CONCRETE DAM EVOLUTION
The Bureau of Reclamation's Contributions

Gregg A. Scott[1], P.E., Technical Specialist, Structural Analysis and Geotechnical Groups
Larry K. Nuss [1], P.E., Technical Specialist, Structural Analysis Group
John LaBoon[1], P.E., Manager, Waterways and Concrete Dams Group

I. Introduction

Over the last 100 years the Bureau of Reclamation (Reclamation) has made significant engineering contributions to the advancement and evolution of concrete dam analysis, design, and construction. The beginning of Reclamation's long history of world renowned concrete dams began shortly after the turn of the 20th century with the construction of landmark masonry dams. Arch, gravity, and buttress dam design evolved through the 1920's. In the 1930's with the design and construction of Hoover Dam, significant strides were made in design, analysis, and construction. Strides were also made in concrete materials, temperature control, and construction techniques. Concrete technology improved to solve the problems of alkali-aggregate reaction and freeze-thaw damage following Hoover Dam. In addition to Hoover Dam, some of the largest concrete dams in the world were constructed by Reclamation during the 1940's and 1950's. Following the failure of Malpasset Dam (France) in the late 1950's, it became fully recognized that foundation conditions were critical to the stability of concrete dams. Reclamation made significant contributions in the areas of rock mechanics and dam foundation design in the 1960's and later. In the 1970's significant attention was paid to the earthquake response of concrete dams, and Reclamation was among the first to apply the finite element method to these types of analyses. A new method of concrete dam construction, termed roller-compacted concrete (RCC), was developed in the 1980's using earthmoving and paving technology to transport and place concrete materials, resulting in shorter construction times and decreased cost. Reclamation advanced RCC materials design and placement methods. Continued evaluations for dam safety, operations, and maintenance have been in the forefront of recent Reclamation activities. As the behavior and risks posed by these dams are better understood, modifications have been made for several concrete dams to improve their safety and service life. Part of the evolution of concrete dam analysis, design and construction, has been associated with waterways; specifically spillways and outlet works. These features are key components to safely passing releases through concrete dams. Although these features are also critical for embankment dams, advances often came during concrete dam design due to the high heads associated with many of these structures.

[1]Bureau of Reclamation, Technical Service Center, Denver, Colorado

Large Masonry and Concrete Storage Dams
Designed and Built by the Bureau of Reclamation
or Currently in the Bureau of Reclamation Inventory

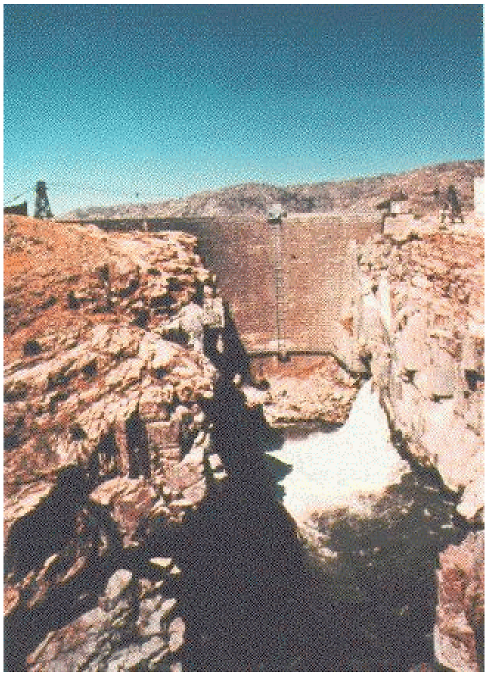
Dam	Year Completed*	Type	Structural Height (feet)**	State
Pathfinder	1909	Thick Arch	214	Wyoming
Buffalo Bill	1910, 1990	Thick Arch	350	Wyoming
East Park	1910	Gravity Arch	139	California
Jackson Lake	1911	Composite Gravity/Embankment	66	Wyoming
Theodore Roosevelt	1911, 1996	Thick Arch	356	Arizona
Arrowrock	1916	Gravity Arch	350	Idaho
Elephant Butte	1916	Gravity	301	New Mexico
Clear Creek	1918, 1993	Thick Arch	84	Washington
Warm Springs	1919	Thin Arch	106	Oregon
Black Canyon Diversion	1924	Gravity	183	Idaho
Gerber	1925	Thin Arch	88	Oregon
Mormon Flat	1926	Thin Arch	224	Arizona
Horse Mesa	1927	Thin Arch	305	Arizona
Stony Gorge	1928	Slab and Buttress	139	California
Gibson	1929	Medium-thick Arch	199	Montana
Stewart Mountain	1930	Thin Arch	207	Arizona
Deadwood	1931	Medium-thick Arch	165	Idaho
Owyhee	1932	Thick Arch	417	Oregon
Thief Valley	1932	Slab and Buttress	73	Oregon

Dam	Year Completed*	Type	Structural Height (feet)**	State
Hoover	1936	Thick Arch	726	Nevada/Arizona
Parker	1938	Medium-thick Arch	320	Arizona
Bartlett	1939	Multiple Arch	309	Arizona
Seminoe	1939	Medium-thick Arch	295	Wyoming
Friant	1942	Gravity	319	California
Grand Coulee	1942, 1974	Gravity	550	Washington
Marshall Ford	1942	Gravity	278	Texas
Altus	1945	Curved Gravity	110	Oklahoma
Shasta	1945	Curved Gravity	602	California
Angostura	1949	Composite: Gravity/Embankment	193	South Dakota
Olympus	1949	Composite: Gravity/Embankment	70	Colorado
Keswick	1950	Gravity	157	California
Kortes	1951	Gravity	244	Wyoming
Hungry Horse	1953	Thick Arch	564	Montana
Canyon Ferry	1954	Gravity	225	Montana
Folsom	1956	Composite: Gravity/Embankment	340	California
Monticello	1957	Medium-thick Arch	304	California
Anchor	1960	Thin Arch	208	Wyoming
Flaming Gorge	1964	Medium-thick Arch	502	Utah
Glen Canyon	1964	Thick Arch	710	Arizona
East Canyon	1966	Double-curvature Arch	260	Utah

Dam	Year Completed*	Type	Structural Height (feet)**	State
Yellowtail	1966	Medium-thick Arch	525	Montana
Swift	1967	Double-curvature Arch	205	Montana
Morrow Point	1968	Double-curvature Arch	468	Colorado
Wild Horse	1969	Double-curvature Arch	110	Nevada
Mountain Park	1975	Double- curvature Arch	133	Oklahoma
Pueblo	1975	Composite: Massive-head Buttress/ Embankment	250	Colorado
Crystal	1976	Double-curvature Arch	323	Colorado
Nambe Falls	1976	Composite: Double-curvature Arch/ Embankment	150	New Mexico
American Falls	1978	Composite: Gravity/Embankment	104	Idaho
Upper Stillwater	1987	RCC Gravity	292	Utah
Brantley	1989	Composite: Gravity/Embankment	144	New Mexico

* For cases where the height or shape was significantly altered, the modification date is also given
** Structural height is generally the difference between the dam crest and lowest point of the excavation

II. Masonry Dams and The Early Years


Pathfinder Dam, WY

Shortly after the beginning of the 20th century, just after the establishment of the U.S. Reclamation Service, explorations were underway for large storage dams. In September 1903, Mr. George Y. Wisner, consulting engineer for the Reclamation Service, addressed a conference of Reclamation Service Engineers in Ogden, Utah. He indicated Reclamation would be required to build masonry dams of great height in order to store the water required to reclaim arid lands. This could be accomplished in narrow canyons where the arch action of the dam could be taken into account, provided the plans were based upon accurate data and correct determination of the stresses to which the dams would be subjected. In 1904, Mr. Wisner began what was to be a leading role in the design of Pathfinder Dam on the North Platte River in central Wyoming, collaborating with Edgar T. Wheeler, consulting engineer, on the analysis. It was recognized that masonry dams are far from rigid, and that temperature was an important load. The modulus of elasticity and coefficient of thermal expansion were estimated for a composite of rock and concrete. The dam was designed as a combination of an arch and a vertical cantilever fixed at the base. The load, both temperature and reservoir, was distributed between the arch and cantilever so as to produce equal deflections. The stresses resulting from the deflections were then calculated. This was the early beginnings of what was to later become the Trial Load method of analysis. The designed cross-section, constructed on a radius of 150 feet, was determined to give sufficient thickness to safely resist the forces that would act upon it. Above elevation 5830, reinforcement was considered necessary to reduce thermal cracking.

The dam was constructed in a narrow granite canyon. A large tunnel was constructed to divert the flow of the river, and later was used for the outlet works. Foundation excavation and dam construction was facilitated by an overhead cableway and guy derricks with steam driven hoist engines. The overhead cableway was key to constructing in the deep narrow canyon. Cableways are still an important component of modern construction for such conditions. Steam engines powered the concrete and mortar batch plant as well as the aggregate crushing and sorting plant. The side walls of the canyon were excavated to produce surfaces normal to the face of the dam. The first masonry was laid in August 1906, and the dam was completed in 1909. It was recognized that an impervious dam could be built at the same cost as a leaky dam, the main difference being more rigid inspection and an understanding at the start that first-class work only would be allowed. Any rock to be built against and any material to be placed in the dam was thoroughly washed and cleaned. A course of masonry was built on the upstream and downstream faces, and granite stone from the spillway excavation, varying in size from one to five cubic yards, was set in a heavy bed of mortar between the faces. The stones were lifted, reset, and vibrated with bars as necessary to get them completely in contact with the mortar. The vertical joints were filled with concrete consisting of cement, sand, and coarse aggregate. The concrete was fairly wet and would flow into most of the joints, where it would be worked by shovels and leveled. Spalls or small stones were placed in the wider joints. The stone was placed from abutment to abutment. Stone of differing heights resulted in beds of mortar at varying elevations throughout the structure. Due to the high cost of cement, which was furnished by the Government, attempts were made to optimize the use of concrete and mortar. This required skilled masonry workers. Flat deformed steel bars were placed in the mortar joints near the face of the dam above elevation 5830. The finished dam has a structural height of 214 feet, and impounds about 1 million acre-feet of water. The dam has performed extremely well for nearly a century, and for all practical purposes should have an indefinite life.

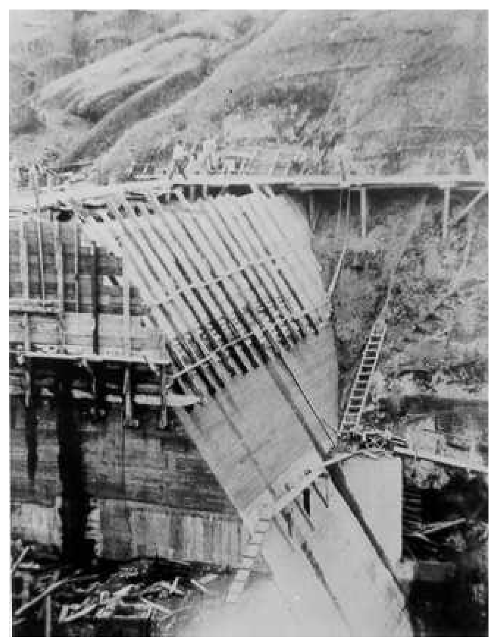
Similar masonry construction was in progress about the same time for Theodore Roosevelt Dam on the Salt River, in south-central Arizona. The design of the dam was somewhat more conservative than Pathfinder Dam, having a more conventional gravity dam section. This probably reflects the fact that it was designed under the direction of different engineers, Mr. F. Teighman and Mr. Louis C. Hill, and that the design for Theodore Roosevelt Dam probably predates that for Pathfinder Dam, even though Pathfinder Dam was completed first. Construction at Theodore Roosevelt Dam began in 1903. It appears that a simpler design methodology was employed. The dam was designed two-dimensionally such that the resultant force from maximum anticipated static loading fell within the middle third of the structure, and then the dam was arched to provide an extra margin of stability. It was recognized that temperature could affect the upper portions of the dam, and records indicate that some reinforcing steel was used in this area. Despite this, the thinner upper portion of the dam cracked at regular intervals, in effect forming contraction joints. Leakage through

Original Theodore Roosvelt Dam, AZ.

Masonry Construction at Theodore Roosevelt Dam, AZ


East Park Dam, CA

masonry construction to concrete construction. Wooden forms were built at the upstream and downstream faces for concrete placement. Concrete was mixed and deposited in 8-inch layers. Granite plum rocks, forming approximately 25 percent of the concrete volume, were placed in the concrete, and were shaken or rammed into final position. This solidified the mass to a remarkable degree, and additional tamping was scarcely required. However, spading and tamping was performed to work the concrete into all the cavities of the rock and ensure consolidation against the forms. The plum stones usually projected about half of their thickness above the surface of the new concrete. This presented a rough surface for bonding with the next layer. When a layer of concrete had set for more than 24 hours, the surface was thoroughly cleaned and a thin coat of mortar was placed prior to the next layer of concrete. The concrete was placed from abutment to abutment without contraction joints. Due to the contractor's desire to complete the work, winter placements occurred under a steam-heated tent. Upon completion in 1910, the dam was 325 feet high, and capable of storing over 400,000 acre-feet of water. The dam was raised 25 feet in 1989.


Construction at East Park Dam, CA. Note the vertical formed contraction joint and concrete forms

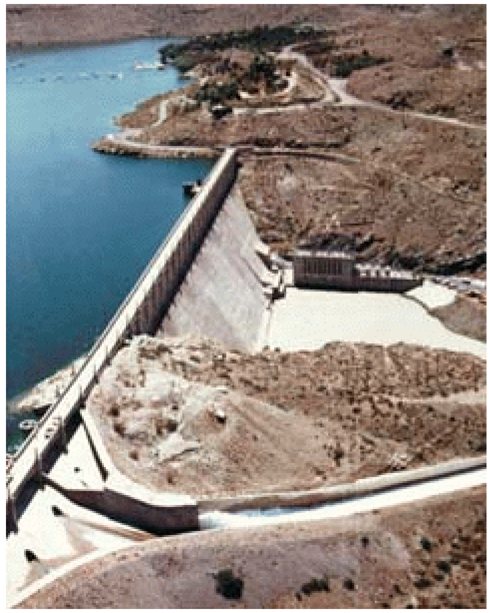
The first use of vertical radial contraction joints for Reclamation concrete dam occurred at East Park Dam in north-central California. The radial joints were spaced at 20 feet, and a shear key, six inches deep by three feet long, was constructed in the contraction joints about six feet from the upstream face. Although there is no indication that waterstops were installed in the joints, a system of four-inch diameter tile drains was constructed downstream of the keys to convey water from the joints to the outlet tunnel. This dam also was constructed entirely of concrete. The original design called for sandstone blocks to be imbedded in the concrete to make up 20 to 30 percent of the mass. However, the sandstone was of poorer quality than first believed, and the sandstone blocks were omitted from the construction. The aggregate was processed and screened into three sizes (1/4, 1, and 3 inch). A little over one barrel (4 sacks) of cement was used for each cubic yard of cement. The concrete was placed quite wet, and water cured for 10 days. The dam was designed as a curved gravity structure, similar to Theodore Roosevelt Dam. It was constructed in a narrow gorge of massive conglomerate. Although the dam was completed in 1910, construction began in 1908, after construction of Theodore Roosevelt, Pathfinder, and Buffalo Bill Dams had begun. Despite the work of Mr. Wisner, a more conservative approach was taken. The 140-foot high dam impounds a reservoir of about 50,000 acre-feet.


Arrowrock Dam, ID

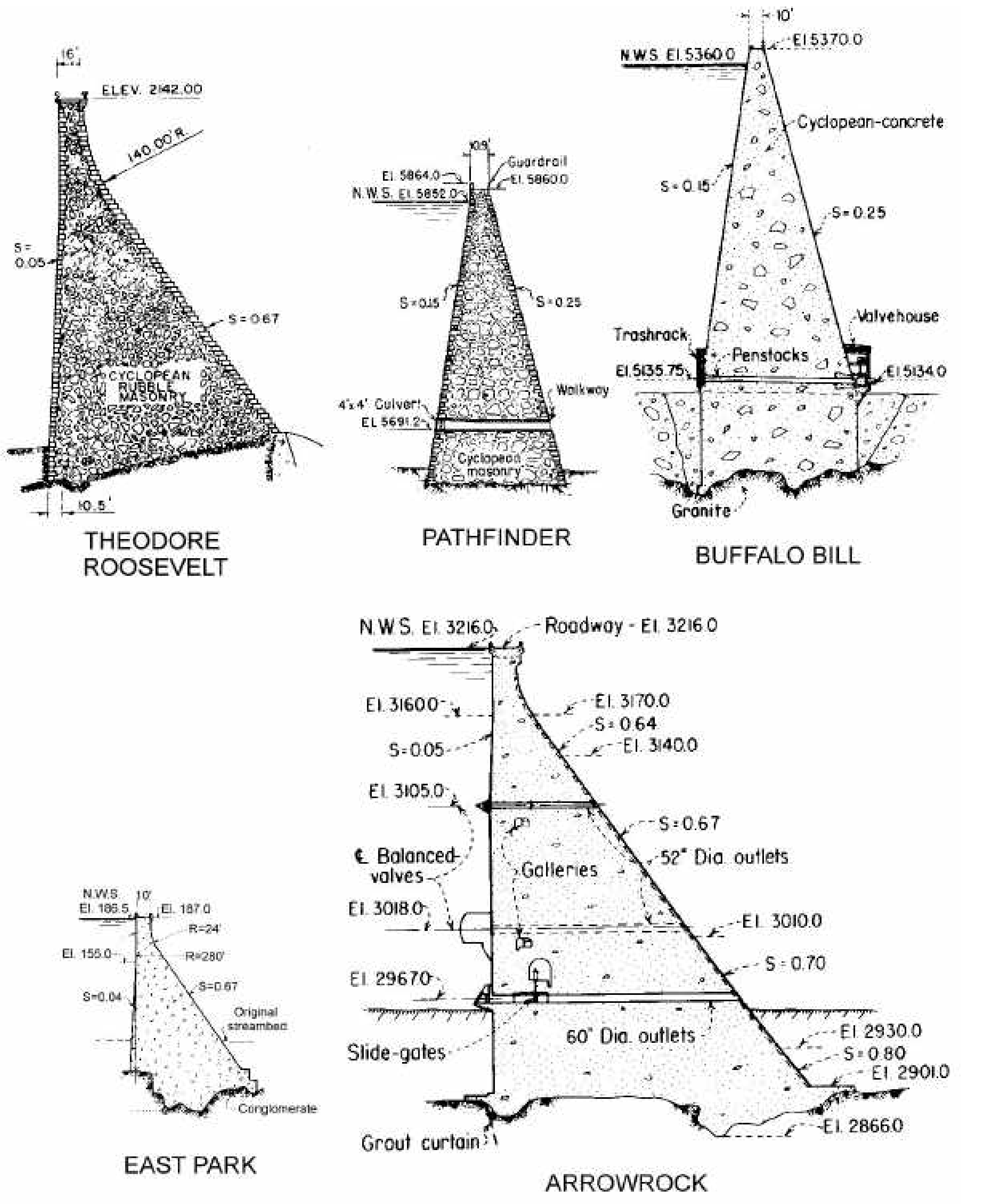
The reign of Shoshone Dam as the world's highest dam was short lived. In 1916, Arrowrock Dam was completed to a height of 349 feet. Once again, the cross section of this dam was similar to a gravity dam, but the dam was constructed as an arch. The construction of Arrowrock Dam also made use of vertical contraction joints. Radial joints were formed in the upper portion of the dam by building alternate sections at different times. The joints were spaced at various intervals dependent on the elevation and thickness of the dam. Three vertical wells were formed in each joint which were later filled with concrete during cold weather, after the dam had undergone contraction. A Z-strip, annealed-copper water stop was installed in each joint 5 feet from the upstream face of the dam, and immediately downstream from this strip a triangular drain was formed in the joint. These drains collect water which gets past the water stop and transports it to inspection or operating galleries. A unique material called "sand cement" was used for the construction of this dam, and for Elephant Butte Dam, a 300-foot-high straight gravity dam near Truth or Consequences New Mexico, completed the same year. This consisted of standard Portland cement to which was added a little less than an equal amount of pulverized sand, reground to such fineness that 90 percent would pass a No. 200 sieve. Although this saved on the quantity of cement used, the concrete did not attain as much strength, and as a result, the durability suffered. This was not significant for the relatively mild climate at Elephant Butte Dam, but at Arrowrock Dam, spray from downstream releases resulted in severe freeze-thaw damage to the concrete. This necessitated construction of a new overlay on the face of the dam in 1936. The use of sand-cement in the construction of concrete dams was discontinued after these projects. The concepts of foundation grouting and drainage appear at Arrowrock and Elephant Butte Dams, and galleries were constructed in both of these dams. Shallow grout and drainage curtains (25 to 30 feet deep) were constructed by drill holes in the granitic near the upstream face of Arrowrock dam. The foundation drainage holes, spaced at about 10-foot centers, exit in an inspection gallery 27.5 feet from the upstream face. Vertical formed drains were also constructed within the concrete, spaced at 15 feet and located 12 feet from the upstream face of the dam.

