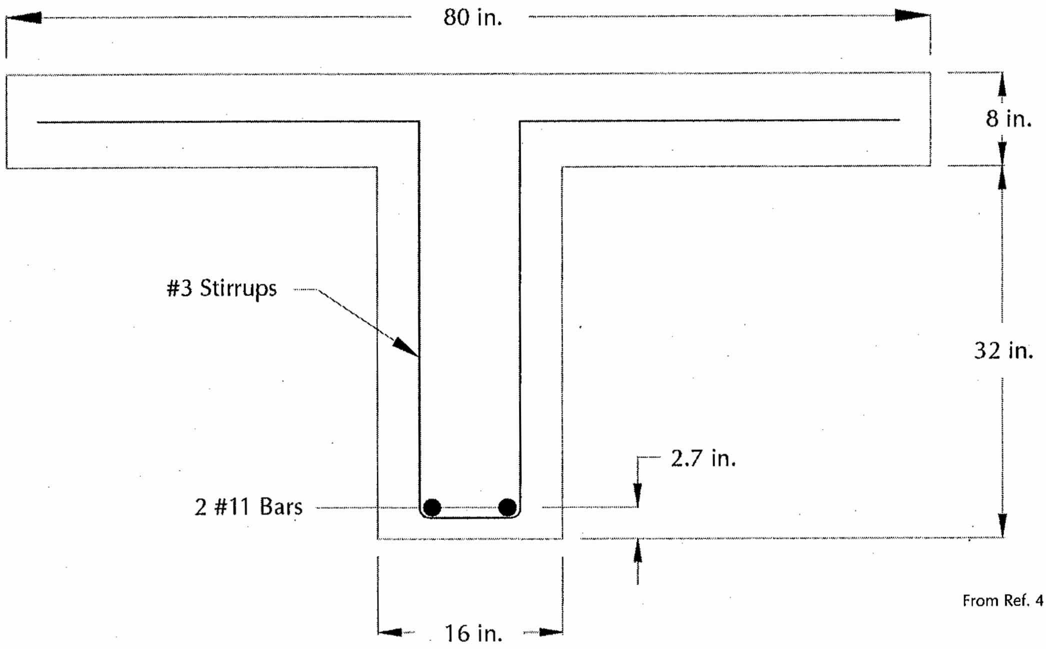

FIGURE 7. Numerical example for comparison of traditional and modified methods.

term. To illustrate this point, the preceding example, in which iteration ultimately predicts the MCFT crack angle, θ, to be 36.4 degrees. The multiplier becomes *cot(36.4)*, or 1.36. Although MCFT predicts an 8.3 percent decrease in nominal shear resistance, the variable crack angle compensates for this loss with a 36 percent increase in the crack spacing calculation over the traditional method, and a resulting 25.7 percent increase in overall stirrup spacing.

To put this calculation into perspective, using the parameters and dimensions given in the preceding example, by the modified compression field approach a 25-foot beam requires approximately 80 percent of the shear reinforcement steel required by the traditional approach.

Iteration & Automation of MCFT

Application of the traditional method for

TABLE 1.
A Comparison of Numerical Example Results

	Traditional Model	Modified Compression Field Theory	Percent Difference
Required V_s	79.40 kip	85.96 kip	8.3
Required s	6.15 in	7.73 in	25.7

analysis and design is relatively straightforward. Unfortunately, as can be inferred by the comparison presented in the previous section, application of the new AASHTO method based on MCFT is not. It requires an iterative process in which different angles of θ are calculated and then tested against a group of assumptions summarized by the new variable, β (not to be confused with the crack angle, β, in the traditional method). These simplifying assumptions, developed by Collins and Mitchell, relate to factors of horizontal strain (or a maximum crack width), which incorporate a maximum aggregate size of 0.75 inches and a maximum crack spacing of 12 inches.[5]

An automated tool was developed to simplify the process of selecting β. This tool, which is programmed into the Visual Basic editor in a Microsoft Excel spreadsheet, requires the user to enter an estimate of θ (crack angle) and then calculates the resulting axial strain, ε_x. The spreadsheet then performs several iterations using the table found in AASHTO Figure 5.8.3.4.2-1 to obtain values of β and θ (see Table 2).[6]

The modified shear design process mirrors that of the traditional method from this point on. The difference is that in the traditional method, β is assumed equal to 2 and θ is equal to 45 degrees, whereas the modified method uses a more conservative higher value of β, and a less conservative, lower value of θ. The difference in crack angle overcompensates for the higher value of β, ultimately providing a higher required shear stirrup spacing.

Parametric Study Comparing Different Beam Designs

The required spacing of shear reinforcement obtained from both the traditional design method was compared to the new MCFT design method specified by AASHTO for a variety of beam geometries and distributed loads. Standard beam cross-sections were selected for beam lengths of 30, 40, 50 and 60 feet, and shear reinforcement spacings were calculated using loads of 0, 1,000, 2,000, 3,000, 4,000, 6,000, 7,500 and 10,000 pounds per foot (lbs/ft). The study assumed 60 ksi steel, 4 ksi concrete and #3 bars for shear reinforcement (this assumption may not always be realistic, but for theoretical considerations, the resulting numbers are comparable).

For loads up to 3,000 to 4,000 lbs/ft (depending on beam size), maximum spacing requirements governed the reinforcement spacing. Because the reinforcement spacing does not depend on the method of design for these cases, but rather AASHTO requirements based on beam geometry, the resulting spacings were omitted from this study.

The results of the remaining cases displayed a significant variation between the traditional and MCFT methods of design in shear bar spacing. The stirrup spacings calculated using the newer, more complex MCFT method of design were an average of 107 percent larger than those calculated with the traditional method of design for those cases not controlled by maximum spacing requirements. In this study, shear spacing calculated at the point of maximum shear along the beam was evaluated. The results using MCFT suggest significant potential savings in shear stirrups due to increased bar spacing. Maximum allowable spacing will probably be found to govern the design in more locations than when using the traditional design method. The potential materials savings make this method attractive from an economic standpoint, assuming the costs associated with engineering efforts are comparable among the two design methods.

Although there is a significant contrast between the reinforcement spacings obtained using the new AASHTO LRFD method compared to the traditional design method, the values for β and θ obtained using the new method did not vary significantly among themselves. Possible values of θ range from 18.1 to 37.3 degrees, yet those in the parametric study ranged from only 26.6 to 32.7 degrees. Similarly, possible values of β range from 1.50 to 6.32, yet those in the study ranged from only 1.86 to 2.94.

