Section 2 Project History

2.1 History of Construction of the Upper Reservoir Dam

The top of Proffit Mountain was leveled and the excavated rock was used to construct the dike that forms the upper reservoir. The stone is predominantly a rhyolite porphyry. As described in available engineering reports, the overburden was stripped for the upstream-most 70 feet and placed downstream to form the bed of the perimeter road. All weathered material was stripped from this area to sound rock. Overburden varied from a few feet to as much as 65-feet thick. Clay seams were to be removed by excavating during construction. Excavated rock was end-dumped from trucks and sluiced with 30-psi water, to form the ring dike. A filter zone and several layers of compacted rock were placed over questionable areas where piping into the foundation might be possible. Outside the 70-foot stripped zone, the weathered rock was left in-place. Low areas in the natural topography were also filled with compacted rock. It was reported in the 1998 Seventh Part 12D Report that excavated fines were used to level the reservoir floor. A video of the original construction was provided by AmerenUE to show the construction of the embankments.

The dike is topped with a 12-foot layer of horizontally compacted rock placed in 4-foot lifts and compacted with a vibratory roller. The parapet wall was cast-in-place on top of this top layer. Based on post-breach inspections, it appears the crushed rock varies from 1000 lb stone to predominately less than 20 lb stone. The stone is predominately angular. The outer shell of the dike contains clean rock fill material with more sandy and pebble sized materials in the closure section, near panel 50.

2.2 Geology

2.2.1 Geology of Southeast Missiouri

The Saint Francois Mountains, a range located in southeast Missouri, is an outcrop of Precambrian igneous rock mountains rising over the Ozark Plateau. This range is one of the oldest exposures of igneous rock in North America. Formed through volcanic and intrusive activity over 1.4 billion years ago, nothing is left of these mountains but their roots. By comparison, the Appalachians started forming about 460 million years ago, and the Rockies a mere 70 million years ago. The St. Francois range was already twice as old as the Appalachians are today.

Unlike the rest of the mountainous areas in the Ozarks, the Saint Francois Mountains were formed by true volcanic activity. The localized vertical relief observed in most of the Ozarks, a dissected plateau, was caused by erosion. The volcanic activity that formed this mountain range is also thought to be the geological cause of the uplift of the Ozark Plateau. Geologists talk of the "Ozark dome" wherein elevations and stratigraphic inclines generally radiate down from the Saint Francois Mountains. These elevations may be the only area in the American Midwest never to have been submerged, existing as an island archipelago in the Paleozoic seas. Fossilized coral, the remains of ancient reefs, can be found among the rocks around the flanks of the mountains. These ancient reef complexes formed the localizing structures for the mineralizing fluids that resulted in the rich ore deposits of the area. The St. Francois Mountains are the center of the Missouri mining region yielding; iron, lead, barite, zinc, silver, manganese, cobalt, and nickel ores as well as granite and limestone quarries.

Mountains in this range include; Taum Sauk Mountain, Bell Mountain, Proffit Mountain, Pilot Knob Mountain, Hughes Mountain, Goggin Mountain, and Lead Hill Mountain. The Taum Sauk Hydroelectric Plant is actually not located on Taum Sauk Mountain, but on Proffit Mountain about five miles from Taum Sauk. Proffit Mountain is the termination of a ridge extending southwesterly from Taum Sauk Mountain. The elevations range from 500 feet to 1772 feet (Figures 2.1 and 2.2). Taum Sauk Mountain is the highest peak in the range, and the highest point in the state, with an elevation of 1772 feet. A part of the Ozark Trail winds through parts of the St. Francois Mountains, including a popular section that crosses Taum Sauk and Proffit Mountains. (From Wikipedia.)

The St. Francois Mountains are only a small remnant of the original volcanic activity in the area. It is thought that two continental plates collided during Precambrian times and led to the creation of the original mountains. Most of the rocks in the area are lighter weight rocks of a granitic composition. The darker dikes in the area, commonly found in road cuts, are formed from more basaltic minerals in the area and formed when rifting in the area started to split the plates apart about 900,000 years ago. These darker and heavier minerals originated deeper in the earth’s crust. This rift failed and is no longer active. Leftover faults from the collision and rift are now thought to form the New Madrid Fault Zone, which runs through far southeast Missouri. This fault zone is still active and has been responsible for some of the largest earthquakes in U.S. history.