These drains also exit in the inspection gallery. Similar construction occurred at Elephant Butte Dam.

In 1918 Mr. Duff A. Abrams first published results of research that investigated the effect of water-cement ratio and grading of aggregates on concrete quality. This was a major breakthrough in developing the science of concrete technology. Obviously, Reclamation concrete dams constructed up to that point did not have the benefit of his research, and the concrete quality and durability was largely a function of fortuitous circumstances and the experience of the on-site staff. With the exception of Arrowrock Dam, which required fairly minor modifications for freeze-thaw damage due to nondurable concrete, the early concrete dams of the Bureau of Reclamation have held up remarkably well.



Elephant Butte Dam, NM



Comparison of Maximum Sections of Early Reclamation Arch Dams

III. The Amazing Arch and Developments of the 1920's


Gerber Dam, OR

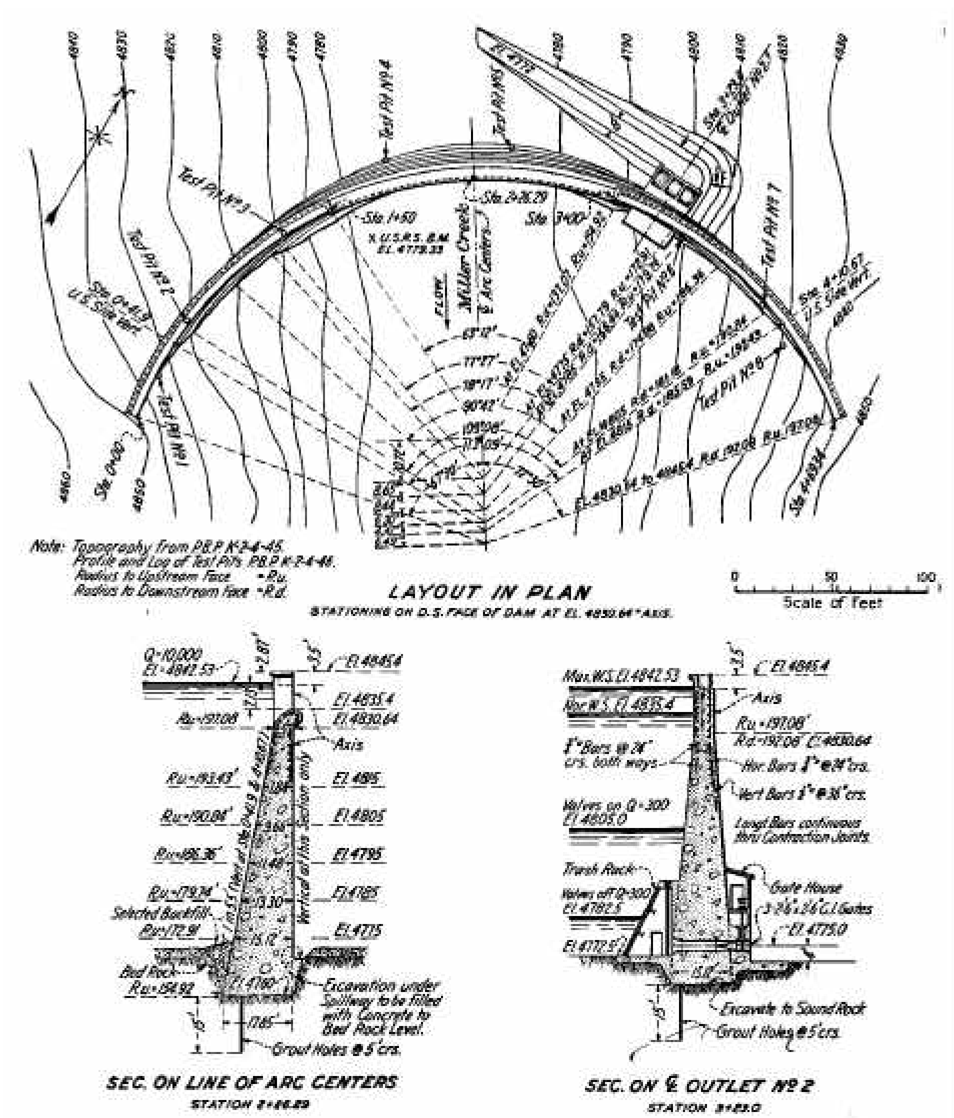
During the 1920's, materials were relatively expensive, and there was a desire to optimize dam design to reduce the required concrete. Independent arch theory became the order of the day, as thinner dams resulted from this method of design. Hence, many thin concrete arch dams were designed and constructed during this era. In addition, buttress dams became popular for wider canyons, since they minimize the required materials in favor of a more labor-intensive construction. The Bureau of Reclamation inventory contains only one gravity dam (Black Canyon Diversion) from this era. Most of the arch dams from this era in the Reclamation inventory were designed and constructed by water user groups. Titles were later transferred to Reclamation for various reasons. One of the exceptions is Gerber Dam. Gerber Dam was completed in 1925 on Miller Creek, a tributary of the Lost River in southern Oregon. The dam is a variable radius arch with a structural height of 85 feet and a reservoir volume of 94,000 acre-feet. The Design Engineer was J.L. Savage and the General Construction Superintendent was F.T. Crowe, two individuals who would play prominently into later Reclamation projects. The foundation for the dam is basalt with weak clayey interflow zones. As was the practice up until this time, the main concern for foundation conditions related to the strength and hardness of the rock, and the water-tightness of the foundation. To assess the water-tightness of the foundation, tests were conducted in drill holes. Pipes were grouted and sealed into eight drill holes. Water was applied to all eight holes simultaneously under pressure from an elevated water tank. The leakage was determined to be small. Still, after excavating a keyway trench for the foundation to a depth greater than anticipated, a grout curtain was installed to a depth of 15 feet in holes spaced about 5 feet apart throughout the length of the foundation. The holes were grouted after the concrete above the grout hole reached a thickness of 6 feet by applying a steam pressure of 100 lb/in^2. No foundation drainage was included in the design or construction.

Concrete was placed in the dam by use of a trestle with rail buggies, a stiff leg derrick, and a high line. Most of the concrete was placed by cars with a 3/4 yd^3 capacity, run on the trestle from the mixer and dumped into chutes and pipes leading to the forms. Five to six sacks of cement were used for each cubic yard of concrete. Plum rocks, not exceeding 20 percent of the volume, were placed in the concrete at locations away from the forms, to reduce the needed concrete volume and provide small keys between lifts. Cold weather placements required heating the sand and mixing water, as well as heating the concrete placements under canvas enclosures. The rock

Photo taken during construction of Stony Gorge Dam, CA from downstream side. Note struts between buttresses and sloping slabs on left side of photo.

and buttress dam built downstream of East Park Dam. It has a structural height of about 140 feet, and a reservoir capacity of 50,000 acre-feet. The dam is made of individual simply supported elements; buttresses, upstream face slabs, and struts bracing between buttresses in the downstream areas. The sloping upstream slabs span between and transfer the reservoir loading to the buttresses; the buttresses carry the upstream-downstream loading and transfer it to the foundation; and the struts provide lateral stiffness to the buttresses and keep them from deforming excessively in the cross-canyon direction. The reinforced concrete members were designed using codes available at the time. Additional horizontal reinforcing was added to the buttresses following the early appearance of vertical cracks in some of the taller buttresses. A recent check indicates the design is generally acceptable for normal static loading conditions, even considering modern American Concrete Institute (ACI) code. The concrete mixing plant discharged into bottom-dump buckets of 1-1/2 cubic yard capacity which were successively transported by hoist, highline cableway, and small cars on light tracks supported by the buttress forms to chutes conveying the concrete from the buckets to its final position.

The method of using chutes to convey the concrete was common practice during this era. This required a wet concrete mix for enough workability to allow the concrete to flow along the chutes. Unfortunately, this also resulted in somewhat weaker and less durable concrete than could be attained with a drier mix. In addition, it often resulted in latence rising to the lift surfaces. If this was not removed and thoroughly cleaned, bonding between successive lifts was compromised. However, many dams from this time period have performed well and are still in service. Although concrete technology had advanced, the effects of alkali-aggregate reaction and freeze-thaw deterioration were not well understood. Most of the arch dams constructed during this era in cold climates suffer from freeze-thaw deterioration, such as Gerber Dam. If built with reactive aggregate, the resulting cracking typically accentuates the freeze-thaw damage. Dams subject to alkali-aggregate reaction in mild climates, such as Stewart Mountain Dam, tend to exhibit cracking but continue to perform well.



Gerber Dam Plan and Sections

IV. Prelude to Hoover Dam


Owyhee Dam, OR

Owyhee and Gibson Dams were built before Hoover Dam and included experimental sections for collecting temperature data and grouting in preparation for the construction at Hoover. These were also the first Bureau of Reclamation concrete dams to use tunnel spillways. Some of the final developments for the trial load method were also performed during the design of these structures.

Owyhee Dam is located on the Owyhee River in eastern Oregon. It is a concrete, thick arch structure with structural and hydraulic heights of 417 and 325 feet, respectively. The crest is 833 feet long and 30 feet wide at elevation 2675. The maximum base width is 265 feet. The dam was completed in 1932. The dam forms a reservoir (Lake Owyhee) with storage of 1,183,300 acre-feet at elevation 2675. Owyhee Dam was the world's highest dam at the time of completion. John L. Savage, Chief Designing Engineer, wrote:

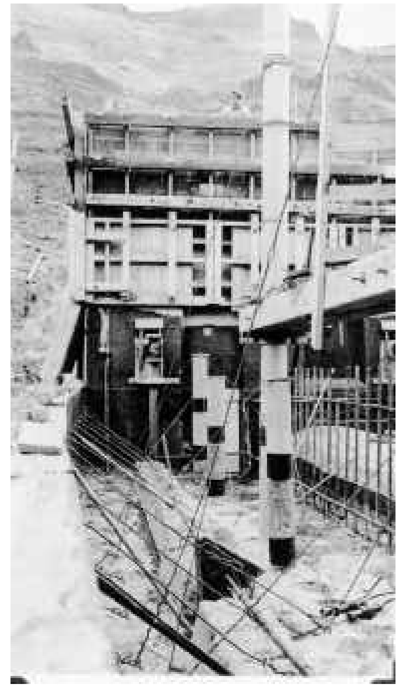
> *"From an engineering standpoint the Owyhee Dam, to be constructed on the Owyhee Project in eastern Oregon, is the most outstanding dam undertaken to date by the Bureau of Reclamation. ... this dam is likely to stand as the highest dam in the world until the great Boulder Canyon Dam [Hoover Dam] is constructed."*

The Owyhee River valley was visited early in the nineteenth century by Hawaiian trappers who are credited with having named the river "Hawaii." Later, this name was handed down phonetically by scouts, Indians, and early settlers as "Ow-Y-Hee", and ultimately the name was given this spelling. The dam site is also referred to as the "Hole-in-the-Ground" site. Intermittent site explorations began in 1903, a feasibility report was issued in 1925, and the project was recommended by the Secretary of the Interior on October 9, 1926. The General Construction Company of Seattle, Washington was the low bidder at $3,198,779 and was awarded the contract on July 7, 1928. The government field organization reached its peak in 1931 with 107 employees under the charge of Mr. F.A. Banks (later to become Construction Engineer for Grand Coulee Dam). In June 1931, the contractor was placing from 40,000 to 50,000 cubic yards of concrete per month. The contractor's workforce reached 274 people. Construction was completed five months ahead of schedule in 1932. The first water was delivered to the irrigation lands in 1935.

Figure III-1. Layout of Gerber Dam

The materials and construction were similar to structures that had come before. The complete details will not be repeated here, but a few items of note are provided. Cobble rock was added to the mix. The cobble rock was sound, clean gravel or broken rock of such size as passed through a screen having 8-inch square or 9-inch round openings and was retained on a screen having 2 3/4-inch square or 3-inch round openings.

Porous concrete tile drains were placed in the dam near its upstream face. The joints in the tile were not cemented. The concrete tile had an internal diameter of not less than 5 inches, and wall thickness of not less than 1 3/16 inches. The tile was made of 1 part Portland cement and 4 parts total aggregate, the aggregates being so proportioned as to give a degree of porosity such that an 18-inch length of tile when set on end on a water-tight base shall discharge water poured into it at the rate of not less than 3 gallons per minute. Construction today would form these drains using a removable five- to six-inch- diameter tapered steel pipe.

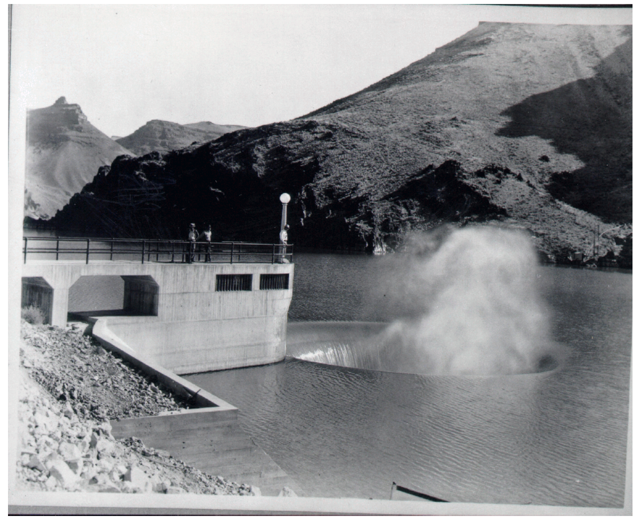

Owyhee Dam, OR: Tile formed drains, strain meter for Hoover test, and gallery reinforcement

The main advancements made during the design and construction of Owyhee Dam involved temperature control. Owyhee Dam was the largest dam at the time in which radial contraction joints were to be pressure grouted. Radial vertical contraction joints were placed at 50-foot intervals with 9-inch deep by 3-feet wide shear keys at 3-foot centers along each joint. The vertical contraction joints were grouted from March 30 to May 8, 1934, which is 2 years after construction of the dam. Internal temperature measurements, concrete cracking, grout operations, grout takes, grout pressures, and contraction joint opening measurements were reported in 1934. The grouting system installed in Owyhee was similar to that previously used in Gibson and Deadwood Dams except for a few minor improvements. A system of pipes was installed along the vertical contraction joints to cool the mass concrete to 50 EF and grout the joints. Grout zones were 100-feet high and isolated with 20-gage soft copper sheets. The radial contraction joints in the dam were pressure grouted with cement grout forced through the pipe grouting systems. The grout was forced in at a pressure


Owyhee Dam, OR: Grout pipes and shear keys on vertical contraction joint.

required to ensure a pressure of at least 100 pounds per quare inch at the highest point in the system being grouted. Vertical keys were built in the joints. The entire face of each vertical joint in the dam, except the grouting units and copper expansion strips, were painted with one thin coat of water-gas tar paint and allowed to dry before the adjacent concrete was place against it. The tar paint served as a bond breaker between the blocks of concrete. Copper grout stops were laid horizontally at vertical intervals of about 100 feet. The top of the grout zone was at elevation 2400, 2500, 2600, and top of dam. Construction today would limit the grout zone to approximate 60 vertical feet. The headers on the upstream face below elevation 2500 were not available for grouting because the reservoir elevation at the time of grouting varied between 2520 to 2527. Owing to the fact that there were quite a few cracks in the concrete in the dam, all cement used was screened through a 200 mesh sieve with the intention that this fine cement would seal most of the cracks. However, considerable cracking in the concrete on the downstream face of the dam occurred, primarily due to alkali-aggregate reaction.

Placing the mass concrete of the dam was begun in the fall of 1930 and completed in the summer of 1932. In the cooler months of the year, concrete was placed at around 52E to 70E F and heated up to around 98E to 116E F. In the warmer months of the year, concrete was placed at around 65E to 82E F and heated up to around 112E to 119E F. At the time of grouting, the internal concrete temperatures varied from 42E to 62E F. Grouting pressures inside the joint were around 100 lb/in^2 . The allowable placing temperatures were much higher than allowed by modern standards, and probably contributed to surface cracking. Electric resistance thermometers were placed in the concrete immediately on pouring. Dissipation of setting heat was accelerated by circulating water through the grout system except in the middle of winter. An experimental cooling system was located in panel 8 at elevation 2486. Tests in Panel 8 measured the effectiveness of cooling coils to dissipate heat in a thick concrete section. Additionally, the upper 82 feet of panels 3 and 4 (blocks 3 and 4, between contraction joints at stations 2+00 and 3+00) at Owyhee Dam were used as a test section to test cooling coils placed on the top of lift lines and their ability to open contraction joints for grouting. In this location of the dam, a system of cooling coils 1-inch diameter were placed 4-feet 7.5-inches apart near the bottom of each 4-foot lift. The section was highly instrumented to obtain temperature and strain measurements. The test section was placed from March 3, 1932 to May 28, 1932 at a fairly uniform rate with about 3.5 days between pours. Reservoir water was circulated in the test section cooling coils for only one month between May 13, 1932 and June 20, 1932. This period of time permitted cooling until the rising river water temperatures and lowering concrete temperatures permitted no further heat extraction from the concrete. Measurement of concrete temperature before cooling shows interior concrete around 117E F and the surface temperatures around 75E F, producing a thermal gradient of 42E F. This amount of gradient is very high and probably contributed to the surface cracking. The contours after the cooling coils were turned off show interior concrete and surface concrete about the same temperature at 70E F. The thermal gradient is very small which would minimize if not eliminate any surface cracking.


Owyhee Dam, OR: Spillway “Burp” (unstable flow condition, sometimes referred to as “blow-back”).

A series of model tests of the Owyhee morning-glory spillway were made from 1930 to 1931. No formal reports were prepared at the time of these studies. In 1944, the hydraulic studies for the spillway tunnels at Owyhee Dam and Gibson Dam were documented. In 1928, when designs for Owyhee Dam were underway, there were few installations of vertical shaft or glory-hole spillways and there was little information available that would assist in the design. The ring gate had no such precedent whatsoever. A 1:48 scale model, which included the topography surrounding the spillway, the spillway and ring-gate control, and the discharge tunnel below the spillway was built to aid in the design. The design included forty-eight 1/16-inch holes equally spaced around the circumference of the lower crest served as air vents to aerate the crest when the gate was raised. Prototype behavior indicates for heads of from 1 feet to 2 feet over the gate, the water falls in a solid sheet toward the center of the shaft, apparently entraining air faster than it can be released at the outlet end of the tunnel. This entrainment causes the pressure to increase until it is sufficient to regurgitate or “break back” through the sheet of overflowing water; then air emerges with sufficient force to carry spray 50 feet or 60 feet above the level of the gate. This phenomenon occurs sometimes as often as once every 15 seconds and sometimes only once in 5 minutes, depending on the tailwater elevation. For heads less than 1 foot over the crest, entrained air can apparently move back up the spillway shaft unhampered. For heads greater that 2 feet, the air pressure is not sufficient to break back and the air is forced through the outlet end of the tunnel, causing spray to be thrown high into the canyon. This action is directly related to the tailwater as a rather large tailwater depth causes a jump to form in the tunnel for most discharges. With a 1000 second-foot discharge, the flow into the stilling basin was undisturbed, but as the flow increased an unexpected disturbance occurred that was not detected in the model. The stream of water from the spillway tunnel created waves on the surface of the stilling pool. These waves traveled across the canyon, reflected, and returned. As they struck the oncoming high-velocity stream from the tunnel an incident occurred which for lack of a better term, is called an explosion. With this particular flow (3000 second-feet) the spray from the explosion was thrown two-thirds the distance up the adjacent cliff. Larger discharges threw spray to the top of the cliff. Evidently the air drawn into the spillway entrance was ejected as a strong wind. When the reflected waves reach the tunnel portal, they are great enough to seal the exit for a short time and the air is quickly compressed to the extent that an explosion results from the release of the air.