Although a more extensive study could be used to obtain a greater variation in results, the relative uniformity of variables in the results of this study suggest that conservative near-average values of β and θ could be substituted into the equation to simplify the itera-

TABLE 2.
Values of θ & β for Sections With Transverse Reinforcement

$V_u/f'c$	$\varepsilon_x \times 1000$								
	≤ −0.20	≤ −0.10	≤ −0.05	≤ 0	≤ 0.125	≤ 0.25	≤ 0.50	≤ 0.75	≤ 1.00
≤ 0.075	22.3 6.32	20.4 4.75	21.0 4.10	21.8 3.75	24.3 3.24	26.6 2.94	30.5 2.59	33.7 2.38	36.4 2.23
≤ 0.100	18.1 3.79	20.4 3.38	21.4 3.24	22.5 3.14	24.9 2.91	27.1 2.75	30.8 2.50	34.0 2.32	36.7 2.18
≤ 0.125	19.9 3.18	21.9 2.99	22.8 2.94	23.7 2.87	25.9 2.74	27.9 2.62	31.4 2.42	34.4 2.26	37.0 2.13
≤ 0.150	21.6 2.88	23.3 2.79	24.2 2.78	25.0 2.72	26.9 2.60	28.8 2.52	32.1 2.36	34.9 2.21	37.3 2.08
≤ 0.175	23.2 2.73	24.7 2.66	25.5 2.65	26.2 2.60	28.0 2.52	29.7 2.44	32.7 2.28	35.2 2.14	36.8 1.96
≤ 0.200	24.7 2.63	26.1 2.59	26.7 2.52	27.4 2.51	29.0 2.43	30.6 2.37	32.8 2.14	34.5 1.94	36.1 1.79
≤ 0.225	26.1 2.53	27.3 2.45	27.9 2.42	28.5 2.40	30.0 2.34	30.8 2.14	32.3 1.86	34.0 1.73	35.7 1.64
≤ 0.250	27.5 2.39	28.6 2.39	29.1 2.33	29.7 2.33	30.6 2.12	31.3 1.93	32.8 1.70	34.3 1.58	35.8 1.50

From Ref. 6

tion process of the new AASHTO LRFD shear design method.

Discussion of NCHRP Report 549 & Proposed Code Changes

In 2005, the National Cooperative Highway Research Program (NCHRP) released Report 549, entitled *Simplified Shear Design of Structural Concrete Members*.[7] The report provides an overview of a research program conducted by the NCHRP that attempted to develop practical equations for the design of shear reinforcement, specifically focused on reinforced and prestressed concrete bridge girders. Report 549 provides several recommendations for the improvement of the existing shear reinforcement design method outlined by AASHTO, with the intention that these improvements would be considered for the 2007 AASHTO Bridge Design Specifications.

The NCHRP research began by reviewing the "structure and underlying bases" of several well-known methods of calculating shear capacity. These methods included old ACI and AASHTO methods, as well as present and past international methods of calculation. The most significant of these methods turned out to be the Canadian Standards Association (CSA) Code for the Design of Concrete Structures, CSA A23.3-04.[8] The CSA A23.3-04 design method is based on the same principles as the new AASHTO method, but it is far less cumbersome since it provides simple methods for calculating β and θ. To calculate β for members with A_v less than $A_{v,\,min}$:

$$\beta = [4.8/(1+1500\varepsilon_x)]*[51/(39+s_{xe})] \quad \textbf{(22)}$$

To calculate β for members with A_v greater than or equal to $A_{v,\,min}$ (note that s_{xe} equals 12 inches):

$$\beta = 4.8/(1+1500\varepsilon_x) \quad \textbf{(23)}$$

To calculate θ:

$$\theta = 29+7000\varepsilon_x \tag{24}$$

After conducting a series of field experiments, the researchers concluded that both the CSA method and the new AASHTO LRFD method of shear reinforcement design were the most accurate of all of methods surveyed, and had only approximately a 10 percent chance of being unconservative.[7] With the preceding equations in mind, it is easy to see the advantages to the CSA method of design compared to the cumbersome AASHTO iterative method of determining β and θ.

Additionally, state departments of transportation and bridge designers were surveyed regarding the new MCFT method of shear reinforcement design in comparison to the traditional method of design.[7] Of the findings, the most significant were that, in general, bridge designers had little experience using the new AASHTO LRFD shear design specifications, and that everyone in the profession agreed that the new provisions must be computer-automated if AASHTO is going to require their use. Unfortunately, the implications of both the use and automation of the new method include the engineers' loss of an intuitive sense of the design, and subsequently their comfort in carrying out the required calculations.

Upon completion of this project, the NCHRP researchers produced a series of recommendations to improve the current AASHTO LRFD Bridge Design Specifications.[7] Among these recommendations was a new method of designing shear reinforcement, which was a modification of the existing method that incorporated the CSA A23.3-04 method of design. These modifications would make the currently required calculation much simpler, and still provide a conservative yet efficient design procedure for reinforced and prestressed concrete bridge girders in shear.

At its 2007 annual meeting, the AASHTO Subcommittee on Bridges and Structures adopted Agenda Item 34 (among others), as a 2008 interim change to the 2007 AASHTO LRFD Bridge Design Specifications.[9] This item was a result of the NCHRP report, and introduces a more general method of calculating β and θ. Agenda Item 34 presents equations that allow for the direct solution of β, and suggests that θ be taken as 30 degrees in all cases, making the new method non-iterative. These changes, if permanently adopted, would greatly increase the efficiency of the MCFT design method. Although this method is attractive from an economic standpoint, it may take years before the new design method makes intuitive sense to those who use it, and is fully accepted by design engineers.

Conclusions

The rationale behind the new AASHTO LRFD method of designing for shear is well-founded. Its use of MCFT provides a more accurate representation of the true shear strength of reinforced and prestressed concrete beams since the assumptions made in the traditional model of derivation are very conservative. As a result, the new AASHTO method affords the designer reinforcement cost-savings, as well as the opportunity to allow beams to carry more load and span further distances than previously recommended under the traditional design method.

Despite these benefits, however, the disadvantages to the AASHTO method far outweigh its advantages. The new method of design is cumbersome and does not make immediate intuitive sense. As a result, the design process is lengthy and confusing, unlike the more traditional design method. Engineers find it difficult to perform quick mental calculation checks because of the lack of intuition involved in the calculation, and this fact in itself could jeopardize the safety of structures designed under the new provisions.

In response to these difficulties, it is recommended that:

- The next edition of the AASHTO LRFD Bridge Design Specifications should use modified provisions for designing for shear that incorporate the CSA A23.3-04 method of design, as outlined in NCHRP Report 549, and proposed by the AASHTO Subcommittee on Bridges and Structures.

- All concrete codes should work to incorporate these modified design provisions in order to increase the accuracy of design, provide greater simplicity in calculation, reduce costs, and create uniformity across the concrete design field.
- Until these changes are adopted, software that automatically performs the necessary iterations to obtain values for β and θ, similar to the spreadsheet developed herein, should be distributed and utilized by concrete bridge designers to lessen the present complexity of the shear reinforcement design process.

MCFT is a great advancement in the field of concrete design, but the methods of design must strike a balance between utilizing the accuracy of the theory and ensuring efficient design for engineers. If these recommendations are adopted and implemented, both will be obtained and the bridge design process will be more efficient, economical and safe.

ACKNOWLEDGMENTS — *This article summarizes the outcome of a research project at Tufts University performed by the author under the direction of faculty advisor Professor Brian Brenner.*

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The Choate Bridge

A greater awareness of the historic significance of this bridge within the engineering community is integral to its future survival.

EMMA FRANCIS & JULIA CARROLL

In 1764, Colonel John C. Choate, Esquire, oversaw the construction of the Choate Bridge in the town of Ipswich, Massachusetts. The Choate Bridge is located along Route 1A and, with the exception of the 1838 widening of the bridge, it has remained nearly unaltered since the time of its construction. The original construction was 80.5 feet long and 20.5 feet wide. In 1838, the bridge was widened to 35.5 feet to accommodate another lane of traffic.

The Choate Bridge (see Figure 1) is the earliest documented masonry arch bridge and the oldest extant bridge in Massachusetts. It is also the second documented masonry arch bridge and the second oldest extant bridge in the United States. The bridge has been in continuous use since its original construction and currently carries heavy commercial truck traffic and an estimated 20,000 vehicles per day.

The Choate Bridge is modeled in the same style as Roman arch bridge construction. The Frankford Avenue Bridge, erected in 1697 in Philadelphia, Pennsylvania, was the first masonry arch bridge in the United States. Unlike the Frankford Avenue Bridge, which has undergone significant structural alterations over its life, the Choate Bridge has merely had its mortar joints repointed and only been subjected to routine parapet maintenance since its 1838 widening.

This landmark serves as a complete example of mid-eighteenth century bridge technology, particularly the use of random-coursed ashlar and dry stone wall construction. Furthermore, as seen in Figure 2, the adjacent arrangement of the 1764 and 1838 spans juxtaposes two distinct eras of masonry arch bridge construction. Of particular note are the differences in stone size and shape, demonstrating the advances in stone-cutting technology.

The Choate Bridge has served as a major transportation route since 1764. It is commonly believed to have served as a thoroughfare during the Revolutionary War, and is depicted as such in a nearby mural (see Figures 3 & 4). With its original construction intact, it is an authentic example of early stone construction in the history of American road building. The significance of this structure in both the historical and engineering arenas has already been demonstrated through its recognition by the Historic American Engineering Record and a listing in the National Register of Historic Places, and through designation as a Massachusetts

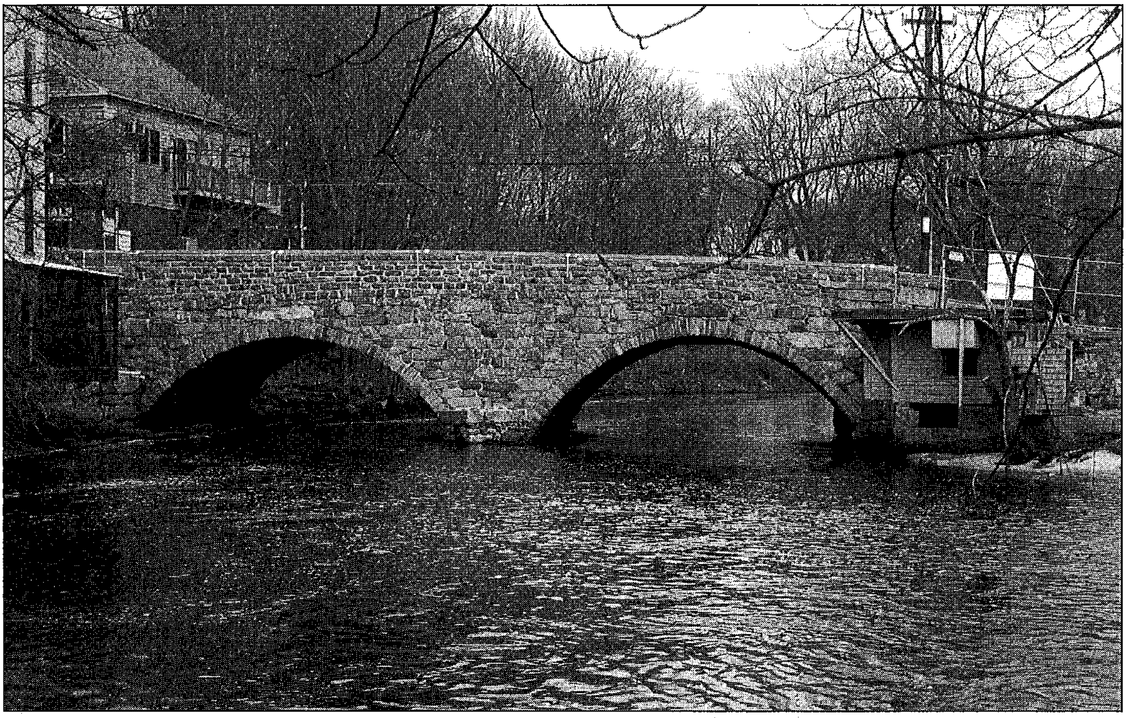

FIGURE 1. Overview of the Choate Bridge from the west.