During construction, a circular concrete-lined 22.6-foot diameter tunnel 1005 feet long was used for diversion. The tunnel was plugged with concrete upstream from the vertical morning glory shaft. Downstream of the vertical shaft is used as the permanent spillway outlet. The diversion tunnel was constructed in rhyolite tuff requiring no timbering. First a 9- by 9-foot pioneer tunnel was driven followed by the full size tunnel. The rock in the tunnel was hard, self-supporting, full of incipient cracks, with an occasional mud seam. Immediately before placing concrete, the foundation surface was cleaned of mud and debris using a combination of air and water under pressure. The invert was placed by hand and screeded to shape. The crown and side walls were placed in 20-foot sections using wooden forms built in place. A one-yard Ransome concrete gun shot the concrete through a 6-inch pipe and rubber hose into a V notched in the crown of the previous placement. The concrete then flowed along training boards into place. The concrete was worked into place by hammering on the forms with air hammers and by workers equipped with hip boots working and spading the concrete behind the forms. Grout pipes were placed into crevices and holes drilled into the foundation rock at frequent intervals. A 5-sack-per-yard mix was used in the tunnel lining between the inlet and the spillway shaft. A 6-sack-per-yard mix was used from the shaft to the outlet portal. The tunnel was equipped with a grouting system, and the lining-rock interface was grouted in 1934 using a 1.0 water to cement ratio in the invert and side walls. Sand was added to the mix in the roof grouting.

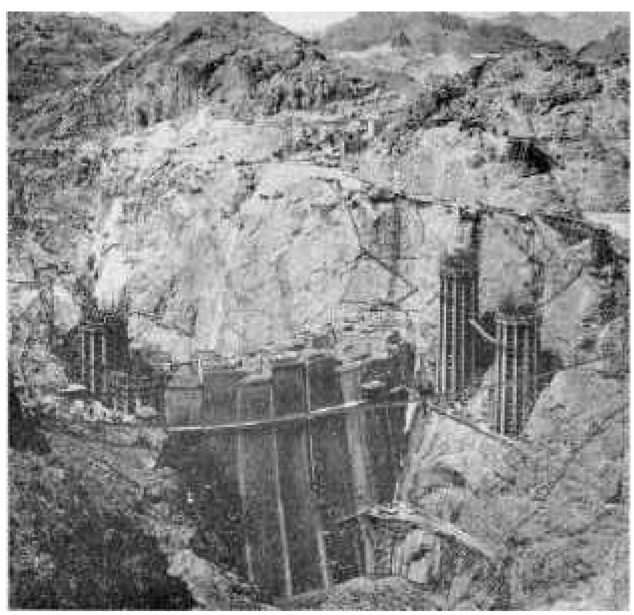
The spillway was featured in the 1956 American Society of Civil Engineers (ASCE) transactions. Excerpts from this article are as follows:

> *"The Owyhee Dam spillway in Oregon, completed in 1932 by the USBR, was a daring design at the time. The capacity is 30,000 cu ft per sec, the maximum head on the crest for this discharge is 12 ft, and the water is dropped 320 ft through a vertical shaft. A flood occurred in 1936 in which 300,000 acre-ft of water were passed in 3 months. The maximum discharge recorded was 15,000 cu ft per sec, or one-half capacity. Subsequent to this flow, smaller discharges have passed through the spillway frequently. A flow of 6,600 cu ft per sec was recorded in 1951. The greatest flood occurred in 1952, when the spillway operated for more than a month. The maximum discharge through the spillway was 20,000 cu ft per sec, or 67 % of capacity. Inspections of the spillway have been conducted frequently since the spillway first operated in 1936; the latest inspection was made after the 1952 flood. The spillway shaft appeared to be in excellent condition. The form board marks still appeared on the concrete surface. The visible part of the invert of the vertical bend showed only slight surface wear, the maximum probably not exceeding 1/4 inch in depth."*

V. Hoover Dam - Quantum Leaps Forward

American Flag displayed during the 1996 Summer Olympics, Hoover Dam, AZ-NV

Hoover Dam is a 727-foot-high concrete thick-arch dam located on the border between Arizona and Nevada about 36 miles from Las Vegas, Nevada. The dam was completed in 1935, has a crest length of 1244 feet, a crest thickness of 45 feet, and a maximum base width of 660 feet. It is the highest concrete dam in the United States, the eighteenth highest dam in the world, and forms the largest manmade reservoir in the United States. The designs for Hoover Dam evolved over several years of careful study, representing the combined efforts of many engineers of Reclamation and various consulting boards. Preliminary designs were prepared from time to time over a period of ten years, so the successive designs reflected some of the developments in design techniques during the 1920 to 1930 decade. In 1920, the first design for a high dam in Boulder Canyon was prepared. At that time the highest dam in existence was Arrowrock Dam in Idaho. Hoover Dam was to be more than double the height of Arrowrock Dam. As such, it was evident from the start that many new problems in design and construction would require solution before the dam could be built.



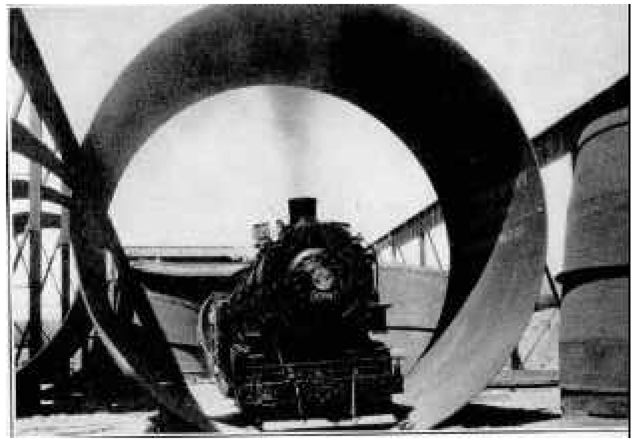
Upstream face of Hoover Dam, AZ-NV

As a result of intensive research, improvements were made in practically every feature in the dam, spillway, and appurtenances. To bring the materials to the site, railroad lines of 48 miles long and 35 miles long were constructed and paved roads from Las Vegas were built. A 150-ton cableway across the canyon was built. Electrical power had to be supplied to the dam site, Government operations, and the newly founded Boulder City. The town of Boulder City had to be planned and built for all the workers at the site. Aggregate, sand, cement, and mixing plants had to be built for the massive amounts of concrete. The concrete was artificially cooled by circulating water through cooling pipes placed at the top of each 5-foot high concrete lift. This required a massive cooling tower 143 feet long, 16 feet wide, and 43 feet high. A steel fabrication manufacturing plant was built to construct the massive penstocks and steel works. Drill crews on elaborate truck-mounted carriages excavated the 56-foot-diameter diversion and spillway tunnels. These tunnels were lined with 3 feet of concrete. The site had to be excavated to sound rock for the foundation of the dam. In the river channel, silt, gravel, and boulders had to be removed to a depth of 120 feet. The foundation was then grouted for the purpose of providing an impervious zone under the dam. The initial grouting involved drilling 6,700 feet of holes and injecting 7,500 sacks of cement. The main cut-off grouting was not started until the dam was at 100 feet high. This operation took 54,000 feet of holes and more than 60,000 sacks of cement (1 sack = 1 cubic foot). The dam was built in a series of 50-foot by 50-foot by 5-foot high blocks. An 8-foot slot was left open down the middle of the dam for the extensive system of cooling pipes. The vertical and horizontal surfaces have formed shear keys. A combination of water stops and grout stops were embedded in the concrete. After each 50-foot vertical section of dam had been cooled, grout was injected into the radial and circumferential joints. The 3.25 million cubic yards of concrete were placed from June 1933 to May 1935 in approximately 23.5 months. A systems of drains were installed in the dam and in the foundation. The foundation drains were 3.5 inches in diameter and extended 100 feet into the foundation at the base and graduated to 30 feet depth at elevation 1200. The internal drains in the concrete were 8-inch porous concrete pipes placed vertically at 10-foot intervals in a line parallel to the dam axis.

Hoover Dam, AZ-NV: Horizontal lift line, and vertical contraction joint

Hoover Dam, AZ-NV: 50- by 50-foot concrete block placements

Hydraulic and structural models were an important part in the design of Hoover Dam to verify existing theories as


Hoover Dam, AZ-NV. Relative size of penstocks

well as advance the current state-of-the-art for applications of greater magnitude than those developed. The hydraulic models provided direct empirical data while the structural models furnished checks on analytical methods using the trial load method. There were two complete models of Hoover. The first model, 1:240 scale, was made of a mixture of plaster and diatomaceous earth. The second model, 1:180 scale, was made of a rubber-litharge compound. In addition, detailed models were made of the crown cantilever and a thick arch at elevation 900 using model tests and slab analogy tests. Therefore, three independent solutions of the same problems were obtained.

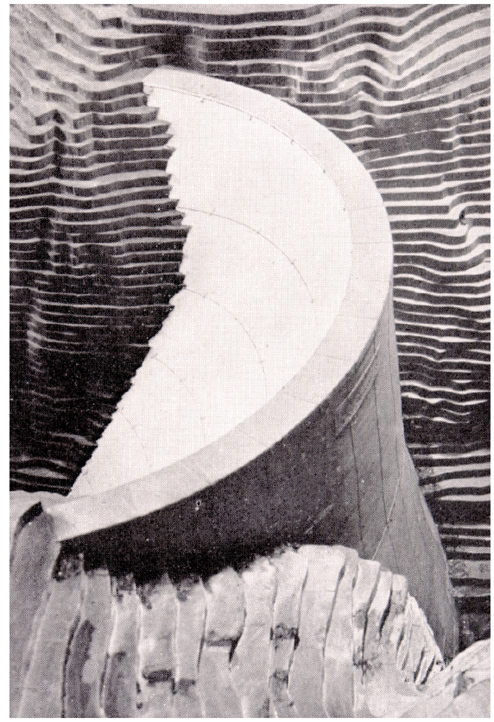
Determining stress distributions in an arch dam requires a 3-dimensional analysis which was very difficult in the 1930s. The trial load method of analysis was developed to represent the 3-dimensional arch structure with a grid of 2-dimensional arch and cantilever elements. The analysis would adjust the load into the elements and bring the elements into geometric agreement. As such, accurate solutions of the arch and cantilever elements had to be known.A.

Part V, Bulletin 2, Technical Investigations - Slab Analogy Experiments

Professor Westergaard, at the University of Illinois in 1931, proposed the use of slab analogy in experimental investigations of stresses in Hoover Dam by means of measurements on rubber slabs. Slab analogy experiments were made to deflect slab models of the crown cantilever and an arch at elevation 900 to obtain stress functions usable in the trial load analyses. Stresses in the slab are proportional to twists and curvature in the slab. In other words, any system of curvatures and twists possible in a slab due to deformation of the boundaries is analogous to a distribution of stress in a plane solid of the same shape distorted by loads applied at the edges. Therefore, to solve a plane stress problem by slab analogy methods, it is sufficient to apply along the boundary of a slab, similar in shape to the original, curvatures proportional at every point of the boundary to the loading on the original. The two structures, being analogous at the boundaries, are thereby analogous throughout; and the direct stress or shear at any point in the solid may be determine from curvatures and twists at the corresponding point in the slab. So proper curvature measurements were made at the desired location and translated into stresses.

B. Part V, Bulletin 3, Technical Investigations - Model Tests of Boulder Dam

Before the Hoover model tests, there were model tests on Gibson Dam in cooperation with the University of Colorado, the Engineering Foundation Arch Dam Committee, and Reclamation.


Scale model of Hoover Dam, AZ-NV

Concrete was mixed with the same aggregate as in the dam and mercury was used for the water load. Results showed the trial load method gives accurate results for an arch dam, and measurements on the model checked closely with measurements on the downstream face of the dam. It was evident however that a different material would need to be used in the Hoover model to permit measurable deflections. As a result, a mixture of plaster and diatomaceous earth (Celite) was developed and used for the first model. During the testing of the plaster/diatomaceous earth model, the Aluminum Corporation of America developed a rubber-litharge compound which was used in the second model of Hoover Dam. It had a lower modulus and same unit weight as concrete. Water could be used for reservoir load instead of mercury permitting measurements on the upstream face. The
model tests showed stress concentrations at the top of Hoover Dam where there was a rapid change in lengths of the arches. As a result, fillets were added to increase the thickness of the dam near the abutments.

C. <u>Part V, Bulletin 4, Technical Investigations - Stress Studies for Boulder Dam</u>

Stress studies for Hoover Dam included several special analyses that had not been previously made including: analysis of tangential shear, twist, Poisson's ratio effects, radial shear in the arch elements, horizontal shear in the cantilever elements, foundation deformation, thermal induced stresses from artificial cooling and exposed surfaces, nonlinear stress distributions in arch and cantilever elements, spreading of canyon walls and settling of the reservoir bottom from reservoir load, grouting and stage construction sequencing, and earthquake loading. Maximum stresses and nonlinear stress variations in typical arch and cantilever elements were checked by slab analogy experiments and by tests on slab models. The method of analyzing nonlinear stress effects was based on the analogy between partial differential equations for an Airy's surface and for a homogeneous slab loaded at the edges. Solutions were obtained both by mathematical analyses and by experiments on rubber slabs deflected by twists and moments applied at the edges. Adjustments were made for cantilever elements varying radially in thickness from downstream to upstream. Supervisors during the stress studies were R. S. Lieurance for the trial load studies, F. D. Kirn for the nonlinear cantilever studies, and R. E. Glover for the nonlinear arch studies and special studies.

D. Part V, Bulletin 6, Technical Investigations - Model Tests of Arch and Cantilever Elements

It was desirable to obtain comparisons between cross-sectional models and the three-dimensional model of the entire dam. The cross-sectional models were performed at the University of Colorado in Boulder.

Cantilever model - The cantilever model was 3-inches thick and at 1:240 scale was the same scale as the 3-dimensional model of the dam. The depth, upstream, and downstream dimensions of the foundation were equal to the height of the cantilever.

Arch model - The purpose of the arch model was to obtain experimental measurements of strains and deflections in a thick arch element. Thin arches had been investigated in detail, but thick arches had not been thoroughly studied. A horizontal section at elevation 900 was selected for the study. Prior to these experiments, this thick arch had been investigated analytically and experimentally by slab analogy. The arch model was built at 1:120 scale.

E. Part VII, Bulletin 1, Cement and Concrete Investigations - Thermal Properties of Concrete

One of the major problems at Hoover was the prevention and removal of heat in the concrete due to the heat of hydration. The problem was compounded by the rapid construction and extraordinary size of the dam locking in temperatures that would take more than 100 years to dissipate. A series of radial and circumferential contraction joints were installed to control shrinkage of the concrete. For the dam to act as a monolithic structure, the joints must not open. However, the joints would open as the dam contracts from cooling of the concrete. Under this scenario, grouting the joints would have to be done over generations. Various methods were considered to remove the excess heat. This included low-heat cement and artificial cooling. Low-heat Portland cement was developed to reduce the heat of hydration by one-third and the temperature rise by about one-fourth. Investigations were performed to determine the effects of physical and chemical composition of the Portland cement on strength, temperature rise, and other properties. The design of the artificial cooling plan was based on the measured properties and mathematical theory of heat conduction. Knowledge base at the time did not provide accurate and applicable values for these properties, so investigations had to be performed. Considerable preliminary testing was necessary to develop apparatus and procedures for accurate thermal tests. Thermal property tests on concrete were also made for Gibson and Owyhee Dams. A method was developed for predicting thermal properties of concrete from these tests. Computed internal temperatures showed close agreement with measured test sections at Hoover and Owyhee Dams, where concrete was cooled by circulating water through metal pipes in the dam. Laboratory tests showed the effect on concrete temperatures of various rock types, water content, cement types, mix proportions, and age. The investigations were made at the Welton Street laboratory of Reclamation under the direction of H. S. Meissner, Arthur Ruetgers, and Robert F. Blanks.

F. Part VII, Bulletin 1, Cement and Concrete Investigations - Mass Concrete Investigations

The selection of the most suitable mass-concrete mix for Hoover Dam and the exact determination of its properties and qualities was one of the most important design problems affecting the economies of the design. The effects of aggregate size, test cylinder size, curing, and relative humidity on the strength, elasticity, permeability of the concrete; and on the bond strength of the horizontal lift surfaces were studied. Rocks as large as two people could lift, called plums, were used in the past in some dams. Reclamation felt more satisfactory results could be obtained with a maximum size aggregate able to fit in a mixer. A size of 9-inch maximum size was arbitrarily chosen to match available sources in the area. Little information existed on material properties using aggregate of this size; therefore, a comprehensive investigation program was initiated. Procedures for this type of concrete mix at the time would screen off any aggregate larger than 1.5 inch and test 6-inch diameter by 12-inch high concrete cylinders. No complete investigation had been performed to study the effect of the screening process.


Concrete cylinder test for Hoover Dam, AZ-NV

Information existed concerning the effect of various curing conditions on concrete properties, but no direct comparison could be made between strengths on concrete cured in the interior of a large dam and the conditions in a laboratory. Only permeability tests on concrete under low water pressures had been performed. Because of the height of Hoover Dam, concrete permeability tests for high water pressures were performed. Most of these tests were performed in the customhouse laboratory under the supervision of E. N. Vidal. Concrete dams are built in lifts. Subsequent concrete placements must be sufficiently bonded. Bond tests were conducted at the University of California Material Laboratory, Berkeley, California.

G. Part VII, Bulletin 2, Cement and Concrete Investigations - Investigations of Portland Cement

Although Portland cement had been used as a building material for more than a century, the unsuitability of the standard product for a structure as massive as Hoover Dam had become generally recognized at the time design work was begun. The main concerns were the heat generated during the hydration process and the shrinkage. The ideal cement for all purposes would be one which would permit the concrete to have no volume change subsequent to setting. Other desirable properties of mass concrete, which are dependent on the cement, are slower and better sustained hardening and adjustment to early stresses. In constructing the dam, contraction joints were provided at regular intervals in both the radial and circumferential directions. The structure was built in columnar blocks, approximately 50 feet square. The joints in between the blocks would allow for contraction of the concrete when it cooled.

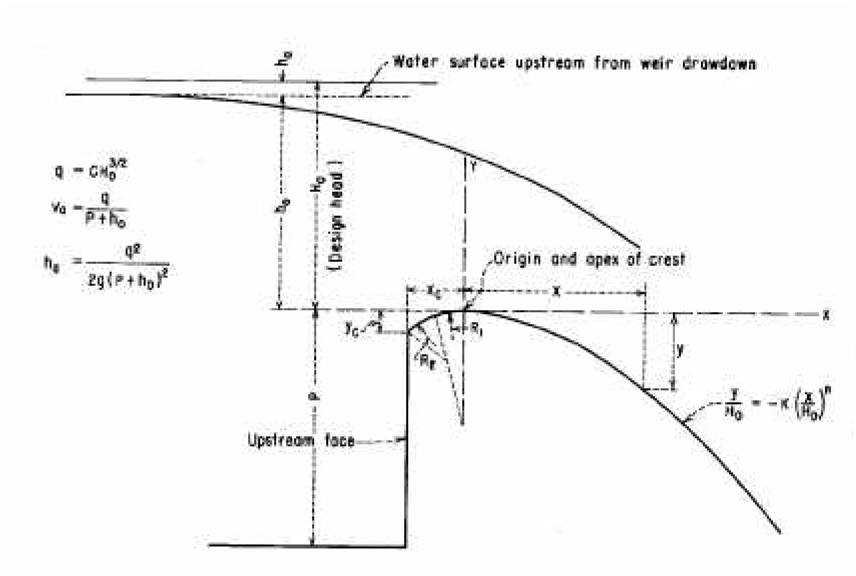
At the time Hoover was designed, little work had been done on the investigation of cements for mass concrete. C. P. Williams during construction of the Rodriguez Dam in Mexico first recognized the value of low-heat cement in reducing temperatures and reducing cracking. Late in 1930, Burton Lowther, a Denver consulting engineer, recognized the desirability of a low-heat cement and performed investigations for Reclamation at the Pierce Testing Laboratories in Denver. At the laboratories of the Bureau of Standards, Washington, D. C., preliminary tests were made of 49 commercial cements, selected from various parts of the United States. The work begun in Washington was continued and greatly expanded in the Engineering Materials Laboratory of the University of California at Berkeley. Some specimens cast and tested were concrete, but the majority were mortar or neat cement. Concurrent with and supplementing the investigations at Berkeley were the investigations made in the laboratories of the Bureau of Reclamation in Denver. Unlike the Berkeley test, most of the tests in Denver were made on concrete specimens rather than mortar specimens.

In summary, it is safe to say that the shear size of the Hoover Dam project, and the associated need to overcome many shortcomings in the design, analysis, and construction of concrete dams up until that time, lead to significant advancements in the state-of-the-art, ultimately to become the state-of-practice. This project, perhaps more than any other, came to represent the Bureau of Reclamation’s world renowned expertise.