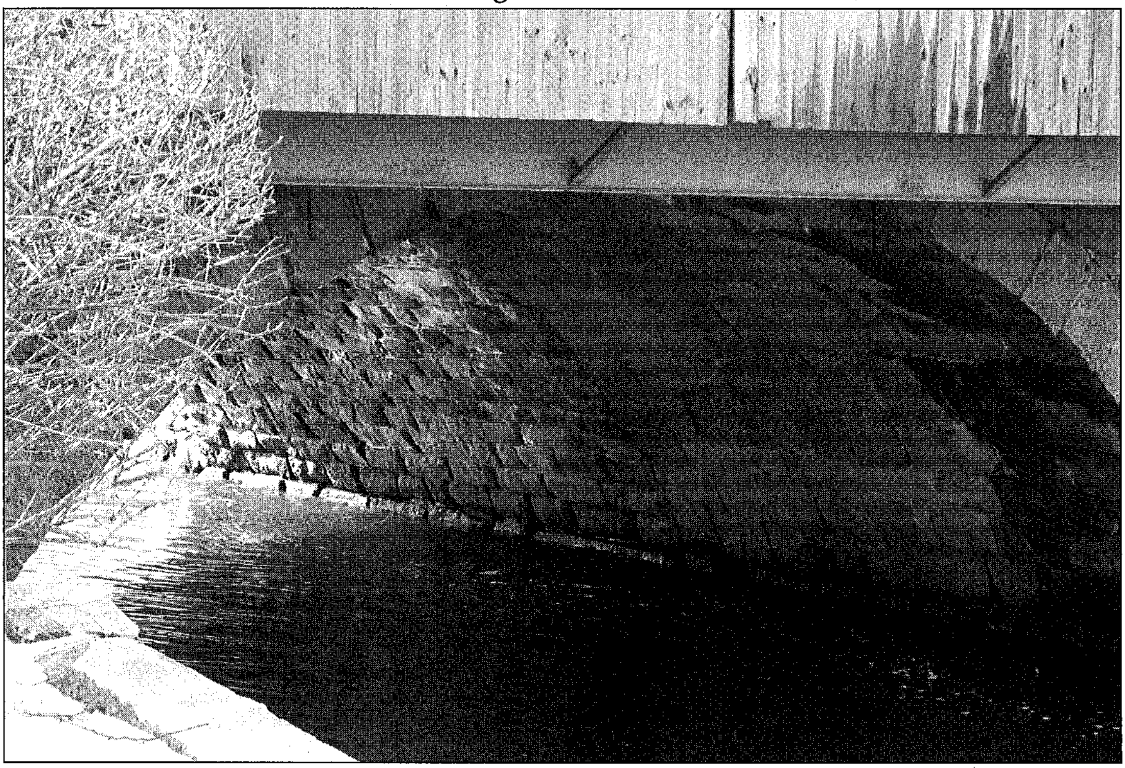

FIGURE 2. The bridge's southern arch from the east side. Note the distinction between the random-coursed ashlar and dry stone wall construction.

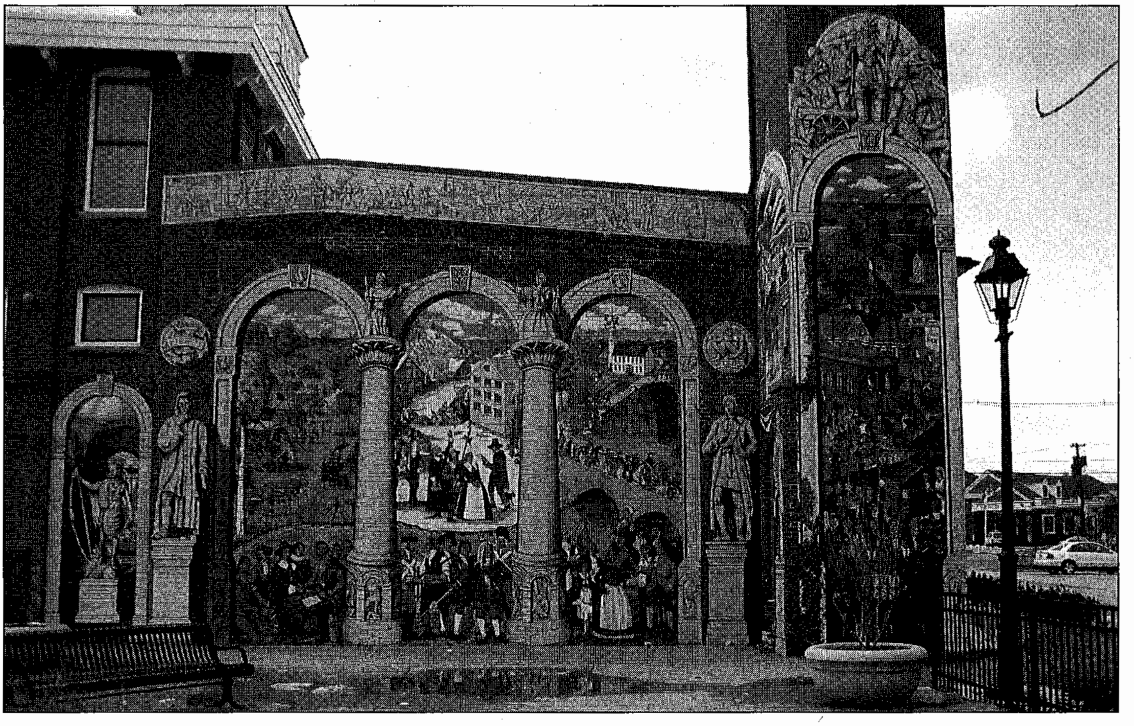

FIGURE 3. Mural depicting historical Ipswich, adjacent to Ipswich River and the Choate Bridge.

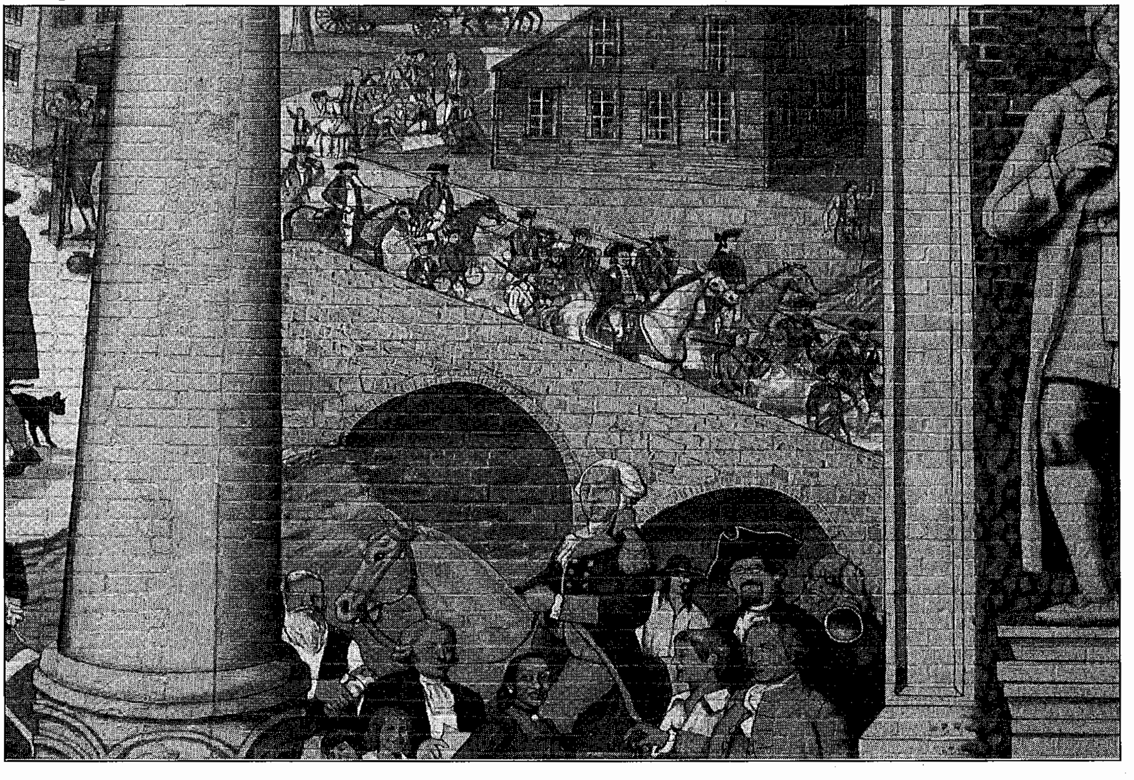

FIGURE 4. Close-up of the Choate Bridge mural showing its use during the Revolutionary War.

Historic Landmark by the Massachusetts Historical Commission.

Though the time-tested nature of its construction and structural form has ensured its longevity, the bridge has already been exposed to natural weathering degradation. In June 2006, the Choate Bridge was closed for 27 days due to scour of the piers, greatly compromising the economy of the nearby businesses. The town of Ipswich has made a great effort to preserve this landmark, but future continued success depends on national recognition by the greater engineering community. In the interests of garnering the attention of the engineering community and the continued preservation of the Choate Bridge, Julia Carroll and Emma Francis, in association with the Boston Society of Civil Engineers Section ASCE (BSCES), prepared the nomination packet for the History and Heritage Committee of the American Society of Civil Engineers (ASCE) to recommend the Choate Bridge for recognition as an ASCE National Historic Civil Engineering Landmark on September 19, 2007. The nomination was approved by the ASCE Board of Direction on May 2, 2008. There will be a formal dedication ceremony at a date to be determined, and a bronze plaque will adorn the bridge.

ACKNOWLEDGMENTS — *A.G. Lichtenstein and Associates worked as a historic preservation consultant on the restoration of the Choate Bridge in the 1990s. PTAI served as the official engineer/architect during the 1990s restoration.*

EMMA FRANCIS *is an associate with Wiss, Janney, Elstner Associates in Cambridge, Mass. She has worked in repair and preservation for seven years and will obtain her graduate degree in Structural Engineering from Tufts University in 2009. Notable projects in which she has participated include the Omni Parker House exterior envelope repair in Boston, Mass.; the historic Mason Temple restoration in Providence, Rhode Island; the Bank of America make-safe annual inspections in Providence, Rhode Island; and the Leonard P. Zakim Bridge Bunker Hill Memorial girders investigation in Boston, Mass.*

JULIA CARROLL *received her B.S. in Civil and Environmental Engineering from Carnegie Mellon University in 2005. Having conducted research in bridge design history, engineering education and sustainable building practices, she will receive her M.Eng. in Structural Engineering from Tufts University in 2008. In October, she will join DMJM Harris as a Structural Engineer I.*

Gephyrophobia

The innate beauty of bridges — both from design and appearance — should inspire awe and not heart-racing anxiety.

BRIAN BRENNER

Everyone is afraid of something, even engineers. But with a slew of things that rightfully inspire fear comes one issue that engineers have a hard time acknowledging: the fear of bridges. For those who have no conception of this phobia, here is a medical definition:

"An abnormal and persistent fear of bridges, especially crossing bridges. Sufferers of this phobia experience undue anxiety even though they realize their fear is irrational. Their fear may result partly from the fear of enclosure (claustrophobia) or the fear of heights (acrophobia). Phobic drivers may worry about being in an accident in busy traffic or losing control of their vehicles. High bridges over waterways and gorges can be especially intimidating, as can be very long or very narrow bridges.

"Fear of bridges is a relatively common phobia although most people with it do not know they have something called *gephyrophobia*. However, the derivation of the word *gephyrophobia* is perfectly straightforward (if you know Greek); it is derived from the Greek words *gephyra* (bridge) and *phobos* (fear)."[1]

Fear of bridges? Gephyrophobia? How could anyone be afraid of a bridge?

Two bridges come to mind that maybe could explain why some people have an aversion to these structures. Figure 1 shows a picture of the first Tacoma Narrows Bridge. This bridge, as everyone knows, danced in the wind and has now served as the primary subject of a film loop displaying the dynamic effects of structures in the wind for generations of engineering students (see Figure 2). Figure 3 shows a picture of the first Sunshine Skyway Bridge, which crosses Tampa Bay. This bridge tanked after being hit by a barge. Figure 4 shows the rebuilt Sunshine Skyway.

All right, maybe it is possible to be afraid of bridges. Although it seems like being afraid of pizza would make more sense. Overall, bridges are pretty safe. The odds are greater that you will choke on a piece of pizza than you will drive off a bridge. Also, a pizza with extra cheese can burn your tongue and throat. Even a really hot-looking bridge cannot burn your tongue and throat.