VI. Hydraulics for High Concrete Dams

Without question, a major breakthrough in the understanding of high-head, high-velocity spillway designs resulted from the Boulder Canyon Project and construction of Hoover Dam. Between 1928 (authorization of the Project) and 1948 (completion of Project documentation), extensive research formed the “benchmark” for present-day spillway designs and analyses. The unprecedented size of the spillways (each with design capacity of 200,000 ft^3/s and a maximum average velocity approaching 175 ft/s) for Hoover Dam was the motivation to initiate a comprehensive research program. Of particular note, was the research and development of methods to design the “ogee” spillway crest, which is still used for spillway designs around the world. Prior to this research, methods of estimating the “under-nappe” of a jet of water moving over a sharp crested-weir were had been based on approximate observations made by M. Bazin in the late 1800's, and typically used a vertical upstream face on the spillway crest. The shape of the under-nappe defines a minimum shape or profile for the spillway flow surface. Unless the flow surface matches or is flatter than the under-nappe, sub-atmospheric pressure can occur, possibly leading to reduced stabilizing tailwater backpressure, increased cavitation potential, or vibrations.

The Boulder Canyon Project hydraulic research expanded on Bazin's methods and developed design tools, which can still be found in Reclamation's Engineering Monograph (EM) No. 9 and Design of Small Dams. The design tools provide considerable flexibility and methods to: (a) determine the spillway ogee shape required to best fit the under-nappe of the overfalling stream for any practical condition of design, (b) derive the nappe shape due to varying approach velocities, (c) determine the coefficient of discharge for overfall dams (or spillways) with vertical, sloping, overhanging and offset upstream faces, (d) determine effects on coefficient of discharge due to different crest shapes with and without control gates, including the effects of adjacent terrain, piers, and position of gates, and (e) determine the effects on the coefficient of discharge due to downstream submergence.



Modern Ogee Spillway Crest Configuration

A second major breakthrough in hydraulic design for high dams occurred in 1958 with the first printing of Reclamation's EM No. 25, Hydraulic Design of Stilling Basins and Energy Dissipators. This publication summarized 23 years of research and design experience, and provided a practical design tool for sizing stilling basins. Since that initial printing this EM has been updated and was last reprinted in 1984. Until the development of this EM, attempts to generalize data from hydraulic model studies and resulting designs lead to inconsistent results. To resolve this, a research program was undertaken, starting with observing all phases of the "hydraulic jump". With an understanding of this phenomenon, it was possible to develop

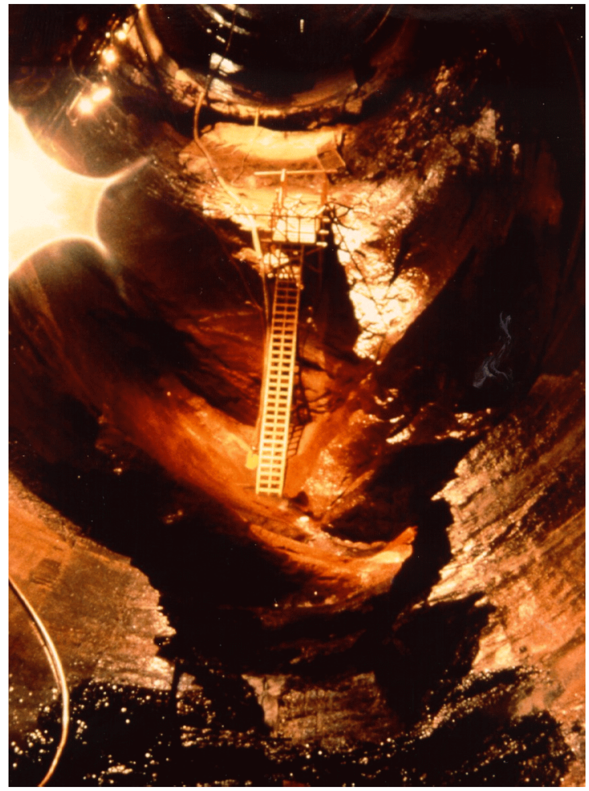
Basin X (tunnel flip bucket), Spillway Discharging Approximately 27,000 ft^3/s - Glen Canyon Dam, AZ practical and common aspects of energy dissipation designs. This EM documents that effort, and provides general design rules and procedures for 10 stilling basin or energy dissipator types, which in some cases eliminates the need for hydraulic model studies. It should be noted that hydraulic model studies still play an important role in the design process. They are used to optimize the structure's size, account for non-symmetrical approach and exit conditions, and to evaluate unusual flow conditions in or through the structure. Three types of stilling basins and energy dissipators have been primarily used for spillways associated with high concrete dams. These include:

1. Basin V (sloping aprons) - This basin relies on a hydraulic jump to dissipate energy. The downstream basin slopes gently downstream. Designs that used Basin V stilling basins included Shasta, Canyon Ferry, Olympus, Friant, and Keswick Dams.

2. Basin VII (slotted and solid buckets) - As with Basin V, this basin also relies on a hydraulic jump to dissipate energy. However, the downstream basin is curved up with a lip at the downstream end. Designs relying on Basin VII stilling basins included Grand Coulee Dam, (solid bucket); and Angostura Dam (slotted bucket).

3. Basin X (tunnel flip buckets) - Unlike the basin V and VII, a hydraulic jump is not initiated. This is an energy dissipater that projects the exiting jet into the air, spreading and aerating the jet before it impinges into the tailwater. Basin X energy dissipators were used for Glen Canyon, Hungry Horse, Yellowtail, and Flaming Gorge Dams.

A third major advancement in evaluating hydraulics for high concrete dams involved the understanding of cavitation. Although Reclamation had investigated cavitation damage and implemented repairs since 1941, the understanding and methodology to adequately mitigate cavitation damage was not fully developed until after significant cavitation damage occurred at Glen Canyon and Hoover Dam tunnel spillways as a result of flooding in 1983. Prior to this, standard practice was to specify very stringent concrete finishes for flow surfaces associated with discharge velocities greater than 75 ft/s. The concrete finishes for these flow surfaces were very difficult to achieve in the field. A more effective method had actually been employed in 1961 and 1969 with the installation of aerators to address the cavitation damage which occurred at Grand Coulee Dam outlet works tubes and spillway chute, and the Yellowtail Dam spillway, respectively. The installation of the aerator for Yellowtail Dam spillway is thought to be the first of its kind, and after which, it was noted that aerators were being installed worldwide. It is interesting to note that research had already illustrated the effectiveness of extremely small quantities of air entrained in flowing water in significantly reducing the tendency for cavitation damage. However, it was not until the mid- to late-1980's that sufficient research, design, and experience had been gained to change Reclamation's approach to mitigating cavitation potential. Cavitation was found to be the result of formation and collapse of vapor cavities at abrupt changes in geometry of the flow surface. Resulting from an eight year effort, EM 42, Cavitation in Chutes and Spillways was published in 1990, providing common-sense guidance on how to identify and mitigate cavitation potential. Two important developments include: (1) generalized guidelines and tools were developed to assess the potential degree of cavitation, and to develop preliminary aeration designs, and (2) concrete finishes (surface textures) were de-coupled from concrete tolerances (surface offsets and irregularities), recommended surface tolerances were revised to be more achievable in the field, and these tolerances were linked to cavitation indices. These indices are a function of the fluid velocity and pressure, and empirically give an indication of the potential for cavitation.


1983 Cavitation Damage in the Left Spillway Tunnel. The “big hole” extends approximately 27 feet below the tunnel invert - Glen Canyon Dam, AZ

Today, as standard practice in the technical evaluations of existing and new spillways, the cavitation potential is evaluated by first evaluating the cavitation index (s) profiles at different discharges. Based on cavitation index profiles, the required surface tolerances are determined as a function of the minimum value of cavitation index. If the cavitation index is less than 0.2, cavitation would be expected, and the effects of changing the spillway geometry on the cavitation index should be evaluated. If low values of the cavitation indices cannot be raised by changing the geometry, a concept change or an aeration device should be considered. Using these procedures, aerators have been installed in the spillway tunnels for Glen Canyon, Flaming Gorge, Hoover, and Yellowtail Dams.

VII. World War II Era - Large Gravity Dams

In the 1930's, the United States was hungry for electric power, and this became even more important to fuel war production factories following entry into World War II in 1941. The technology developed during the design and construction of Hoover Dam was available to construct large concrete dams and associated hydroelectric powerplants. In order to tap the energy reserves of large and wide rivers, it became necessary to construct gravity dams. Two of the largest of these, Grand Coulee and Shasta Dams, were constructed by the Bureau of Reclamation in the late 1930's and 1940's. These dams became engineering landmarks, and have been studied and emulated by other countries around the world. During this time, Dr. J.L. “Jack” Savage served as Chief Design Engineer. His office in Denver then was the foremost engineering office in the world for water resource heavy construction projects. Dr. Savage gained world-wide renown for his work with the Bureau of Reclamation, and received may honors and awards. He was reputed to be modest to an extreme, and was of such character as to readily receive the loyalty of his capable organization.

Grand Coulee Dam, Forebay Dam and Third Powerplant, WA

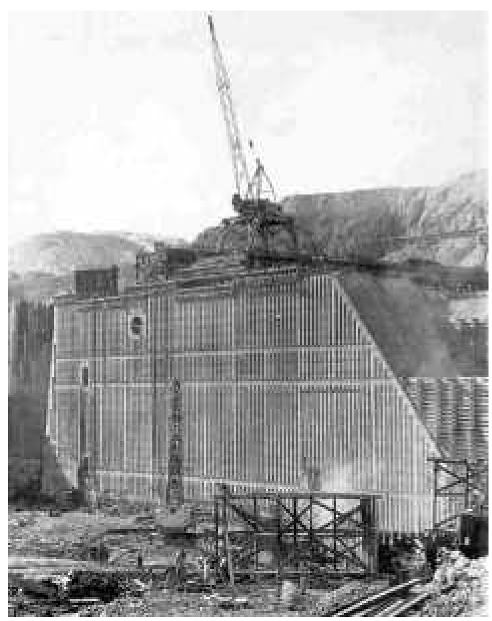
The original design of Grand Coulee Dam called for a low dam to be built to elevation 1116 with the left power plant included. It would accommodate a future dam raise and expansion of the powerplants, but originally would not provide irrigation water. The specifications were issued, the contract awarded, and the Notice to Proceed issued on September 25, 1934 for the low dam concept. Shortly after the construction activities began, renewed pressure came from the local agricultural constituents for the high dam. They caught the ear of President Franklin D. Roosevelt during his August 4, 1934 visit to the site. A reevaluation of the economics and technical issues associated with raising the dam indicated substantial benefits in going directly to a high dam. By June 5, 1935, a major change order was issued, increasing the excavation and changing the shape and details of the dam to allow immediate construction of a high dam to elevation 1311 through a second contract. The dam would be a gravity structure nearly a mile long and 550 feet high, with a downstream slope of 0.8:1, and a central spillway section controlled by drum gates capable of releasing 1,000,000 ft^3/s. Water would be pumped from the Columbia River to a lake formed by the Grand Coulee, a basin eroded by the River during the ice age when ice blocked the main course of the river. In January 1942, about a month after the Japanese invaded Pearl Harbor, a contingent of the U.S. Army took up quarters in Mason City and performed guard duty at the dam due to concerns about a possible enemy thrust into the area. All efforts were concentrated on getting the power online to supply energy to the aluminum plants and shipyards.

Diversion of such a large river posed many problems, but a series of cofferdams, and diverting flows over the low blocks in the dam allowed the construction to proceed. Landslides in the fine-grained deposits from the ice age mantling the river banks were also problematic. Stabilization included flattening slopes, installing drainage, and temporarily freezing the soil. The dam was founded on hard granite scoured by the pre-ice age river. As had become standard practice, foundation grouting and drainage were constructed. Three-dimensional trial load twist analyses, fully developed during the design of Hoover Dam, were performed for the high gravity dam design. Due to stress concentrations in the portion of the dam adjacent to the sharply rising abutments and concerns for potential cracking, vertical “twist slots” were designed for the

abutment sections to give the structure some flexibility to adjust to loads. Five twist slots were constructed, two on the left side and three on the right side. The slots were initially filled with sand. After the reservoir had filled to elevation 1150, the
sand was removed and the slots filled with concrete.

Construction of Grand Coulee Forebay Dam, WA (Note unkeyed contraction joint)

Low heat cement was used for the project. It had a slower set time delaying stripping of the forms, but lower heat of hydration than conventional cement which was a great bonus in cooling the concrete and keeping cracking to a minimum. The concrete was made of aggregate, cement, and water. No admixtures, other than limited quantities of calcium chloride to accelerate the set, had become acceptable at that time. Two mixing plants were constructed, one on each side of the canyon, and at the peak of production, 20,684 yd^3 of concrete were placed in 24 hours on May 29, 1939. The rock and concrete surfaces were thoroughly cleaned for placement of concrete using wire brushes, sand blasting, and water jets. The concrete was placed in 5-foot lifts and about 50-foot square maximum size blocks. At least 72 hours was required between successive lift placements. Cooling coils were placed on the lift surface, and drain forms installed. Then a ½-inch-thick layer of mortar was placed on the surface to provide a good bond. Concrete with a 2-inch slump or less was delivered in four-yard buckets using small trains running on a trestle and cranes. The concrete was placed in one foot layers and thoroughly consolidated with electric and pneumatic vibrators. The exposed surfaces were kept wet for 14 days. River water was pumped through the cooling coils to cool the concrete. An evaporative cooling tower was eventually installed to enhance the concrete cooling. The concrete was cooled to about 45 degrees Fahrenheit, and then the transverse keyed contraction joints, spaced at 50 feet, were grouted. Reclamation's 5,000,000 lb. testing machine was installed in the Denver laboratory at the U.S. Customs House during the period of dam construction to permit testing the strength of large aggregate concrete, using cylinders up to 36 inches in diameter.



Construction of Grand Coulee Dam, WA (Note keyed contraction joint)

The Forebay Dam and Third Powerplant were completed in 1974, and greatly increased the power generating capacity of the project. Two concrete mixes were used for construction of the Forebay Dam; a richer mix for exterior surfaces and a somewhat leaner mix for the interior mass concrete. Fly ash and air entrainment were used in all concrete. The fully automatic batching plant had provisions for handling five aggregate sizes ranging to 6-inch maximum, and a refrigeration plant for chilling water and making ice to cool the mix to the required 40 to 50 degree Fahrenheit placement temperature. All concrete was membrane cured. Vertical contraction joints normal to the axis were spaced at alternating distances of 50 and 70 feet, the large spacing required to accommodate the 40-foot-diameter penstocks. Artificial cooling was performed in the lower portions of the blocks. The contraction joints contain water stops, but only the lower portions were grouted, presumably to stabilize the sections of the dam that contain the penstocks. However, the more important consideration is that the contraction joints keyed. This allows each block to adjust to movements individually, but also reduces load transfer between adjacent monoliths in the case of local instability.


Shasta Dam, CA

Construction of Shasta Dam in northern California overlapped with construction of the original Grand Coulee Dam. At the time, Shasta Dam was second only to Grand Coulee in volume, and second only to Hoover in height. The dam is on the Sacramento River in northern California, and is the cornerstone of the Central Valley project. Although curved in plan to match the site conditions, the dam was designed as a gravity dam, with a downstream slope of 0.8:1. By this time efficient placement and cooling of large volumes of concrete could be readily achieved, due largely to the development that occurred during the design and construction of Hoover Dam. Construction methods were nearly identical to those at Grand Coulee Dam. As an interesting note, two generators lay idle at Shasta Dam in the early days of World War II, with no prospect for immediate use. They were shipped and installed at Grand Coulee Dam, providing power during the critical war years, and then returned to Shasta following the war.

It should be noted that during this period of time the effects of alkali-aggregate reaction (AAR) came to the forefront. A chemical reaction between the alkali in the cement and certain types of aggregates causes expansion of the concrete usually leading to cracking, and in cold climates the damage can be exacerbated by freeze-thaw mechanisms as water enters the cracks. Extensive cracking and deterioration at Parker Dam in Arizona, and American Falls Dam in Idaho led the Bureau of Reclamation to conduct studies into the phenomena beginning about 1941. Petrographic examination of aggregates became the primary means of identifying potentially reactive aggregates in about 1941. The limitation of alkalis in the cement to less than 0.6 percent as a means to control AAR was first published in the Fourth Edition of the *Concrete Manual* by the Bureau of Reclamation in October 1942. Investigations into the effects of pozzolans to reduce alkali- aggregate reaction were begun in the early 1940's. Using 20 percent Class F or N pozzolans as a replacement for cement became standard practice for the Bureau of Reclamation in about 1970. This not only reduces the cost of the cementitious material, but also provides additional protection.

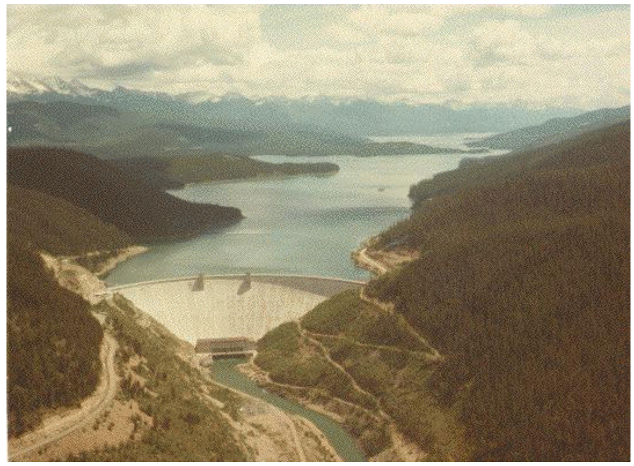

Cracking at Friant Dam due to Alkali-Aggregate Reaction, Friant Dam, CA

By this time, deterioration of some concretes in cold climates had been noted, and was described in general terms as durability. The problem was freeze-thaw damage, whereby water present in the saturated cement expands upon freezing, exerting pressures that far exceed the tensile capacity of the paste, causing cracking and ultimately failure of the concrete after repeated cycles. It was found that high strength concrete made with good quality aggregates and low water to cement ratios generally had better durability. However, experience accumulated during the 1920's and 1930's suggested that other factors also contributed to whether a concrete was susceptible to freeze-thaw damage. The Bureau of Reclamation began testing concrete for freeze-thaw durability in about 1937 with the development of accelerated freezing-thawing test apparatus. The first studies of standard concrete mixes of the time indicated that failure usually occurred after about 150 to 200 cycles. Formal studies of the effects of an air-entraining admixture performed in 1942 reported an increase in the number of cycles to 400 to 450. However, the Fourth Edition of Reclamation's *Concrete Manual* provided no reference to air entrained concrete. Due to World War II, this information was not published until 1949 in the Fifth Edition of the *Concrete Manual*. Air-entrained concrete as a means to increase concrete durability has been standard practice since.


Freeze-Thaw Damage on Downstream Face of Deadwood Dam, ID (Note that damage is near contraction joint due to leakage)

VIII. The Post-War Boom - Developments Continue

Following World War II, the country entered into a boom period. The demand for power was high, and the developments that occurred with the building of large concrete dams and associated powerplants such as Hoover and Grand Coulee were put to use in quickly building several more monumental concrete dams and powerplants, such as Glenn Canyon (a 710-foot high thick arch dam on the Colorado River), Yellowtail (a 525-foot high arch dam on the Bighorn River in Montana), and Flaming Gorge Dam (a 502-foot high arch dam on the Green River in Utah). The first of these large post-war concrete dams was Hungry Horse.


Hungry Horse Dam, MT

Hungry Horse Dam, constructed in 1948-1953, is a concrete arch structure that has a structural height of 564 feet and a crest length of 2,115 feet at crest elevation 3565.0. The dam is located on the South Fork of the Flathead River in northwestern Montana, south of the southern border of Glacier National Park. The dam impounds a reservoir containing 3,467,000 acre-feet of storage at elevation 3560.0. The reservoir provides the benefits of power generation, flood control, irrigation, river regulation for fisheries, and recreation.

Hungry Horse Dam was designed and analyzed by trial load methods. (Though not used for Hungry Horse, physical model studies were still in use, and were performed later for Glen Canyon and Morrow Point Dams.) The analyses include the stage construction of varying reservoir elevations and grout zones. Concrete was cooled by embedded cooling pipes to 38E F. Original designs called for the vertical radial contraction joints to be 50 feet apart, but based on temperature studies, an 80-foot spacing was used. One cross canyon contraction joint was used across blocks 10 to 23 at alternating distances of 134 feet and 186 feet from the axis. The vertical contraction joints have shear keys. Formed drains were constructed at each contraction joint and at 10 feet on centers across the dam. Collected drainage flows by gravity into a sump consisting of two pumps each discharging 500 gallons per minute.