A Thing of Beauty & Awe

The first time I visited Tampa, I went out of my way to drive across the new Sunshine Skyway (I tend to go out of my way to find bridges a lot

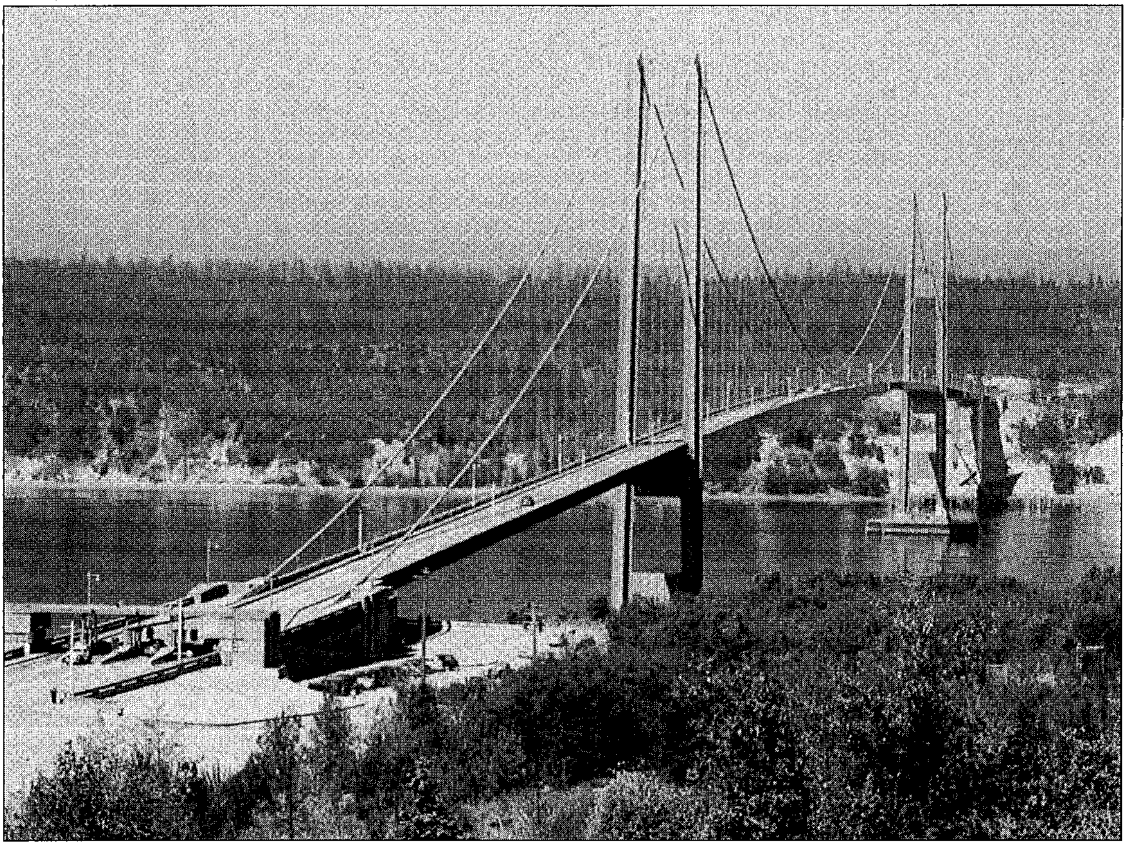

FIGURE 1. The first Tacoma Narrows Bridge.

on trips). The Skyway is the spectacular, signature cable-stayed bridge that crosses Tampa Bay for five and a half miles, and replaces the older defunct span. The bridge is so long, that when driving on it after a while you cannot see the shoreline. It feels like you are driving out on the ocean. Mile after mile, there is just you and your lonely automobile, lost atop the deep blue of the sea, with no land in sight. Then, way out in the middle of the bridge (in which you are lost! lost!), you start to rise on a fearsome, steep climb on narrow lanes with only a tiny, insubstantial concrete rail parapet protecting you from a dismal plunge into the shark-infested bay. The steep slope flattens out, but then you find yourself on the middle of the main span, suspended in mid-air, impossibly high above the infinite deep blue abyss. The single plane of golden cables are probably strong, but from your vantage point they look like twine, ready to snap at the slightest jerk or breeze. A fierce crosswind buffets your car. You grip the steering wheel in terror, knuckles turning white as the blood flow is constricted. You try to chart a straight line in the road lane, hoping not to slip ever so slightly to the side, next to the flimsy, insubstantial rail and then over, in a screaming plummet into the anonymous brine.

OK, so maybe I felt a slight twinge of anxiety as I crossed the Sunshine Skyway — me, one of the world's great appreciators of bridges, and I found myself coming down with gephyrophobia. There was no way this structural engineer could develop a fear of bridges. What if people found out? I would have to nip the anxiety in the bud, and I would have to do it fast, or who knew would mental malady I would come down with next: Dementia? Schizophrenia? An insatiable desire to save old engineering reports?