The dam consists of 27 blocks numbered from 2 on the left abutment to 28 on the right abutment. Lifts were 5 feet in height. There were different concrete mixes for the interior and exterior (5 foot minimum to 9 foot average exterior concrete thickness on the faces and crest roadway) of the dam consisting of cement, fly ash, and 6-inch maximum sized aggregate. Flyash used as pozzolan helped reduce the heat of hydration while providing long term strength gain.

Another major development of the post-war era was the use of air-entraining admixtures to increase the durability of concrete to freeze-thaw damage. Problems with air entrainment persisted throughout construction of Hungry Horse Dam, but were perfected at later structures. Early stripping of forms was a major cause of surface damage.

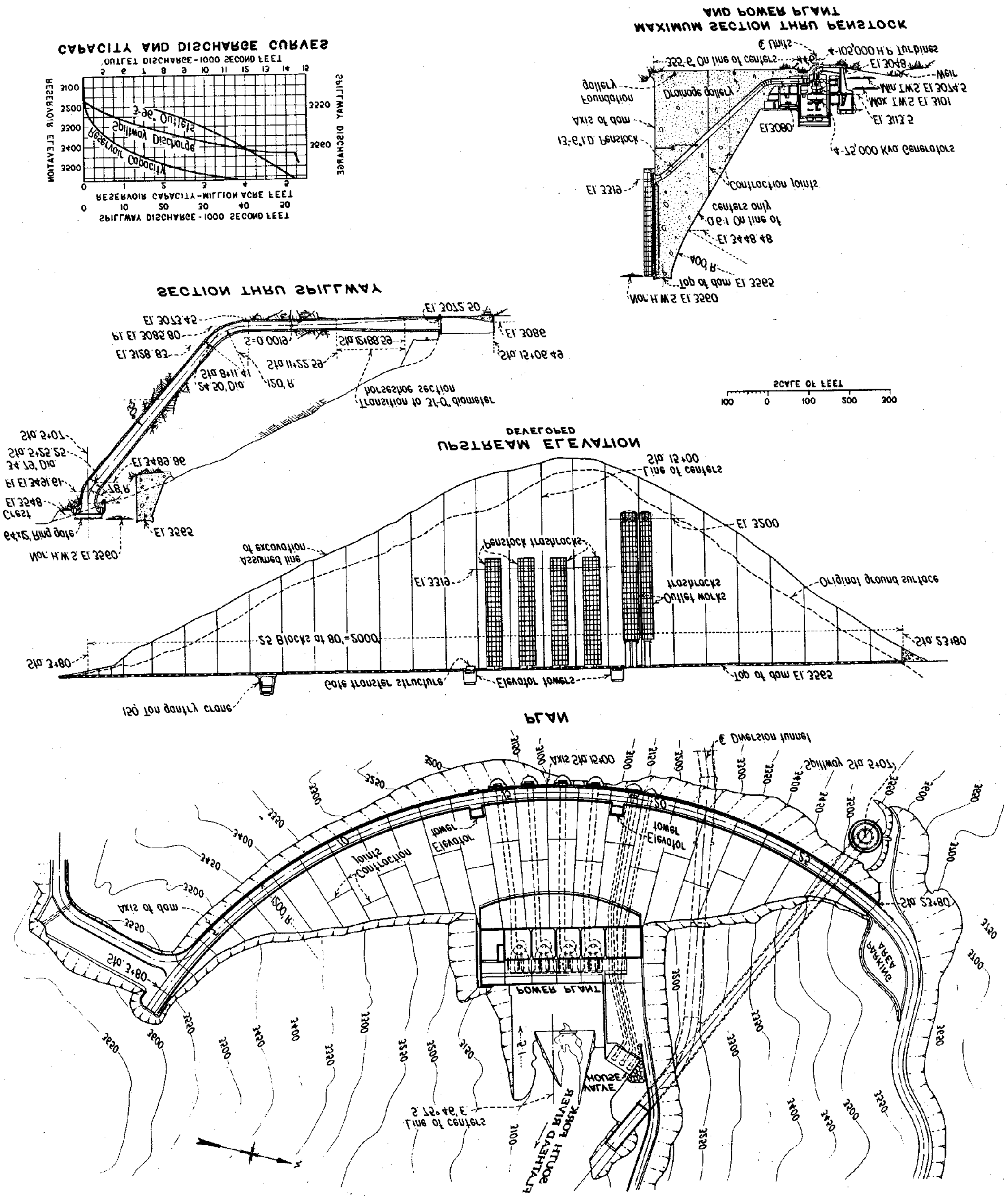
Extensive instrumentation systems had become standard by this time. The dam has 7 lines of uplift measurements at the dam to foundation contact, 3 plumb lines, and flow measurements from drain holes in the right abutment. Deflections are measured with 3 plumblines located in blocks 8, 17 (crown), and 24. The dam has permanently shifted upstream about 0.3 inches since 1962. The dam moves a total less that 0.4 inches season to season.

The dam was constructed close to current day standards with vertical contraction joints, formed drains at 10 foot centers in the concrete, foundation drains at 10 foot centers in the foundation, foundation grouting, artificial cooling of the mass to 38E F and contraction joint grouting, cleaning of the lift lines and dam to foundation contact for bond, and concrete strengths (tested during construction) averaging over 4000 lb/in^2. There is radial cracking on the crest in blocks 4 and 24 progressing 30 feet down on the downstream face and into the roadway gallery. Radial cracking on the crest is probably thermal induced cracking because the contraction joints are 80 feet apart and not the typical 50 feet.

The spillway at Hungry Horse Dam is a concrete-lined tunnel with a morning-glory intake on the right abutment designed for a maximum discharge capacity of 53,000 ft^3/s for a reservoir elevation at the crest of the dam (elevation 3565.0). The normal high water surface is 5 feet lower than this maximum with the ring gate in the raised position. The spillway was designed using two laboratory models and approximately 200 tests. Subatmospheric pressures were reduced to very low levels by shaping the crest profile, developing an efficient venting system, increasing the lower bend radius from 55 to 120 feet, and providing a guide vane for the upper bend together with a pier on the spillway crest. The only difference in the actual spillway was the elimination of the vane and pier because of difficulty in construction. The venting system vents the undernappe from the crest structure with nine 8-inch pipes at 30 degree centers around the crest and vents the crown of the spillway tunnel in the upper bend at elevation 3514.0 with an additional inlet. Air is supplied by a 6-foot square air inlet tunnel in the right abutment. With 53,000 ft^3/sec discharge, velocities of the water at the outlet portal are computed to be between 132 and 146 ft/sec. The spillway crest is controlled by a 64-foot diameter buoyant ring gate having a maximum lift of 12 feet from elevation 3548.0 to 3560.0. A de-icing system using compressed air bubblers prevents ice forming on the gate. Spillway discharge varies from free-flow discharge at low heads to orifice-flow discharge at higher heads.

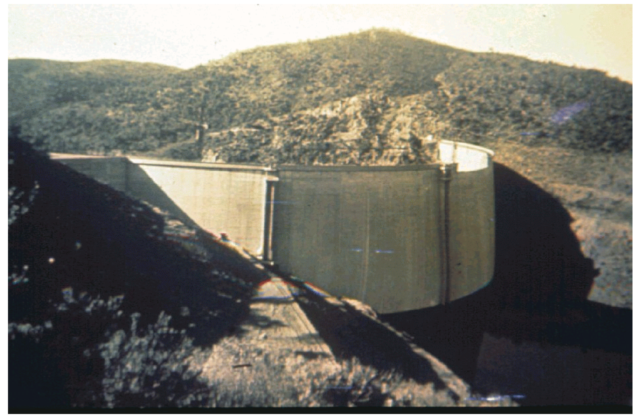
Several precautions were taken during construction of the spillway to assure accurate alignment and smooth concrete surfaces. Even construction joints were eliminated in the vertical curve and deflector sections to avoid offsets at the joints. A 50 degree inclined shaft was chosen over a vertical shaft for economic reasons and ease of excavation, to cross bedding planes at right angles and confine overbreaks to the upper right-hand quadrant of the shaft because of one of the joint systems. After placement of the tunnel lining, the surrounding rock was thoroughly grouted using pressures varying between 125 lb/in^2 and 150 lb/in^2. Irregularities in the lining were eliminated by grinding, sandblasting, hand-stoning with a fine-grit carborundum stone, and then final grinding after 7 days of cure. The vertical bend and deflector sections were placed without construction joints and cooled with river water pumped through cooling coils. Rather extensive repairs of the concrete surfaces in the spillway tunnel were required because of retractions and bulges in the wood forms. Concrete was placed in above-freezing temperatures and curing was by hand sprinkling.

The foundation at Hungry Horse Dam is the Siyeh limestone formation with beds ranging in thickness from a few inches to several feet. The average strike of these beds is N38W and an average dip of 30NE which is upstream and into the right abutment. Several faults were present in the foundation which required excavation and backfill concrete treatment. Foundation grouting and drainage were typical for the time. However, an unusual foundation treatment was used for the first time. A clay seam along bedding was discovered in blocks 11, 12, and 13. It was decided to wash out the clay with water and air at less than 30 lb/in^2 pressure and backfill with grout rather than to remove the 7,100 cy of rock above the seam. At some point, pressures of 250 lb/in^2 were used. The seam was excavated above fault 3. The treatment was verified to be effective by extracting core and inspections down calyx sized holes.



Plans and Sections of Hungry Horse Dam and Spillway

IX. The Failure of Malpasset Dam - Rock Mechanics and Foundation Design Develops


Malpasset Dam, Cannes District, France

Although several concrete dams failed due to foundation deficiencies during the early years of concrete dam construction in the United States, it wasn’t until the failure of Malpasset Dam in 1959 that the profession recognized a need for more rigorous foundation investigations and analytical design methods. Malpasset dam was a 216-foot high thin arch dam completed on the Reyran River upstream of Frejus in the Cannes District of France. The reservoir had a capacity of 41,700 acre-feet. Although the foundation contact was blanket grouted with 16-foot deep holes, a grout curtain was considered unnecessary due to the low permeability of the rock. No drainage had been provided in the dam or foundation, and no instrumentation, other than surface measurement points, was installed. The foundation consisted of metamorphic schists. Heavy rainfall occurred during the fall of 1954 shortly after completion of the dam, and by mid-November the reservoir was within 17 feet of the normal maximum level. At that time operators discovered a trickle of clear water about 60 feet downstream of the dam on the right abutment. Cracks had been seen in the concrete apron at the toe of the dam, but no one knew when they first appeared. Another intense rainstorm began on November 28, and by December 2 the reservoir was full and the outlet was opened. At 8:45 p.m., the caretaker left the dam without observing anything unusual. At 9:10 p.m. the dam failed suddenly, causing total destruction along a 7-mile course to the Mediterranean Sea.

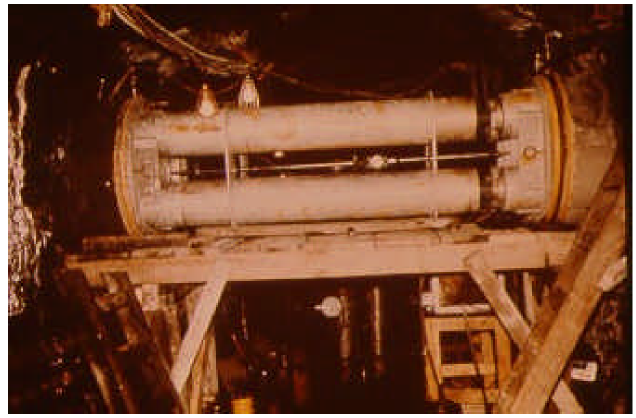

Malpasset Dam Failure, Cannes District, France

Analysis of the displacements of the dam remains showed that the left side of the dam and underlying foundation lifted and rotated as a monolithic unit about a vertical axis located where the crest met the right abutment. Conventional structural analyses using a wide range of material properties showed concrete stresses were well within strength parameters, and did not explain the failure. Arch buckling analyses also indicated an ample margin of safety. The failure left an upstream dipping fault zone and downstream dipping foliation plane exposed on the left abutment, intersecting below where the dam once stood. The measured movements and post-failure evidence pointed to abutment sliding on the fault as the cause of failure. Dr. Pierre Londe developed three-dimensional limit equilibrium analysis techniques to evaluate the stability of a dihedral wedge formed by the fault, the shear, and a third joint release plane. The stability of the wedge was evaluated under loads consisting of dead weight, water uplift forces on each plane, and the thrust from the dam. Instability was explained by this analysis when large uplift forces were assumed to develop on the foliation shear.

Thus, the science of rock mechanics was applied to concrete dam foundations. Shortly after this, in the late 1950's and early 1960's during the design of Yellowtail, Glen Canyon, and Morrow Point Dams, the Bureau of Reclamation began further developing rock mechanics methods in application to concrete dam foundation design and analysis. Large scale in-situ tests were developed for determining rock mass deformability properties. Exploratory drilling and geophysical testing were performed to evaluate foundation conditions, and careful attention was paid to major discontinuities within the rock. However, it was not until the designs for Auburn Dam were underway in the late 1960's that the foundation exploration, analysis, and design were coherently integrated. Under the direction of Mr. Louis R. Frei, Mr. James S. Legas, and Mr. J. Lawrence Von Thun, world class foundation investigations, testing, evaluation, design and treatment occurred at the Auburn Damsite. Although Auburn Dam was never completed, this work was an enormous contribution to the profession, and formed the basis for future evaluations within the Bureau of Reclamation.

The Auburn Damsite consists of complex metamorphic geology. The basic rock type is a dense amphibolite, but numerous faults and talc zones cut the rock, and metasediments occurred within the foundation. Careful diamond core drilling using split inner tube core barrels, trenching, and excavation of exploratory tunnels and drifts was performed to define the geologic conditions. The results of this exploration were portrayed on geologic plan, section, and structural contour maps to provide a complete three-dimensional picture of the foundation. Weathering profiles and fracture density characterization were used to define the foundation excavation to suitable rock. It was recognized that the rock deformation properties were key in determining how load was distributed to the foundation from the dam, and that jointing and discontinuities within the rock had a pronounced effect on these deformation properties. In-situ deformation testing was performed in the exploratory tunnels and drifts. Despite the large size of the tests, it was recognized that they still represented a small point in the foundation rock. Methods were therefore developed to extrapolate these results to the rest of the foundation. From this, the deformation properties of the foundation were defined for input to finite element and trial load structural analyses of the concrete arch dam.

Seepage analyses were performed to evaluate potential foundation uplift pressures. Exit gradients at fault and talc zones near the toe of the dam were analyzed, and testing was developed to determine critical exit gradients where piping of these zones would initiate. Potential modes of instability were identified by evaluating discontinuities (faults, shears, joints, foliation planes, talc zones) within the foundation. "Failure mode assessment" as it is sometimes called, was developed fully in the rock mechanics arena, and has been a valuable contribution to other areas of engineering. Foundation blocks formed by discontinuities that intersected beneath the dam, with the intersection or one of the planes daylighting downstream, were analyzed using limit equilibrium techniques. The shear strength of the critical potential sliding planes was evaluated by laboratory and in situ testing of samples from the appropriate faults, talc zones, and joints. Arch thrust from gravity, reservoir, and temperature loads; dead load of the foundation blocks; uplift on the planes that formed the blocks; and earthquake loading were all considered in the evaluations. Finally, foundation treatment, in the form of excavation of the weak zones and replacement with mass concrete, was designed based on the results of all the studies. In some cases the treatment was controlled by the need to develop a smooth deformation pattern or transfer of load across discontinuities. In others, the treatment was controlled by the extra shear strength needed for stability, or by the need to reduce exit gradients.



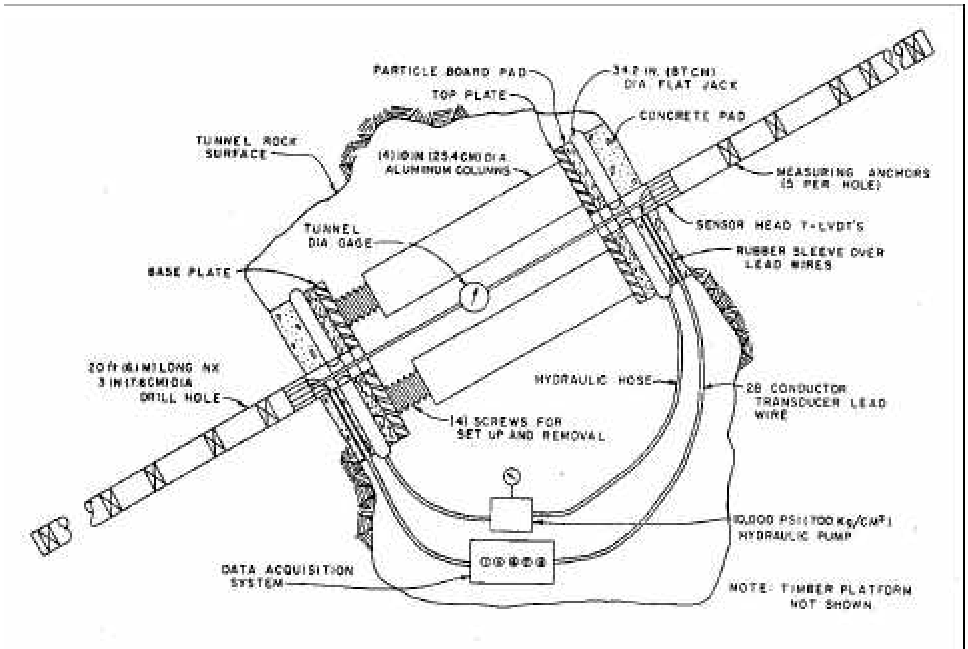


Photo and Schematic of Uniaxial Jacking Test Performed at the Auburn Damsite to Measure Deformation Properties of Foundation Rock Mass

Although improvements to the analysis methods have been made over the years including better methods for evaluating seismic stability, the basic evaluation process remains essentially the same as that developed at the Auburn Damsite. Many concrete dam foundations have been evaluated using these procedures. Detailed foundation rock mechanics analyses are now an important aspect of the standard practice for evaluating concrete dams.

X. The Double-Curvature Arch - A New Standard for Efficiency

Beginning in about the early 1960's a new concept for shaping arch dams found its way to the Bureau of Reclamation. This shape, termed "double-curvature" provided for more efficient distribution of loads within the structure and to the abutments. A double-curvature arch is curved in plan view and section view. This results in more of a "bowl" shape to the structure. The undercutting at the heel of the dam that results from this shape, and the inward curvature on the downstream face, eliminate areas where tensions typically develop in arch dams.


Morrow Point Dam, CO

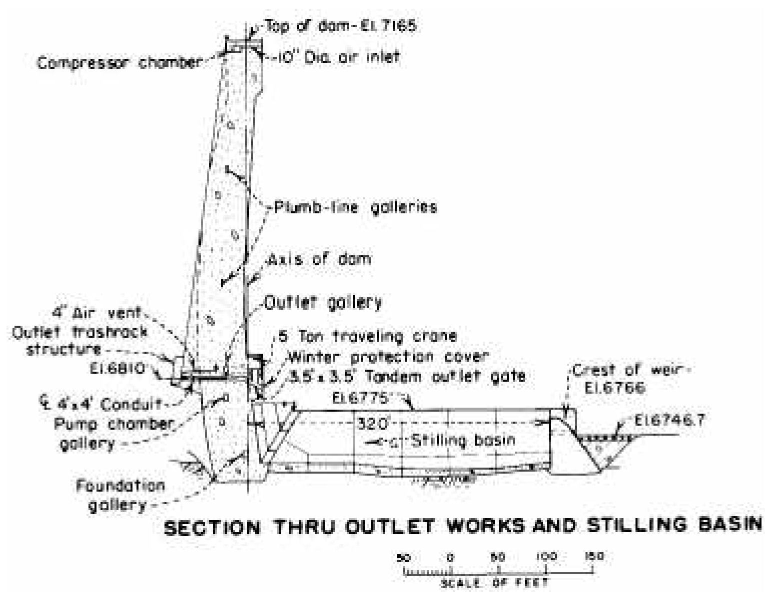
The first double-curvature dam constructed by the Bureau of Reclamation is Morrow Point Dam. The dam has a structural height of 468 feet and a crest length of 724 feet. The dam is a variable-center arch structure with an axis radius of 375 feet. The crest of the dam at elevation 7165 carries a 12-foot-wide roadway. Storage in the Morrow Point Reservoir is 117,190 acre-feet at the top of active conservation.