The Downside of Gephyrophobia

I found a website that specializes in all types

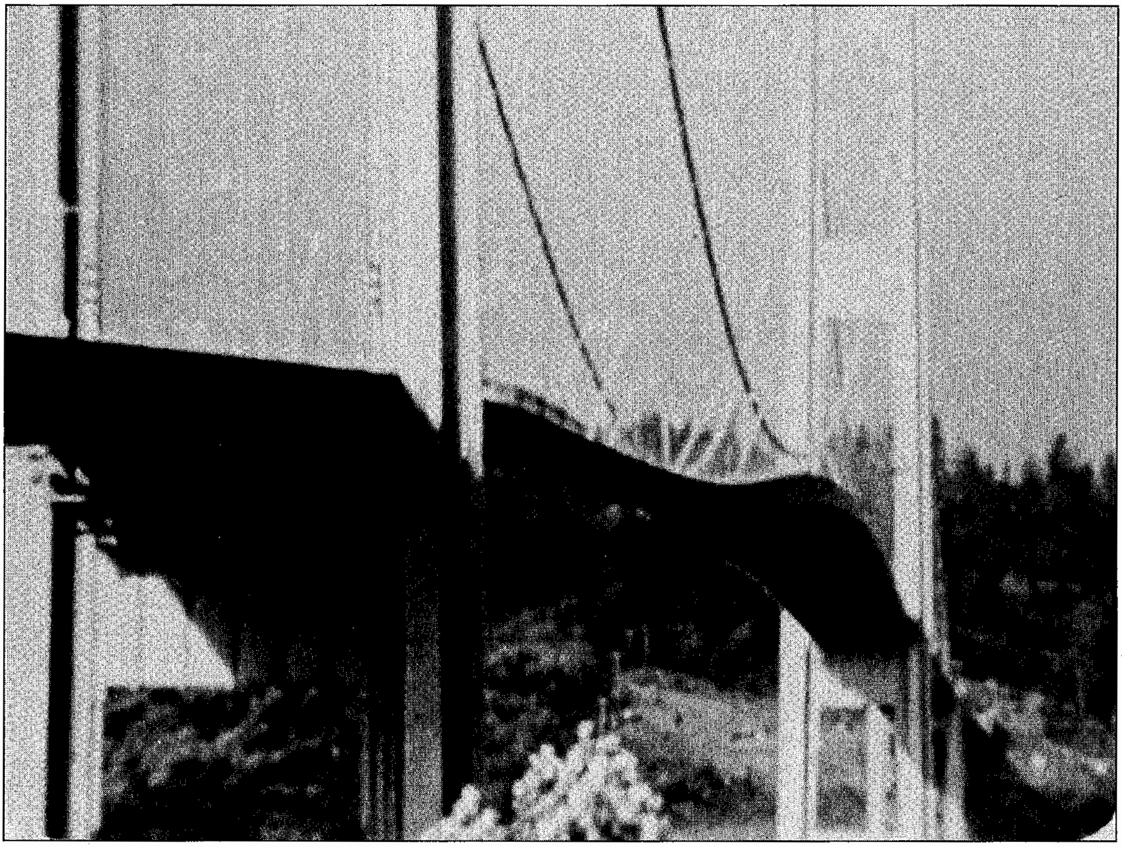

FIGURE 2. The first Tacoma Narrows Bridge doing what it is best remembered for.

of anxieties. According to Change That Right Now (CTRN), gephyrophobia can be more debilitating than you might think:

"If you are living with fear of bridges, what is the real cost to your health, your career or school, and to your family life? Avoiding the issue indefinitely would mean resigning yourself to living in fear, missing out on priceless life experiences big and small, living a life that is just a shadow of what it will be when the problem is gone.

"For anyone earning a living, the financial toll of this phobia is incalculable. Living with fear means you can never concentrate fully and give your best. Lost opportunities. Poor performance or grades. Promotions that pass you by. Your phobia could cost you tens, even hundreds of thousands of dollars over your lifetime, let alone the cost to your health and quality of life. Now fear of bridges can be gone for less than the price of a round-trip airline ticket."[2]

Apparently the fear of bridges is quite common, and hundreds if not thousands of people avoid bridges by taking tunnels instead — that is, until they develop a fear of tunnels, and then they are either stuck on the ferry, driving tens or even hundreds of miles out of their way, or they have to stay home.

According to CTRN, there are hundreds of things to be afraid of (over 1,300, in fact). Most of these phobias are associated with issues that, in a rational sense, are not really life-threatening — fear of public speaking, for example, in comparison to the fear of being eaten by a squid. Many people report that public speaking is their greatest fear of all. If they had a choice, they would probably choose to swim with killer squid instead of getting up in front of 300 people for a pres-

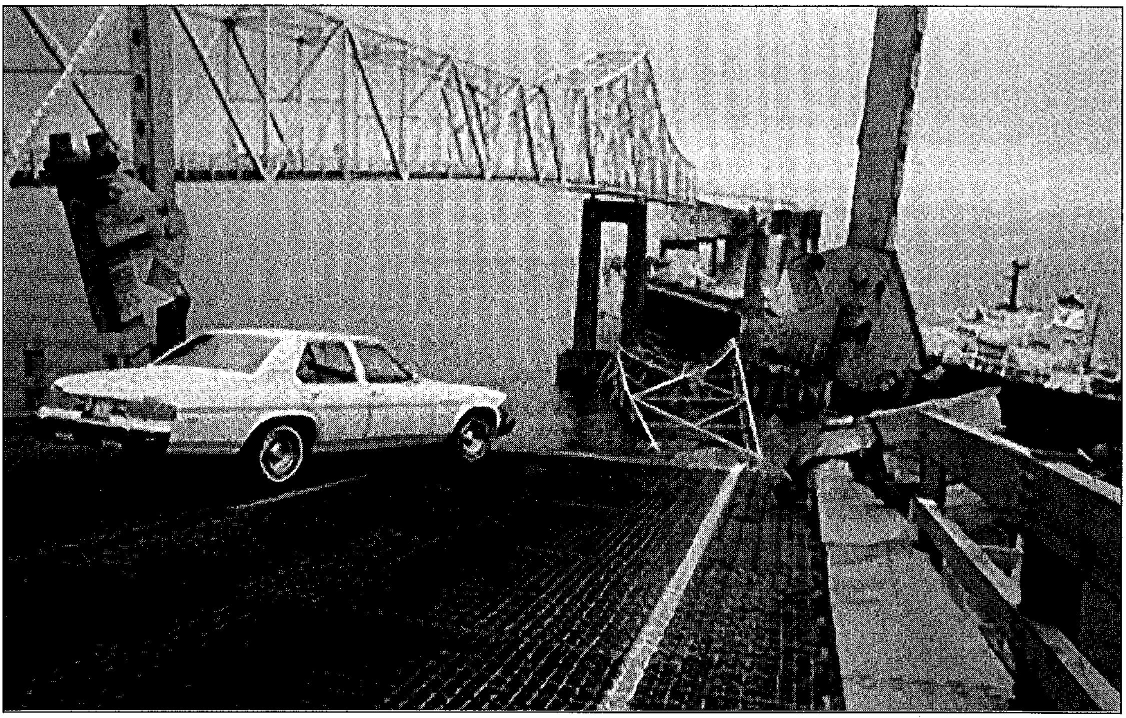

FIGURE 3. The Skyway Bridge after its tussle with a barge.

entation. In that case, they would be eaten by a squid before the presentation and then they would not have to worry about the speech.

The anxieties reported by CTRN are modern maladies. At the dawn of time, mankind did not have to worry about gephyrophobia. There were not many bridges and no one drove across them. There were plenty of things to be afraid of, of course. For example, while there were no PowerPoint presentations in the cave, you did have to worry about getting attacked by a large carnivore (in the grand scheme of things, a much more substantial fear). With mankind's triumph over the carnivores, it seems like our psyches, which are probably naturally wired to be afraid (that flight or fight response can probably take credit for our species' current dominance on this planet), are experiencing a fear deficit. We have filled the fear gap with modern anxieties, including, sadly, the fear of bridges.

A Cure

What to do about this grievous malady? Hypnosis is one option. A website named *HypnosisDownloads.com* offers self-hypnosis tapes that can help you overcome this particular fear:

> "The thought of being relaxed while driving across a bridge probably feels impossible to you, but you will be amazed after listening to *Fear of Driving Over Bridges*."[3]

Your therapy begins with only a modest payment of $12.95, plus shipping and handling. As an extra bonus, after being hypnotized by this tape, when you wake up, you will think you are a chicken. (I apologize: That was a cheap hypnosis joke.)

Gephyrophobia is not a joke, although I have done my best to milk it for what it is worth here. The fear of bridges is big business. You can shell out quite a bit for hypnosis tapes and therapy. If that does not work, you can actually hire people to drive your car across the intimidating bridge. The Chesapeake Bay Bridge is a good candidate for this solution. The bridge is so long, so high in the air, so exuberant and so separated from the shore that

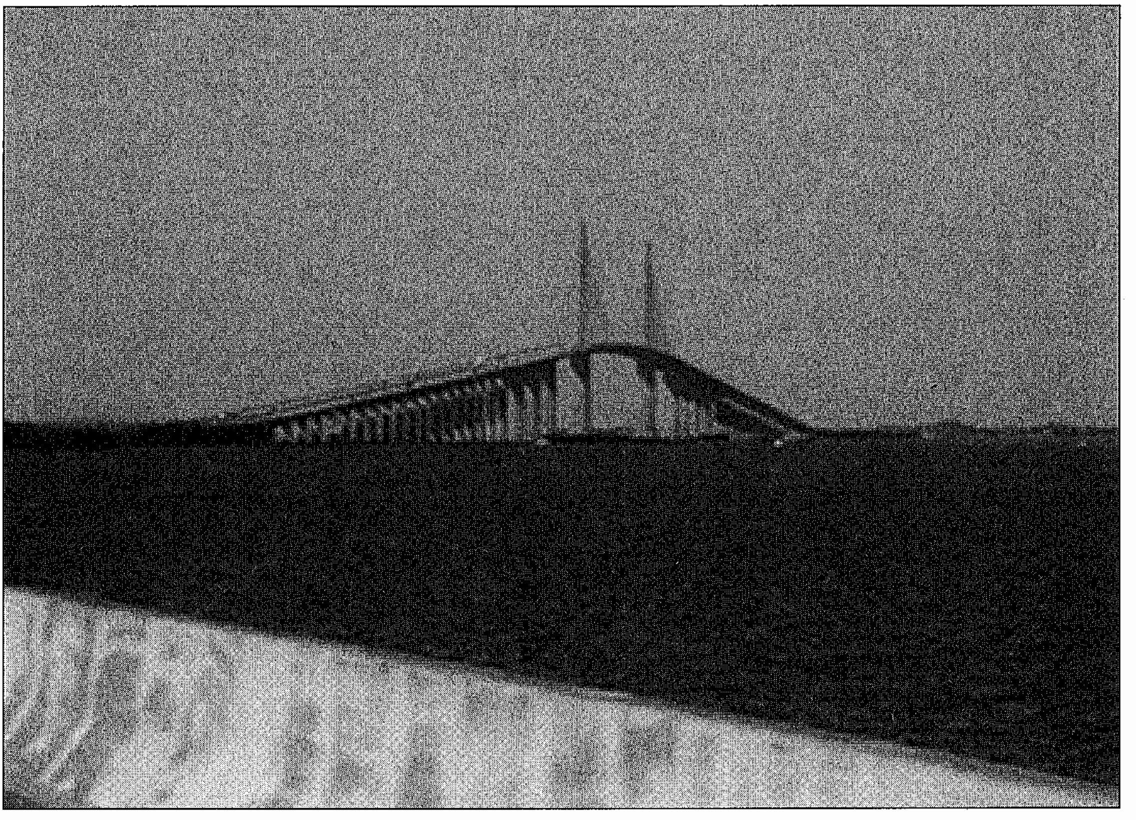

FIGURE 4. The rebuilt Sunshine Skyway.

driving across it is not for everyone. In fact, the Maryland Transportation Authority provides information for "drive over" services, companies that will drive your car across the bridge for a modest fee. There must be some demand for this service since 4,000 people use it annually.[4] Hire a driver, add a little sleeping medication to the mix and you can get from Annapolis to the east shore of the bay without ever seeing how close you ended up swimming with the fishes.

Pragmatic Relief

One way of overcoming anxiety is to confront your fears. Using this method is how I overcame my fear of public speaking, and how I have now overcome my slight, not-worth-mentioning case of gephyrophobia. By sharing my anxiety with you, by airing it all out in public, I am freed from the mildly gripping fear. Now I can proceed with my life, and I do not have to shell out $12.95, plus shipping and handling. Are you afraid of bridges? You can overcome it! Here is my program: Watch the Tacoma Narrows videotape for a few hours and then find a nice, long, high and delicately supported bridge to drive across. Do this a few times on a windy, stormy day until your confidence builds and the anxiety slowly drains. Follow this program and you, too, can join the ranks of the bridge-fearless (I wish there was a Greek-derived word for this). Do it because bridges should be appreciated and not feared. Do it now, and start to experience a life free of gephyrophobia.

Do it, and don't worry about the fact that every time you cross the bridge, the odds are infinitesimally increased that something bad will happen the next time you cross it.

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Article Index 1986–2008

The following index covers the Spring 1986 to the Spring/Summer 2008 issues and is grouped by discipline. Articles are alphabetized by title within each grouping. The scope of many articles included here fall across more than one discipline. Articles are not cross-referenced or included in more than one grouping, so it is recommended to scan related groupings when seeking a particular article. Each entry gives the article title (in italics), author(s), issue, and page citation.

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A Model Coastal Zone Building Code for Massachusetts, Ad Hoc Committee on Coastal Zone Building Codes of the BSCES Waterway, Port, Coastal & Ocean Engineering Technical Group, Fall 1991, pp. 21-30

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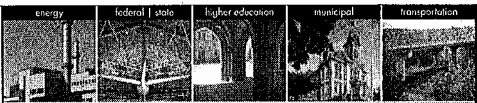

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Photo by Patrick Barber.