In addition to being Reclamation's first double-curvature arch dam, the project also boasts Reclamation's first (and only) underground powerplant. The powerplant chamber is tunneled into the canyon wall in the left abutment about 400 feet below the ground surface. Two 13.5-foot-diameter steel penstocks carry flow to the powerplant, which contains two 86,667-kilowatt generators driven by two 83,000-horsepower turbines.

Because Morrow Point Dam was the first double-curvature thin arch dam built by Reclamation, the geologic exploration program was one of the most extensive programs ever carried out. The geologic data was developed through a comprehensive investigation which included detailed geologic mapping, diamond core drilling, excavation of five exploratory tunnels, examination of drill holes by television, and seismic surveys. Geologic studies were also coordinated with horizontal and vertical insitu jacking tests and with Whittemore and borehole strain gage measurements. However, failure mode assessment and foundation stability analyses were not part of the original foundation studies.

Morrow Point Dam is located in a narrow section of the Black Canyon of the Gunnison River with very steep canyon walls and many overhangs. The rock encountered at the damsite consists of alternating lenticular and irregular beds of biotite schist, mica schist, micaceous quartzite, and quartzite, all of which were intruded by granite pegmatite ranging from small veinlets to massive intrusions. The quality of rock type varies considerably, the hardest being the granite pegmatite and the quartzite with variations of hardness down to the weaker biotite schist.

The damsite is located on the axis of a synclinal fold which plunges gently to the south (or into the left canyon wall) at about 5 degrees. The fold is expressed by the attitude of foliation or bedding which dips toward the axis from both upstream and downstream. The rock contains stress relief jointing which generally parallels the canyon walls and dips steeply toward the river, probably resulting from unloading through the removal of overlying rock by river erosion. Another indication of stress relief is an apparent halo of fractured rock which extends to a depth of about 80 feet beneath the valley floor.



Morrow Point Dam, CO

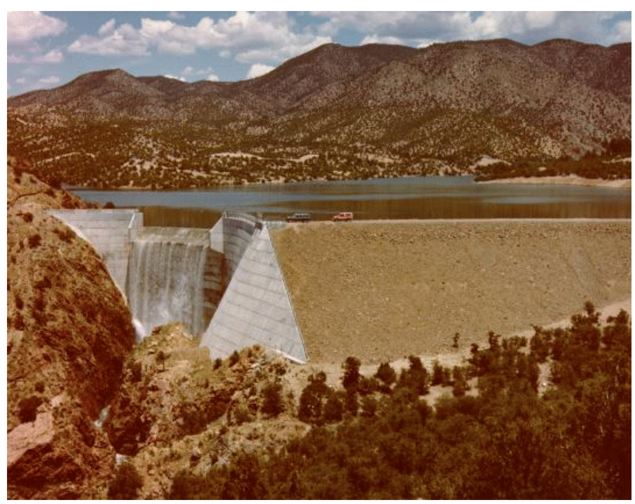
The analyses were very thorough since the design and layout requirements went beyond the state-of-the-art of that time. The dam was mathematically modeled and analyzed using the Trial Load Method of Analysis when subjected to static load and was further analyzed using the computerized adaptation of the Trial Load Method (ADSAS - Arch Dam Stress Analysis System) to refine the design and layout for nine different loading conditions, including seismic loads, construction loads, various temperature and grouting conditions, and the as-excavated foundation layout. In addition, the dam was analyzed by the use of physical models as a check to the mathematical modeling process. One model of the dam and foundation was prepared by Reclamation and the other was made by the Laboratorio Nacional de Enenharia Civil of Portugal. All the analyses indicated the dam could safely withstand any of the loading conditions applied.

The contract for the construction of Morrow Point Dam and Powerplant was awarded to a joint venture of the Al Johnson Construction Company and Morrison-Knudsen Company on May 14, 1963, with construction completed on May 24, 1968. In general, the dam and powerplant were constructed in accordance with the designs and specifications with only a few complications arising requiring changes in the planned construction.

Open stress relief jointing, especially in the left abutment, caused several small rockslides in the excavation for the access road and the upper left keyway. To keep the excavated surfaces stable and at grade, the contractor had to use controlled blasting techniques and the installation of many rockbolts. Asphaltic grouting was later performed to control seepage along relief joints. This adverse jointing and the presence of shears within the excavation for the underground powerplant caused movement of large blocks of rock within the powerplant walls. This prompted the contractor to install additional access/drainage tunnels and extensive systems of rockbolts, post-tensioned cables, and flat-jacks to support the rock mass and prevent further movement.

At the beginning of concrete placements in 1966, two longitudinal cracks were found in the top of blocks 9 and 11 at elevation 6777.5. Both cracks were in the center of the block, extended completely across the block, and had a maximum width of 0.03 inches. A mat of No. 11 reinforcement bars was placed over these cracks and concrete placements continued with no additional problems identified in this area.

In May of 1966, the center formed drain in block 10 was found to be plugged at elevation 6815 and had filled with sand and debris to about elevation 6897. The contractor requested permission to use high pressure water to loosen and remove the plug. Reclamation granted permission as long as the pressure in the formed drain did not exceed 100 lb/in^2. On May 5, 1967, the contractor applied the water pressure to the hole, but used pressures of almost 300 lb/in^2 and cracked the concrete in block 10 shortly after placements in this block reached elevation 7100. The crack formed in block 10 extended completely across the block and extended a short distance into block 9. The repair work included the following: all concrete was removed upstream of the crack, 24 rockbolts were installed within the dam below the crack to prevent downward propagation, 56 No. 11 dowel bars were installed to anchor the replacement concrete to the undamaged concrete, concrete was replaced using an epoxy bonding agent, and a mat of No. 11 bars was placed over the repair area to prevent any upward propagation of the crack. No problems have been identified at this area since the repairs were completed.


Nambe Falls, NM

Several other double-curvature arch dams were successfully designed and constructed by the Bureau of Reclamation in the late 1960's and 1970's. One that bears mention is Nambe Falls

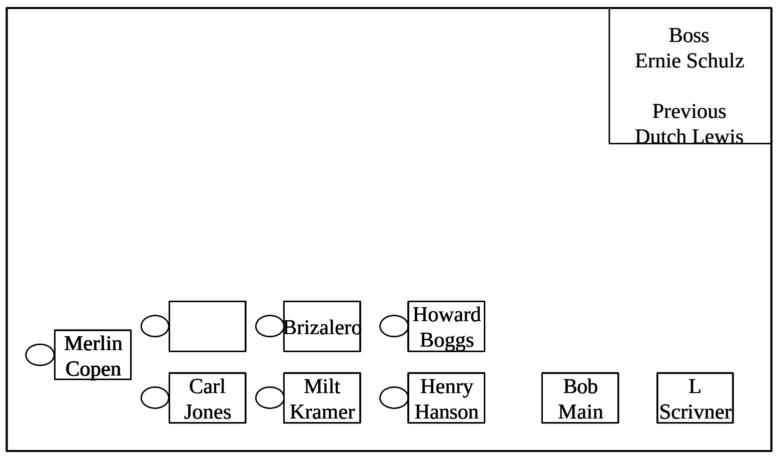
Dam, a 150 foot high dam on Rio Nambe in New Mexico. The arch is part of a composite structure with a massive concrete thrust block on the left abutment that ties into an embankment dam. The dam is quite thin, and temperature loadings were difficult to design for. Therefore, a series of flat jacks were installed in the crown cantilever, and the flat jacks were pressurized to prestress the dam into a state of compression that could handle all loading conditions adequately. Another item of interest is the development of elliptical arches by the use of "three-centered" geometry. The elliptical arches are approximated by a central section with a smaller radius, flanked by abutment sections with larger radii. This allows double-curvature arch dams to be designed for wider canyons. Although none of theses were built by Reclamation, the method was developed and several designs were completed.

XI. Structural Analysis Developments

A. Development and Computerization of the Trial Load Method

The trial load method of stress analysis assumes that the load applied to an arch dam would be divided between horizontal (arch) and vertical (cantilever) elements in such a way as to produce equal movements in all directions at points of intersection of these horizontal and vertical elements. Each arch and cantilever element is assumed to move independently of all others, but at the conclusion of the analysis, geometrical continuity exists at the intersections. Only a few representative arch and cantilever elements (5 to 10 each) need be analyzed. The basic concept is the internal loads equal the external loads at any intersection point. The internal loads are divided between the arch and cantilever elements until the deflections match. Thus the name, trial-load method of analysis. Then tangential and twist loads are applied in equal and opposite directions, one on the arch and one on the cantilever. This way the arch and cantilever deflections are brought into tangential and rotational agreement without changing the external load on the structure. These internal loads set up the three-dimensional interaction between two-dimensional arch and cantilever elements. To facilitate the process of dividing the internal loads between arches and cantilevers, certain patterns of loads called unit loads were developed. In applying the unit loads, it was advantageous to compute the movements of arches and cantilevers from unit loads before attempting to divide the external load between the arches and cantilevers. The total load resisted by the arches and cantilevers are determined by the trial-load adjustments. With these loads, stresses are then computed.

There are basically three levels of trial-load analysis depending on the desired accuracy and time duration for computations. 1) The crown cantilever analysis consisted of adjusting deflections of arch elements and the crown cantilever (the maximum vertical section in the center of the dam). The results were crude and neglected the effects of tangential shear and twist, but the computation time was relatively short and with judgement was an effective tool for preliminary designs. 2) The radial deflection analysis added 2 more cantilevers so radial deflection agreement was obtained at the crown and quarter points of the dam. The distribution of load along the arch was more accurate but the tangential shear and twist were still neglected, so the accuracy was only slightly better. The time for a radial deflection analysis was only slightly longer than the crown adjustment. 3) The complete trial-load analysis produced agreement of all three linear and all three angular displacements by properly dividing the radial, tangential, and twist loads between the arches and cantilevers. The accuracy was only limited by the number of arches and cantilevers used, the exactness of the basic assumptions (stress distribution), and the magnitude of error permitted in the slope (angular) and deflection adjustments. The results from a complete analysis were confirmed by the Hoover Dam model studies. The major limitation was the time required to perform an analysis.



Seating arrangement in the Analysis Section

In the 1960's before the application of computers to structural analysis, computations for the trial load analysis were done by a group of 6 to 8 engineers operating mechanical "adding" machines and filling in values on large tables. One analysis would take a pair of engineers from 6 to 8 weeks depending on the skill of the designer. As such, not many load combinations were analyzed. New rotation engineers performed these tedious computations. They would work in pairs so one could check the other's computations as they were performed. The seating arrangement in the Section was like a Viking ship with the row master behind the rowers. They worked with an experienced design engineer. It would take about 5 years to transition from a human calculator to a beginning designer. Arch dam designers would layout a preliminary shape for an arch dam. The loading conditions to analyze were decided upon and younger engineers would start the trial load computations. When the computations were complete, the results were returned to the designer and displacements were plotted. Adjustments to the loads between the cantilever and arches were determine, and the process repeated. Some designers, such as Howard Boggs, Milt Kramer, and Carl Jones, had a tremendous feel for how an arch dam reacted to loads and were very skillful in making adjustments. This took many years to develop. Howard Boggs wrote Engineering Monograph No. 36 explaining the beginning steps to layout an arch dam. However, this produced only a beginning shape. The real skill then came in trying to adjust the shape and produce the most optimum design. Layouts were done on a topography map with a large beam compass, french curve, and graph paper. Mechanical calculators ran 8 hours a day, 5 days a week, for weeks. There were replacement calculators on hand and Eddie Carlson was a full-time repair person from the Marchant company. The mechanical machines had 100 keys (10 rows of 10 keys) and the decimal point was set with a key.

With the application of computers to civil engineering problems in the 1960s, some engineers saw the potential of having the computer do the tedious manual calculations while other engineers viewed the computer as a threat to their jobs. Merlin Copen wrote:

> *"The major limitation to the use of the complete trial-load analysis is the time required to perform such studies, and the high degree of technical training necessary to efficiently conduct such an analysis. The time element has been effectively reduced by use of the electronic computer, and will be further diminished as the analysis is completely programmed. The number of highly-trained engineers required will also be greatly reduced."*

In 1957, Loyd Scrivner was the first engineer to write a computer program to compute geometric values. Reclamation initially rented time on an IBM 650 located in downtown Denver and eventually obtained one for themselves. Scrivner's initial programs were not written to be reused for other dams but had hard coded values inserted so a new program had to be written for each dam. Bob Main started with Reclamation in the summer of 1958 in the newly created Data Processing Section. Darrell Webber, who later became the Assistant Commissioner of the Engineering and Research Center, was a rotation engineer in that unit at the time. Because Bob could program on the IBM 650, he was hired into the analysis section. Bob wrote the general purpose geometry program for the lines of centers, introduced the idea of inputting values so the same program could be used for other dams, and introduced the concept of subroutines.

Loyd Scrivner wrote:

> *"In 1957, the Analysis Unit of the Concrete Dams Section (USBR) began the development of a series of electronic computer programs to reduce the time and cost required to complete a trial-load study. Programs have been developed utilizing the IBM 650 digital electronic data processing machine (system). . ."*
>
> *Most of the programming was done using a modified form of an interpretive routine (Bell Interpretive Language) which was developed to handle floating decimal arithmetic including the computation of the elementary transcendental functions. The electronic computer, to date, has been used primarily for the computation of forces and deflections in arch elements due to unit arch loads. This approach has been followed because of the following:*
>
> > *1. About 70 percent of the man-hours, and therefore the cost of performing a trial-load study, is expended making these computations.*
> > *2. These computations are repetitive in nature, which is a factor favoring advantageous use of electronic computers.*

Although we are not committed to any particular solution for the deflection adjustments, serious consideration will be given to an iteration process as opposed to a procedure based on the solution of a large group of simultaneous equations.

Merlin Copen wrote:

> *"The initial layout for an arch dam is based largely on the experience and judgement of the designer. ... As soon as a layout for a particular site has been completed, it is checked by means of a crown cantilever analysis to obtain an estimate of the stresses in the proposed dam. Currently the deflections of the arches and cantilevers produced by unit loads are computed by electronic digital computer. The time required, in a normal situation, to determine stresses with a crown adjustment, is approximately three days for two men. Several layouts may be necessary before a satisfactory stress condition is obtained. Then a radial deflection adjustment is made. This provides a more complete stress picture and might indicate the possibility of necessary or desirable changes. The radial deflection analysis requires approximately two days more than the crown cantilever analysis, or a total of approximately five days for two men.*
>
> *In practice, after a design has been analyzed and found to be acceptable with a radial deflection analysis, the effects of tangential shear and twist are estimated, based on the experience of the designer. ... Now the final test of the efficiency of the dam is made. While the detailed design work proceeds, a complete trial-load analysis is made of the structure. This will require approximately 100 to 150 man-days, depending on the size and complexity of the dam and the accuracy required from the analysis. It is anticipated that in the near future, further application of electronic computer processes will result in considerable reduction in the layout, such changes are made and incorporated in the detailed design procedures."*

There was plenty of arch dam work in the 1960s. Merlin Copen, George Wallace, and Dr. George Rouse went on a 10-week tour to Europe to see how they designed arch dams. As stated in their report:

> *"In recent years European engineers have made many important contributions to the design and construction of concrete dams. Through experimentation and studies European engineers have devised new techniques and have extended or improved existing practices. ... The team traveled in six countries and visited 15 organizations. ... Forty-three dams in various stages of completion were inspected together with 25 power stations. Thirteen laboratories were visited as well as six manufacturing plants and morel that 100 engineers were interviewed."*

It was this trip that lead to the development of double-curvature design methods at Reclamation. Yellowtail and Flaming Gorge were being designed and Morrow Point was on the horizon. Additional design staff probably would have been hired for this work. Additional design groups would probably have been created and promotion to heads of these groups would have been made. However, as Merlin Copen predicted, the large staffs were not required for this workload because of the advent of the efficient computer methods. Interviews for this paper revealed there may have been bitter feelings about lost advancements and lost promotion potential because of the computer. However, the computer did reduce the tedious part of structural analysis for arch dams. Some engineers that left the analysis section because of the tedious, boring, and repetitive computation work actually came back to the unit because of the joy and prestige of designing and working with arch dams.

There were disagreements on the best way to determine the response and design of arch dams. In 1960, Merlin Copen wrote:

> "*Since the end of World War II, interest in the design and construction of dams has received considerable impetus. This interest has resulted in novel approaches to problems of design. Currently the methods used appear to fall in one or more categories: (1) analysis of small scale models; (2) thin cylinder theory; (3) relaxation methods; (4) shell theory; and (5) trial-load analysis. Each of these has advantages and disadvantages. The choice of methods generally resolves into accuracy and reliability desired as opposed to time, finances, and experience available for design procedures.*
>
> *After exhaustive study of the various possibilities, the United States Bureau of Reclamation, Dept. of Interior (USBR) adopted the trial-load method of analysis for designing and analyzing arch dams. Whereas there have been notable advances in the use of other methods, the USBR has still found the use of trial-load to be completely satisfactory and unexcelled in this field. Recent developments in the use of electronic digital computers, and the effective application of simplified analyses have made this method even more effective.*"

The steps to develop a computerized trial-load method was to first program the geometry, then the arch computations, next the cantilevers, and then combine this into a crown adjustment (several arches and one cantilever). The computer being used could only handle 42 equations. The final step was a complete analysis. This was a very challenging task with limited computer capabilities. After the IBM 650, Reclamation obtained time on a Honeywell machine in Minneapolis. Cards would be sent in on Friday and results would be back on Wednesday. Reclamation obtained their own Honeywell 800. The programming language was Automath, Honeywell's version of Fortran. Mr. Harry Beck, Assistant Division Chief of the Data processing group, taught the new rotation engineers this version of Fortran. The dams section hired Dale Morsette as a GS-12 because he had a Masters Degree. This caused some bad feelings in the Section because most individuals were GS-11's and the requirement to be a GS-12 was the ability to do a complete analysis unassisted. Dale worked on the initial phases of computerizing a complete analysis from 1963 to 1967. This was a very frustrating task for Dale. In 1967, H. Walter Anderson realized the Honeywell did not have the capability needed for arch dam analyses, so he arranged time on a Control Data Corporation (CDC) 1601 at the Environmental Science

Services Administration (ESSA), currently the National Bureau of Standards, in Boulder, Colorado. Reclamation had a daily shuttle that would take cards up to Boulder at 3:00 and return the next day at 10:00. Reclamation started moving into Building 67 on April 13. Dale left in early 1967, so Glenn Tarbox was assigned the programming task since he knew how to do a complete analysis. Bob Main, a computer programmer, started assisting in June of 1967 and a working version was accomplished in September 1967. The programming was divided into 4 phases: 1) data reorganization, 2) equations, 3) solution, and 4) stresses.

These computer programming efforts and advancements for the trial load method evolved into what is called today the Arch Dam Stress Analysis System (ADSAS). ADSAS was a computerized version of a flexibility method of analysis referred to as “trial load”. However, equations were developed and written to compute deflections at any location along the cantilevers and arches. The equations for deflections could be solved directly without using trial-loads. This essentially is a precursor to the finite element method. The computers still did not have enough storage space to hold all the matrices at one time. So ADSAS used an iteration method to solve the simultaneous equations. The solution technique used in ADSAS is unique and innovative and based on approaches developed for the hand calculations.

ADSAS really advanced the state-of-the-art in arch dam analysis, sped up the design process, and helped justify the engineering mainframe computers. ADSAS changed the way the concrete dam group operated because more load combination and geometrical shapes could be investigated in minutes rather than weeks. Output from ADSAS was still in paper form and was about one inch thick. Designers would quickly thumb through the large volume of paper output, propose changes to the dam geometry, and have the younger engineers run ADSAS and bring back the paper output.

Despite the advances that came with ADSAS, it was still not appropriate for dynamic analysis. In addition, the ADSAS program and users manual were developed for internal use, there was machine dependent computer code specifically for a Cyber 70-74/28, and the program was in excess of 39,000 cards long with over 240 subroutines. This caused problems for others to convert the program to their computers and use the program.

B. <u>Linear Structural Analysis</u>

In 1974, the Structural Analysis Program (SAPIV) was written by Klaus Bathe and Ed Wilson at the University of California at Berkeley. Glenn Tarbox and Karl Dreher were instrumental in getting SAPIV operational on the CDC mainframe computer at Reclamation, debugging the program, and developing the finite element capability for arch dams. Many sensitivity runs were made comparing the trial load method (ADSAS) with the finite element method (SAPIV) during the design of Auburn Dam. Full dynamic time-history, linear elastic, 3-dimensional, modal superposition analysis were performed. Auburn Dam was the first “test” case. Since that time, almost every arch dam in Reclamation's inventory has been analyzed using SAPIV for earthquake loading. SAPIV also has the ability to handle static loading including reservoir, temperature, and stage construction, making it a powerful tool for dam analysis. Many engineers in the analysis group wrote pre- and post-processing programs to work with SAPIV, which sped-up and advanced the finite element analysis of arch dams.

Evaluating the results of dynamic finite element analyses required advances in estimating concrete strengths for comparison to the calculated stresses. It was postulated that concrete would be stronger in both tension and compression under the rapid loading associated with earthquake events. Rapid loading laboratory tests were developed which confirmed this is the case. An increase in tensile strength of approximately 50 percent can be expected under dynamic loading.

Reclamation funded the University of California at Berkeley to develop a computer finite element program specifically for arch dams: the Arch Dam Analysis Program (ADAP). The development was suppose to occur over three years, but funding got tight after the first year. As such, only a partial program was developed. Dr. John R. Mays, from the University of Colorado at Denver was hired part-time to debug the program and get it operational. Over the years, the University of California at Berkeley, continued to develop ADAP. The Enhanced Arch Dam Analysis Program (EADAP) contained hydrodynamic interaction and ADAP-88 was a nonlinear version that implemented contraction joints in the form of contact surfaces. This program has not been used much at Reclamation, but has found some use on the outside.

The University of California at Berkeley also developed a series of computer programs specifically for arch dams: Earthquake Analysis of Concrete Dams (EACD). The current version implements hydrodynamic interaction with incompressible or compressible fluid elements and dam to foundation interaction incorporating the damping effects of the foundation. Engineers in Reclamation have developed pre- and post-processing programs to aid in the use of this program. It has been used for the earthquake analysis of several Reclamation concrete dams.

In 1978, Reclamation obtained the first general purpose nonlinear finite element program from Klaus Bathe from the Massachusetts Institute of Technology (MIT): Automatic Dynamic Incremental Nonlinear Analysis (ADINA). The program mainly implemented the material nonlinearity of concrete. Dr. John R. Mays developed a nonlinear joint element within ADINA. Howard Boggs and Dr. Mays were some of the first engineers to analyze an arch dam with nonlinear contraction joints. ADINA was used sparingly for specialty problems at Reclamation until 1996 when Reclamation made the transition to ABAQUS. In 1984, the structural analysis group purchased a Hewlett-Packard UNIX workstation for pre- and post-processing finite element data using PATRAN. The finite element analyses were still run on the Cyber mainframe computer.

In August 1993, the mainframe Cyber computer was being decommissioned and the structural analysis group made the transition to a larger Hewlett-Packard UNIX workstation (HP-755). In 1997, an HP J2240 was obtained that had 2 CPUs, 2 Gigabytes of internal memory and 90 Gigabytes of hard disk storage. This was more powerful than the early computers at Reclamation just 35 years previous. Structures modeled with 38,000 nodes, 100,000 degrees-of-freedom, contraction joint contact surfaces, and nonlinear concrete material properties are now being analyzed for earthquake loads.

C. Nonlinear Structural Analysis

Linear finite element analysis has long been accepted as a way to analyze structures. There are limitations, however, when performing a linear analysis. Stresses calculated in a linear analysis can exceed the allowable strengths of materials. In these cases the actual behavior of the structure after the material strengths were exceeded could be significantly different than that predicted by the linear analysis. Also, response of geometric non-linearities (contraction joints or compression only members) cannot be modeled using linear analysis. In the past, attempts have been made to model these conditions by modifying the modulus of elasticity in a particular direction and by using a combination of members to simulate the expected behavior of a connection with limited success. Analysis tools have now progressed to the point where good non-linear capabilities are available. Non-linear analysis is the next step in addressing these limitations.

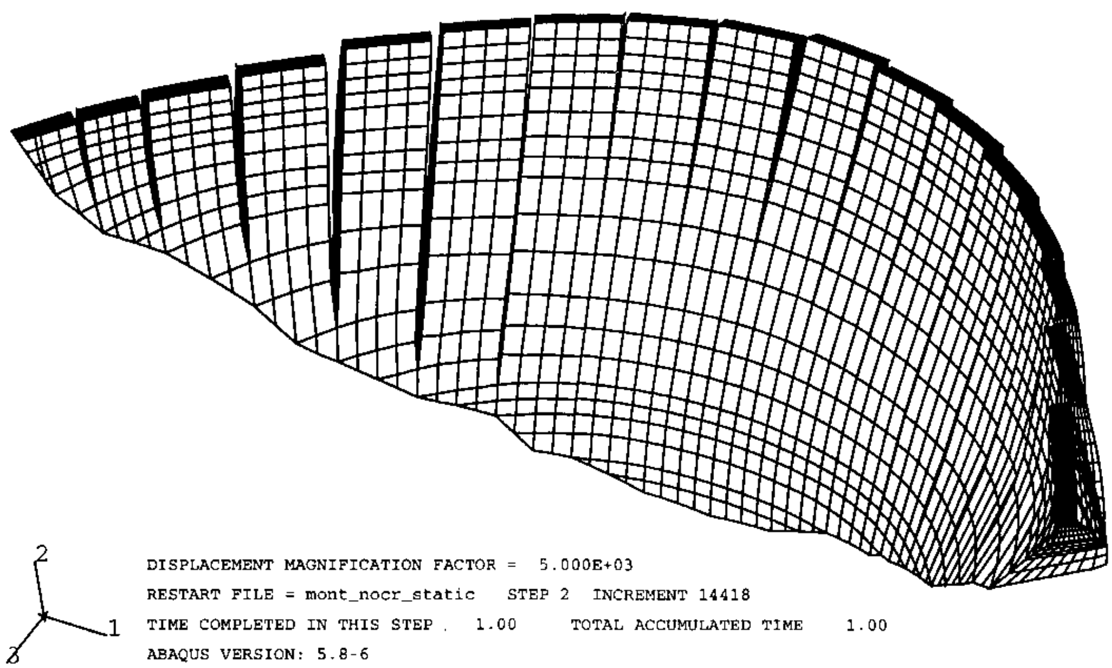
Engineers at Reclamation are very familiar with linear finite element analysis. In the past several years, work has been done using non-linear capabilities as well. Two non-linear analysis methods have been used using ABAQUS finite element code. The first method employs the standard stiffness formulation (F=Kx). The second method solves an explicit formulation with Newton's 2^{nd} law, F=Ma. Each method has advantages and disadvantages and therefore, it is important to choose the appropriate formulation for a given problem. The following examples illustrate the use of nonlinear analysis for dynamic and static loading conditions.

1. Nonlinear structural analysis of Monticello Dam

Monticello Dam is a 304-foot-high constant-center concrete arch dam, with fillets at the abutments, located on the Putah Creek, 30 miles west of Sacramento, California. The dam was constructed from 1953 to 1957, has a crest length of 1,023 feet, a crest thickness of 12 feet, and a maximum base thickness of 100 feet. The earthquake response of the structure, incorporating the vertical contraction joints and weak horizontal lift lines, was analyzed non-linearly using the ABAQUS/Explicit computer code.

Monticello Dam, CA

In this analysis, eight elements through the thickness were chosen to better model the contact surface interactions. The 8-noded linear brick element and the 6-noded linear wedge element were chosen for the 3-D model. The 8-noded element is a reduced integration element. The foundation rock was modeled to a distance of two times the dam height to properly model earthquake energy around the dam itself. It was modeled with the same type of elements that were used to model the dam. For this analysis Rayleigh damping values of a = 3.0 and ß = 0.0 were used. This is comparable to the 5 percent of critical viscous damping used traditionally in dam analysis.



Finite element model of Monticello Dam, CA (foundation mesh not shown)

As expected, the tensile arch stresses are less with the model that incorporates the contraction joints in comparison to a linear elastic analysis. Cantilever compression stresses increase in the center portion of the dam on the downstream face, and tensile cantilever stresses decrease slightly in the bottom center of the dam on the upstream side. The existence of tensile cantilever stresses on the upstream face with the contraction joint model indicates that the cantilevers are taking load. This is because when the winter temperature load is applied, the cantilevers contract and create openings in the joints. The hydrostatic loads tend to close these openings, but can not fully because of resistance offered from the cantilevers in bending (initially no cracking of concrete or horizontal weak lift lines was incorporated in this model to relieve the stress). Thus, a large tensile cantilever stress continues to exist on the upstream side toward the bottom center of the dam. Gravity load was applied first. Although gravity was applied to the entire structure at once, the contact surfaces used to model the vertical contraction joints prevented the structure “hanging” from the abutments as would be the case if gravity was applied without contact surfaces. The gravity load caused the cantilevers to displace upstream, thereby, allowing the weight of each cantilever to act independently. Next the reservoir load was applied. This caused the cantilevers to move downstream and the contraction joints to close. The temperature load was applied as temperature differentials at all the nodes in the dam. Hydrodynamic interaction was incorporated by adding mass to the upstream nodes of the finite element model based on an incompressible fluid element formulation.

Three earthquake records were applied to the contraction joint model. Crest displacements, crest velocities, contraction joint opening and closing and arch and cantilever stress histories were obtained for each record. Crest displacements at the centerline of the dam reach peak values of about 7 inches. Permanent offsets at joints were less than 1 inch. Maximum crest velocities at the centerline of the dam are on the order of 40 in/sec in the cross canyon direction, 14 in/sec in the vertical direction and 100 in/sec in the upstream /downstream direction. Contraction joints at the centerline of the dam open to a maximum value of about 0.4 inches. Tensile arch stresses reduced significantly in comparison to linear elastic analyses

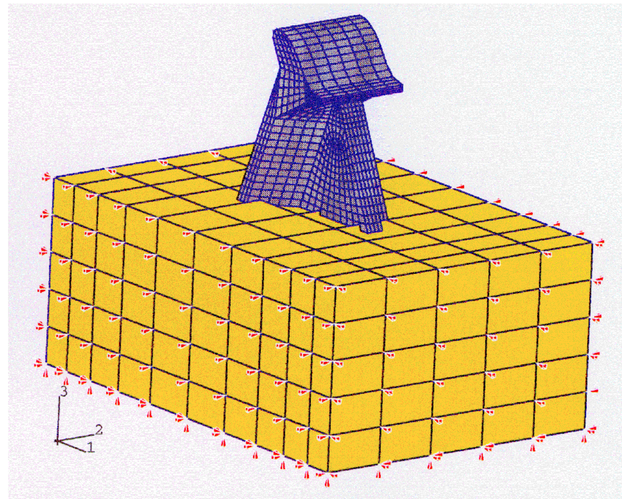
Large tensile cantilever stresses continue to exist in the dam during static and dynamic loadings with the contraction joint model. These stresses will be relieved by horizontal crack formation in the dam. Since the lift lines of the cantilevers are weak in comparison to the parent concrete (based on laboratory test of drill core), these cracks will occur at the lift line locations. There are two ways to model these lift lines. The first method is to set the cracking stress to a low value in the nonlinear concrete material property statement. This would allow the concrete to crack and relieve any cantilever stress that would exceed the cracking stress value specified. However, it isn't possible to specify that the lift lines are weaker than the parent material within the concrete cracking model. The second method, which was ultimately used, is to insert a series of horizontal contact surfaces, spaced so as to model the effect of the weak lift lines. This approach further lends itself to a kinematic study; i.e., a series of blocks stacked on top of each other held in place by the arch action of the dam. The analyses indicated the dam would be stable even with cracked lift lines. Although 6-inch-deep shear keys exist at each contraction joint of the dam, these keys were not included in the finite element model because of the need to keep the contact surfaces simple in order to obtain a stable solution. The effects of neglecting the keys, and better methods for modeling contraction joints, are the subject of ongoing research.

2. Nonlinear Structural Analysis of Pueblo Dam

Pueblo Dam is located near Pueblo, Colorado. Pueblo Dam is a composite concrete and earthfill structure approximately 10,230 feet in length. The concrete portion consists of a massive head buttress dam including a 550-foot overflow spillway sectionlocated near the central part of the concrete dam. The dam was designed and constructed by Reclamation, and completed in 1975.


Pueblo Dam, CO

The purpose of this nonlinear study was to reevaluate the sliding stability at potentially disbonded lift lines and the vertical stress level at the dam heel using a three dimensional finite element model incorporating horizontal contact surfaces. Previous linear-elastic finite element analyses completed at Reclamation resulted in acceptable factors of safety against sliding (with some cohesion) but they also indicated that tensions would develop at the dam heel under some static load cases. Since the linear elastic analyses completed previously used a continuous mesh, the potential nonlinear characteristics existing along the dam-foundation contact surface were not captured; therefore, it was necessary to complete a nonlinear finite element analysis incorporating a horizontal contact surface in order to capture the effects of stress redistribution upon opening of the contact at the dam heel, representing crack propagation along the contact if weak lift lines are actually present.



Finite element model of an overflow buttress at Pueblo Dam, CO

A single overflow buttress of Pueblo Dam was modeled using ABAQUS / STANDARD. The model used three-dimensional 8-noded fully integrated brick elements throughout the dam and foundation. The foundation was modeled as a large rectangular block of solid sandstone, approximately 350 feet long, 250 feet wide and 150 feet in depth. The upper surface of the foundation block, at elevation 4755 feet, was used to define the lower half of the non-linear contact surface in these analyses. The dam model was positioned in the center of the foundation block with the bottom surface of the dam model forming the upper half of the non-linear contact surface. The edges of the foundation were fixed, but there were no translation or rotation boundary conditions applied at nodes in the dam model. Although a tension limit could be input, once cracked the only force preventing rigid body motion of the dam was the frictional force developed on the contact surface; therefore, additional iterations were required to obtain convergence of the first increment of the gravity loading to establish normal forces on the contact surface.

The ABAQUS STANDARD finite element program uses time varying load application for all of the static loads. The gravity load was applied gradually from zero to one second of analysis time, followed by application of the reservoir and uplift pressure loads. The uplift pressures were also automatically recalculated at each analysis time increment as both a function of the current reservoir depth and the crack (open contact surface) length. The non-linear analyses indicated that the dam was stable for these static loading conditions. The tensile stresses which developed at the dam heel in the previous linear analysis were relieved upon opening of the contact surface when zero tensile strength was assumed on the contact surface, but a significant portion of the dam remained in compression, and was capable of carrying the load.

XII. Roller-Compacted Concrete - Rapid Construction for Gravity Dams

Despite advances in automated mixing, handling, and placement of mass concrete, the procedures were still somewhat labor intensive and time consuming in comparison to earthfill production rates. In the late 1970's and early 1980's some relatively small projects were completed using the concept of roller-compacted concrete (RCC). The concept involved placement of a lean and dry concrete mix by spreading it in thin layers with a bulldozer, and compacting it with vibratory drum rollers. The lean mix reduced the heat generated, and rapid production rates could be achieved, as the placement was mechanized and there was no need to wait for curing before placing the next lift. The Bureau of Reclamation began testing a high paste (cement plus flyash) RCC concept in 1980. This resulted in a strong and stiff material with similar properties to conventional concrete. Thus, the design of gravity dams using this type of material could be based on conventional gravity dam design methods.


Upper Stillwater Dam, UT

In 1985 RCC placements began at Upper Stillwater Dam, the Bureau of Reclamation's first RCC dam and at that time the world's largest. The straight gravity dam is about 280 feet high, and nearly 2700 feet long, and contains more than 1,600,000 yd^3 of concrete (most of which is RCC). Although the downstream slope is 0.6:1, the point of intersection of the downstream and upstream slopes is above the dam crest, which results in an equivalent downstream slope of about 0.7:1 for the height of Upper Stillwater Dam when compared to other typical gravity dams. The upper part of the downstream slope was steepened to allow sufficient crest width for the construction equipment. This increases the mass and stiffness of the dam when compared to traditional gravity sections.

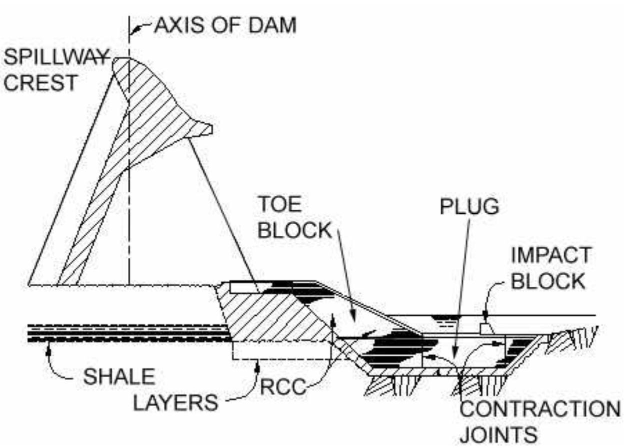

Compacting RCC at Upper Stillwater Dam, UT

Typical excavation and treatment of the quartzitic sandstone and argillite foundation rock were performed. Crushed aggregate and sand were manufactured for the RCC. A richer RCC mix was
used near the upstream face. The RCC contained between 135 and 160 pounds of cement per cubic yard, and between 290 and 350 pounds of flyash per cubic yard. Temperature control was achieved by placing the RCC below 50 degrees Fahrenheit and by replacing cement with flyash to limit the heat rise. The RCC was tied to the abutments and to the foundation by use of conventional concrete. At the base of the dam, conventional concrete was first placed to form a level surface to start RCC placements. At the abutments, conventional concrete was placed between the RCC and the rock. Laser-guided slip-form machines were used to place concrete elements forming the upstream and downstream faces of the dam. This proved to be a fairly rapid means of forming the dam, and eliminated the relatively time consuming and

labor intensive process of erecting and stripping conventional forms. RCC was delivered to the dam from the batch plant using a conveyor belt. There it was loaded into trucks, transported to the placement, and spread with a small bulldozer using a laser controlled blade. A vibratory drum roller then compacted the material into a dense mass. In 1986, over 715,000 yd^3 of RCC was placed in less than five months. The peak shift placed over 5400 yd^3. Joint cleanup was required, depending on the age of the concrete, and joints greater than 72 hours old were required to be sandblasted or waterblasted. Very good bond was achieved. In fact, it was difficult to find the lift lines in the core taken from the dam.

The major drawback to the design and construction of Upper Stillwater Dam was the exclusion of contraction joints or other means to control the cracking and subsequent leakage through the dam. Thermal and structural analyses had indicated that cracking would be limited to the face of the dam, and would not extend through the dam thickness. However, this proved to be incorrect, and regularly spaced vertical cracks propagated through the dam normal to the axis. Leakage from some of these cracks became significant, and the grouting and drainage gallery constructed about 20 feet from the upstream face of the dam received large inflows. The leakage at two of the cracks was exacerbated by small sliding movements on an argillite layer within the foundation that stopped when the passive rock mass downstream of the dam was mobilized. This tended to open the cracks on either end of where the movement occurred. All open cracks were grouted twice. The upper portions of the cracks were grouted with hydrophillic polyurethane grout, and the lower portions were grouted with cement grout. This proved to be effective for several years. However, seasonal movements of the cracks due to variations in reservoir level and temperature eventually reopened the cracks, resulting in renewed leakage. Plans are being developed to seal the cracks with an upstream membrane or a secant wall drilled across the cracks upstream of the gallery.

The contraction joint issue in RCC dams is critical. For gravity dams it is adequate to control the cracking by forming joints or placing crack inducers to control the crack locations. Water stop features can then be designed to reduce flow through the cracks. If RCC is to be used for arch dams, it will be necessary to develop a way to grout the joints to lock in arch action at the desired temperature. The Bureau of Reclamation developed such a system for the foundation modifications at Pueblo Dam in the late 1990's.



Section Through Buttress 8 or 9 at Pueblo Dam, CO Showing RCC Stabilization Measures.

By way of background on this project, nearly horizontal shale layers beneath the massive head buttresses of the dam daylighted in the spillway

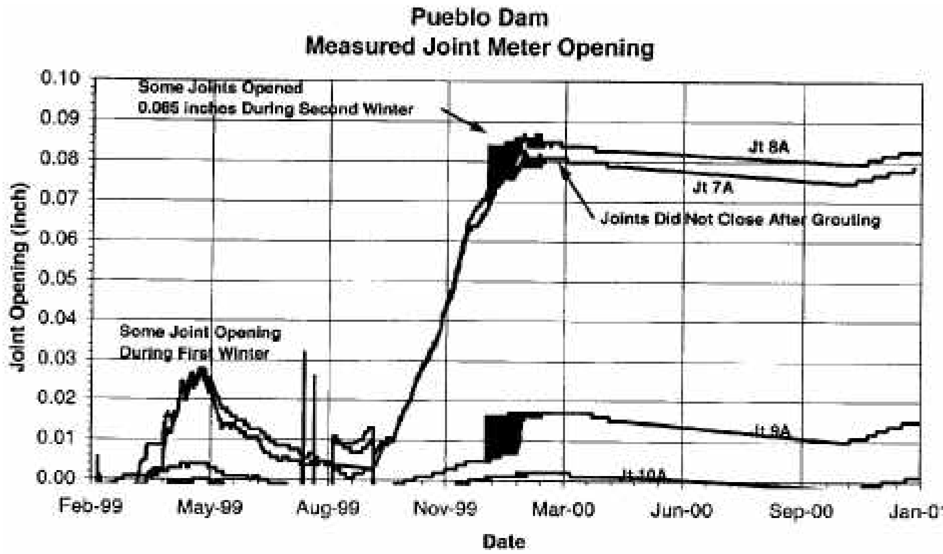

RCC Placement, Spreading, and Compaction Operations in the Spillway Plunge Pool - Pueblo Dam, CO

stilling basin excavated at the toe of the dam, downstream of some of the buttresses. Due to the large population downstream of this dam, potential sliding of the structure on these shale layers posed a high risk, and was a dam safety concern. A RCC plug and toeblock, anchored with double-corrosion-protected high strength rock bolts, were constructed in the stilling basin to
block the daylighting planes and buttress the foundation. State-of-the-art distinct element analyses, and probabilistic stability analyses were performed to ensure the RCC geometry would be effective in stabilizing the dam. The RCC material was somewhat different than that used at Upper Stillwater Dam. Rounded river aggregates up to 1 ½ inch maximum size were used. Approximately 120 pounds of cement and 180 pounds of flyash were used per cubic yard of RCC. Surface cleanup and bonding mortar were used on all lift surfaces of the toe block and on lift surfaces more than 12 hours old in the plug (below elevation 4728). Although the design strengths were met, a somewhat porous zone developed a few inches below the lift surface, particularly for lifts that were a day old when the next layer of RCC was placed. It was thought that the rounded aggregate made the RCC more susceptible to damage from construction traffic on lifts that were in a fragile condition just after setting of the RCC. Windy conditions at the site may have also prematurely dried the surface of the RCC lifts during and shortly following placement.

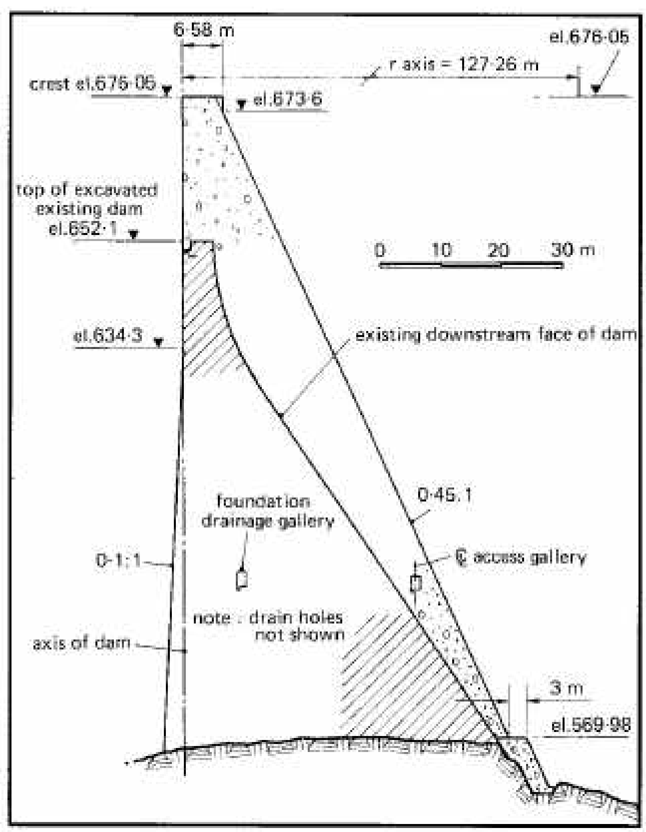
Installing Joint Inducing Plates in RCC at Pueblo Dam, CO

Contraction joints were formed in the RCC by vibrating steel plates into the freshly compacted lifts. The joints trending in the cross-canyon direction needed to be grouted to ensure that load could be transferred across the joints with minimal displacement. The plate locations were carefully surveyed prior to installation so that the joints could be intercepted by vertical grout holes. Six-inch-diameter holes were drilled at 10-foot spacing in the upstream-downstream direction and 5-foot spacing in the cross-canyon direction. Steel plates were not placed in the drill hole locations. Some holes were filled with polyurethane grout to isolate grout zones. Tubing was designed and installed in the holes to provide grout supply and return lines, and venting to remove air and water from the system.

Grouting was performed the second winter following RCC placement when joint meters indicated sufficient joint opening for grouting. The grouting was successful, and the joints did not close the following summer, indicating good filling of the joints



Opening of Transverse Contraction Joint in RCC at Pueblo Dam, CO. Grouting of Joints Occurred in February 2000.

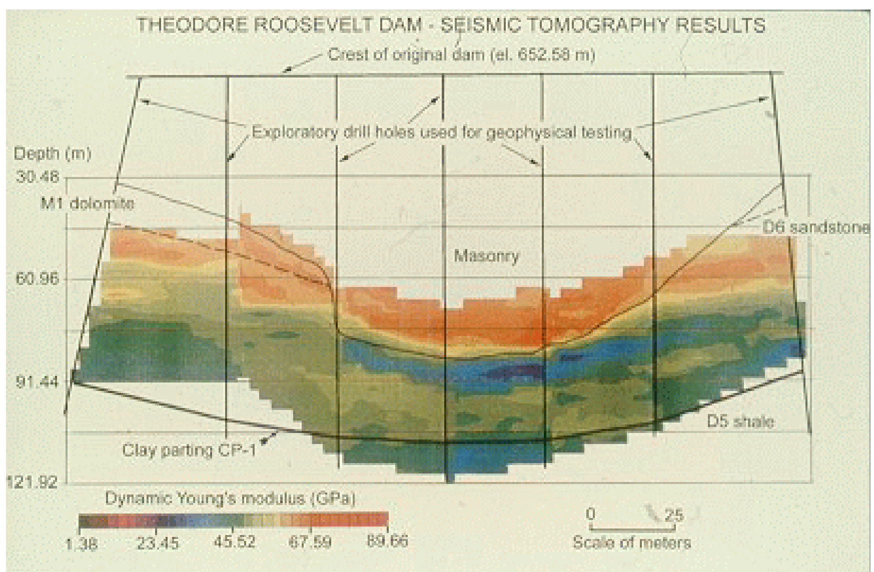


Schematic of Raising Theodore Roosevelt Dam, AZ

XIII. Transition to Dam Safety - Applying Technology to Reduce Risk

The Bureau of Reclamation has been actively involved with a formal safety of dams program since April 1977, when an Executive Order was issued initiating the Federal guidelines for dam safety. The aim of Reclamation's dam safety program is to ensure that the agency's dams do not pose an unacceptable risk to the downstream public. To that end, Reclamation has pioneered the use of risk analysis in assessing dam safety. Once it is determined that structural modifications are needed to reduce risk, Reclamation has used the design and construction technology developed over the past century to fix existing dams. For concrete dams, this means application of detailed analyses, design procedures, and modern concrete technology. Two cases, Theodore Roosevelt and Pueblo Dam Modifications, illustrate this point. The case of Pueblo Dam was discussed in the previous section on roller-compacted concrete (RCC). Additional details of the Theodore Roosevelt Dam Modifications are provided here.

Potential deficiencies with regard to the potential to pass large floods, potential instability during large earthquakes, inadequate release capacity, and the need for more water storage resulted in major modifications to Theodore Roosevelt Dam between 1988 and 1995. Part of those modifications resulted in raising the arch dam 77 feet. It was necessary to determine whether the dam and foundation could withstand this increase in head. Combinations of joints and bedding planes (dipping upstream at about 20 to 25 degrees) in the Precambrian sedimentary foundation rock formed potentially unstable blocks. Initial stability analyses indicated that the foundation would not meet the desired factors of safety under the increased loading. Therefore, foundation drainage was installed from adits excavated in the rock and a gallery excavated through the existing masonry. Piezometers were installed to measure foundation water pressures before and after construction of the drainage, and pressure contour maps were developed for determining uplift forces in the foundation analysis. The drainage was very effective, reducing pressure heads by about 43 to 68 feet. In situ uniaxial jacking tests were performed in the drainage adits, and correlated with seismic tomography testing to estimate the deformation properties of the foundation rock mass and concrete masonry of the existing dam. These properties were included in finite element structural analyses to study the behavior of the dam and more closely determine loads acting on the foundation. Final foundation analyses indicated that the raised dam with the drainage in place met the desired safety factors, and was more stable than the existing dam without drainage.



Results of Seismic Tomography Testing at Theodore Roosevelt Dam, AZ

Constructing an overlay of conventional concrete on the existing dolomite masonry dam posed some additional challenges. A concrete test panel was constructed on the downstream face of the dam to determine the likely bond strength between the new concrete overlay and the masonry. Core samples were extracted and the interface was tested in tension and direct shear. This information was used in extensive computer modeling to verify the design and shape of the overlay. The dam was analyzed for static and dynamic loading using finite element methods. The existing masonry was modeled in three horizontal stages to simulate the layered construction. The mass concrete overlay was modeled as it was constructed, in blocks separated by (keyed) contraction joints. Recommended block dimensions, lift heights, concrete placement temperatures, and cooling requirements were based on temperature control studies. These studies took into account the thermal properties of the concrete mix design, and the expected temperature rise within the mass concrete during construction. The concrete was cooled using cooling coils embedded in the 10-foot lifts, and the contraction joints in the overlay were grouted to provide arch action and improve the stress distribution within the structure. The numerical modeling simulated this construction sequence. Final analysis of the composite structure indicated improved stress conditions within the existing masonry portion of the dam, and results meeting Bureau of Reclamation stability and stress criteria. Seismic response analyses indicated the structure should perform well under large seismic loadings. Construction of the overlay followed typical mass concrete placement techniques, developed and refined since the construction of Hoover Dam. A high line was used to transport concrete to the placement in buckets. The concrete was placed in layers and vibrated into place. Something not done before included placement of geo-composite strip drains between the existing masonry and the new concrete to provide drainage at the interface.

Other modifications to the dam included construction of a lake tap and tunnel system to provide a new outlet works and power penstock. New mass concrete thrust blocks were constructed on each abutment to fill the gap formed by the original spillway cuts. New spillways were constructed through each thrust block. Hydraulic model studies were used in the hydraulic design of the spillways. Spillway flows enter a diverging chute and flip structure before plunging to an excavated basin in the river channel below. The spillway alignments cause the discharge jets to impinge at or above tailwater level, while both spillways are operating under higher reservoir heads.

Concrete Placement during Modifications to Theodore Roosevelt Dam. Note placement and vibration of concrete in layers (left), placement in blocks against the masonry (top), and new thrust block and spillway (bottom).

Modified Theodore Roosevelt Dam, AZ - Completed and rededicated in 1996.

XIV. Conclusions

We hope you have enjoyed this tour of the evolution of concrete dam design, analysis, and construction within the Bureau of Reclamation over the past century. There is no question that the early pioneers in this effort were extremely talented and set the stage for some of the great feats of human engineering that were to follow. Monumental projects like Hoover and Grand Coulee Dams are still "wonders" today. During the heyday of dam construction in the United States, the Bureau of Reclamation developed a reputation as a world leader in concrete dam technology. The construction of dams in the United States is winding down now after a century of extensive development. The last new concrete dam constructed by the Bureau of Reclamation was completed over a decade ago (1989). The legacy and expertise in concrete technology and dam construction at the Bureau of Reclamation remains a valuable national resource, and has been recently used to efficiently fix dams where safety concerns exist. So what does the next 100 years hold? As long as dam safety projects remain to be done, the expertise will be maintained and developments will slowly occur. However, without large projects, it is likely that the leadership in this area will gradually shift to developing countries in the future. The Bureau of Reclamation can be proud of the giant springboard they have provided from which these efforts can be launched.

BIBLIOGRAPHY

North Platte Project History, Volume 1, 1902-1913, U.S. Reclamation Service.

Wisner, G.Y., and E.T. Wheeler, "Investigation of Stresses in High Masonry Dams of Short Spans," Engineering News, Vol. LIV, No. 6, August 10, 1905.

Elliot, B.R., "Roosevelt Dam, Salt River Project, Arizona," Dams and Control Works, U.S. Department of the Interior, Bureau of Reclamation, Government Printing Office, Washington D.C., Third Edition, 1954.

Smith, C.W., "The Construction of the Roosevelt Dam, An Account of the Difficulties Encountered in Constructing a High Masonry Dam in Arizona," Engineering Record, Vol. 62, No. 27, December 31, 1910.

Feature History of the Shoshone Dam, U.S. Reclamation Service, June 1, 1910.

Eastwood, J.S., "An Arch Dam Design for the Site of the Shoshone Dam," Engineering News, Vol. 63, No. 23, June 9, 1910.

Cole, D.W., "The Shoshone Dam of the United States Reclamation Service, A Description of the Method of Constructing the Highest Dam in the World," Engineering Record, Vol. 62, No. 4, July 23, 1910.

Dollar, D., D. Hinchliff, L. Pimley, and P. Ruchti, "Storage Capacity Increased at Buffalo Bill Dam," International Water Power and Dam Construction, Volume 43, Number 7, July 1991.

Hopson, E.G., "Methods and Cost of Constructing East Park Dam, California, Orland Project, U.S. Reclamation Service," Engineering and Contracting, Vol. XXXVI, No. 16, October 18, 1911.

Hoffman, C.J., "Arrowrock Dam, Boise Project, Idaho," Dams and Control Works, U.S. Department of the Interior, Bureau of Reclamation, Government Printing Office, Washington D.C., Third Edition, 1954.

Coghlan, R.R., Report of Laboratories Operated in Connection with the Manufacture and Use of Sand-Cement at Elephant Butte, N.M.," U.S. Reclamation Service, Denver, Colorado, May 1, 1916.

Abrams, D.A., "Proportioning Concrete Mixtures," Proceedings of the 18th Annual Convention of the American Concrete Institute, Vol. 18, Detroit, MI, February 1922.

Darr, A.L., Construction of Gerber Dam, Bureau of Reclamation, February 25, 1925.

Simonds, A. W. “Movement of Owyhee Dam Due To Grouting Contraction Joints,” Bureau of Reclamation, June 14, 1934.

“Contraction Joint Opening - Owyhee Dam, Hoover Dam Concrete Research,” Bureau of Reclamation, October 15, 1932.

“Morning-Glory Shaft Spillways - A Symposium,” Paper No. 2802, Transactions Vol. 121, page 311, American Society of Civil Engineers, 1956.

Lowe, F.C., Assistant Engineer, “Hydraulic Model Studies for the Glory-Hole Spillways at Owyhee Dam - Owyhee Project, Oregon-Idaho and Gibson Dam - Sun River Project, Montana,” Hydraulic Laboratory Report No. 159, Bureau of Reclamation, Denver, Colorado, November 15, 1944.

“Design of the Owyhee Irrigation Dam,” Engineering News Record, April 26, 1928.

“Another Huge Dam,” Western Construction News, Volume III, May 10, 1928.

“Final Record on Design and Construction of Owyhee Dam, Owyhee Project - Oregon - Idaho”, Written in Denver Office, June 24, 1941. (Original available in the National Archives, Denver, Colorado)

Warnock, J. E., “Report on Inspection Trip to Correlate Present Hydraulic Design Practice and the Operations of Structures in the Field,” Denver, Colorado, Bureau of Reclamation, August 8, 1938.

Boulder Canyon Project Final Reports, Bulletin 1, Part VI - Hydraulic Investigation - Model Studies of Spillways, United States Department of the Interior, Bureau of Reclamation, 1938.

Boulder Canyon Project Final Reports, Bulletin 3, Part IV - Design and Construction - Diversion, Outlet, and Spillway Structures, United States Department of the Interior, Bureau of Reclamation, 1947.

Boulder Canyon Project Final Reports, Bulletin 3, Part VI - Hydraulic Investigation - Studies of Crests for Overfall Dams, United States Department of the Interior, Bureau of Reclamation, 1948.

Bazin, M., Recent Experiments on the Flow of Water Over Weirs, Annuls des Point et Chaussées, October 1888 (translated by Arthur Marichal and John C. Trautwine, Jr., and published in the Proceedings of the Engineers' Club of Philadelphia, Vol. VII, No. 5, 1890, p. 259 and Vol. IX, No. 3, 1892, p. 231).

Brandley, J. N., Engineering Monograph No. 9, Discharge Coefficients for Irregular Overfall Spillways, United States Department of the Interior, Bureau of Reclamation, March 1952.

Design of Small Dams, Third Edition, United States Department of the Interior, Bureau of Reclamation, 1987.

Engineering Monograph No. 25, Hydraulic Design of Stilling Basins and Energy Dissipators, By. A. J. Peterka, United States Department of the Interior, Bureau of Reclamation, Eighth Printing, May 1984.

Falvey, Henry T., Engineering Monograph No. 42, Cavitation in Chutes and Spillways, United States Department of the Interior, Bureau of Reclamation, Eighth Printing, April 1990.

Effect of the Admixture, T.D.A., on the Durability of Concrete, Columbia Basin Project, Laboratory Report No. Ce-33, Denver, CO, June 1942.

Concrete Manual, U.S. Department of the Interior, Bureau of Reclamation, Denver, CO, October 1942.

"Hungry Horse Dam and Powerplant - Technical Record of Design and Construction", Bureau of Reclamation, May 1958.

"Morning-Glory Shaft Spillways - A Symposium", American Society of Civil Engineers, Paper No, 2802, Vol 121, 1956.

"Hydraulic Model Studies of the Morning-Glory Spillway for Hungry Horse Dam - Hungry Horse Dam Project," Hydraulic Laboratory Report No. Hyd-355, Bureau of Reclamation, April 23, 1954.

"The Hungry Horse Project," by Eric A. Stene, Bureau of Reclamation History Program, 1994.

Londe, P., "The Malpasset Dam Failure," Engineering Geology, Vol. 24, 1987, pp.295-329.

Design and Analysis of Auburn Dam, Volume One: Design Data (August 1977) and Volume Two: Foundation Studies (May 1978), United States Department of the Interior, Bureau of Reclamation, Denver, Colorado.

Scott, G.A., “Guidelines - Foundation and Geotechnical Studies for Existing Concrete Dams,” Technical Service Center, Bureau of Reclamation, Denver, Colorado, September 1999.

Risk Analysis Screening Study for Morrow Point Dam MDA, October 1998.

Morrow Point Dam and Powerplant - Technical Record of Design and Construction, Bureau of Reclamation, September, 1983.

Copen,Merlin D., “Design of Arch Dams by the Trial Load Method of Analysis”, Journal of the Power Division, Proceedings of the American Society of Civil Engineers, August 1960.

ADSAS User’s Manual, Bureau of Reclamation, Denver, Colorado.

Interview with Bob Main on June 16, 2001.

Interview with Louis Roehm on June 13, 2001.

Electronic mail message from Glenn Tarbox, August 20, 2001.

Hand-written notes from J. Lawrence Von Thun, September 17, 2001.

Scrivner, Loyd R., “Use of Electronic Digital Computers to prepare data for Trial Load Analysis of Arch Dams, “ ASCE Convention, New Orleans, Louisiana, Power Division, March 1960.

Main, Robert B. “Users Manual for the Arch Dam Stress Analysis System - PC - Version 2.00,” February 25, 1997.

Mills/Bria, Barbara and Payne, Terry “Geometric and Material Non-linearities in Finite Element Structural Analysis of Dams and Dam Appurtenances”, 2000.

Copen, By M. D., Rouse, G. C. and Wallace, G. B., “European Practices in Design and Construction of Concrete Dams,” Volumes 1, 2, and 3, February 1962.

Boggs, Howard L., “Guide for Preliminary Design of Arch Dams,” Engineering Monograph No. 36, Bureau of Reclamation, November, 1966.

“Speed and Grit Beat Problems on BuRec Dam, Upper Stillwater is RCC Benchmark,” ENR, October 29, 1987.

Richardson, A.T., “Performance of Upper Stillwater Dam,” Roller Compacted Concrete III, Proceedings of a Conference Sponsored by the American Society of Civil Engineers, San Diego, CA, February 2-5, 1992.

Scott, G.A., "Deformation of Rock Foundations Under Dams, Proceedings, Tenth Annual USCOLD Lecture Series, New Orleans, LA, March 1990.

Trojanowski, J., "RCC Used to Stabilize Pueblo Dam," *USCOLD Newsletter*, Issue No. 120, March 2000.

Hepler, T.E., and G.A. Scott, "Increasing the Capacity of the Roosevelt Dam," International Water Power and Dam Construction, Volume 44, Number 5, May 1992.