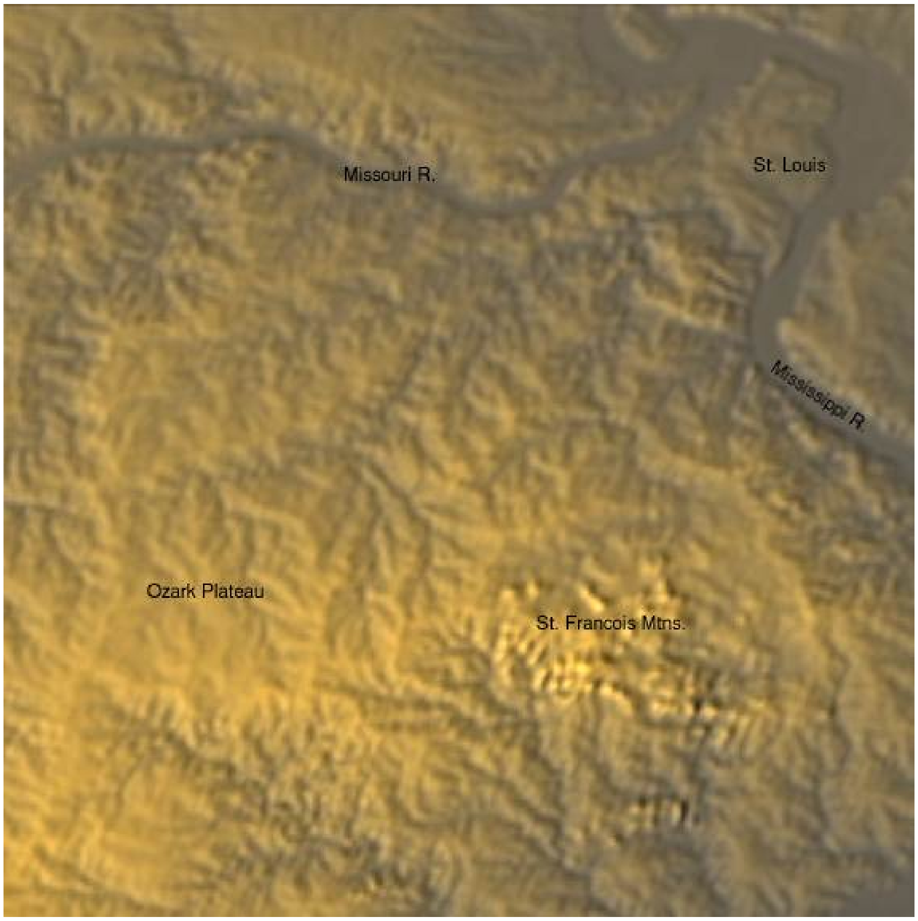


Figure 2.1 - Shaded relief map of area. (From Wikipedia.)

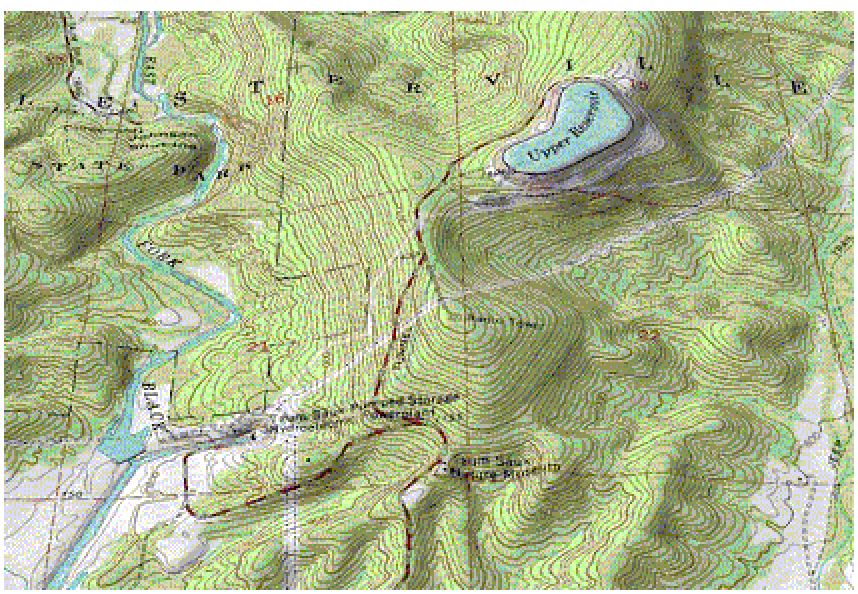


Figure 2.2 - Shaded topographic map of project. (From Wikipedia.)

2.2.2 Upper Reservoir Geology

The top of Proffit Mountain was leveled and the excavated rock was used to construct the dike that forms the Upper Reservoir (Figure 2.3). The foundation area was stripped to bedrock during the dam breach. The bedrock is hard rhyolite porphyry with areas of closely spaced vertical joints (Figures 2.4 and 2.5). This rock is volcanic and formed by relatively quiet lava flows on the earth's surface. These rocks are fine grained but contain mineral crystals that formed before the rock was erupted. Even though they are 1.4+ billion years old, these rocks still exhibit flow patterns from the original lava flow.

The vertical joints are in an orthogonal set that run roughly N-NE and W-NW. A second set of slickenside joints with lower dip angles were observed that had a line of intersection in a northerly direction. This joint set dipped roughly 45 degrees west and 45 degrees east. The rhyolite porphyry rests on granite porphyry, the contact is dipping easterly and is exposed just downstream of the breach area. During original exploration, it was conjectured the rhyolite had flowed out on the weathered surface of the granite, scorching and baking it. This means that the granite porphyry may be older than the rhyolite porphyry. However, there are different opinions regarding the age and sequence of intrusions and it is possible the granite porphyry is younger.

The series of Precambrian rhyolites at the adjacent Church Mountain form a stratigraphic sequence of flows that strike N45ºSW and dip 20ºN, and reportely have similar strikes as the rhyolites of Taum Sauk. As described in a 1973 report of the geology of the adjacent Church Mountain, the principle rock formations are:

"<u>Precambrian Hogan Mountain Rhyolite</u>. The rhyolite is 'typically reddish-brown, or reddish-purple in color and has a dense aphanitic groundmass. About 20 to 30 percent of the rock consists of salmon – red feldspar and glassy quartz phenocrysts. Flow layers, lighter in color than the massive rock, consist of microangular zones that generally dip at consistent low angles to the north and west... Many of the quartz phenocrysts and quartz grains in the ground mass are replaced by feldspar... The field relations and micro-textures suggest that this is a devitrified welded tuff'..."

<u>Precambrian Munger Granite Porphyry</u>. "This was encountered at the bottom of the upper Taum Sauk Dam... The predominant features are 'orthoclase phenocrysts up to 8 mm in length and quartz up to 4 mm in diameter. The rock is brownish-red with greenish mottling due to fine-grained mafic minerals. Quartz comprises about 30 percent: orthoclase, 33 percent: oligoclase (ab89), 33 percent: and extensively altered biotite and hornblende about 4 percent'...

.

Figure 2.3 - View of upper Reservoir.

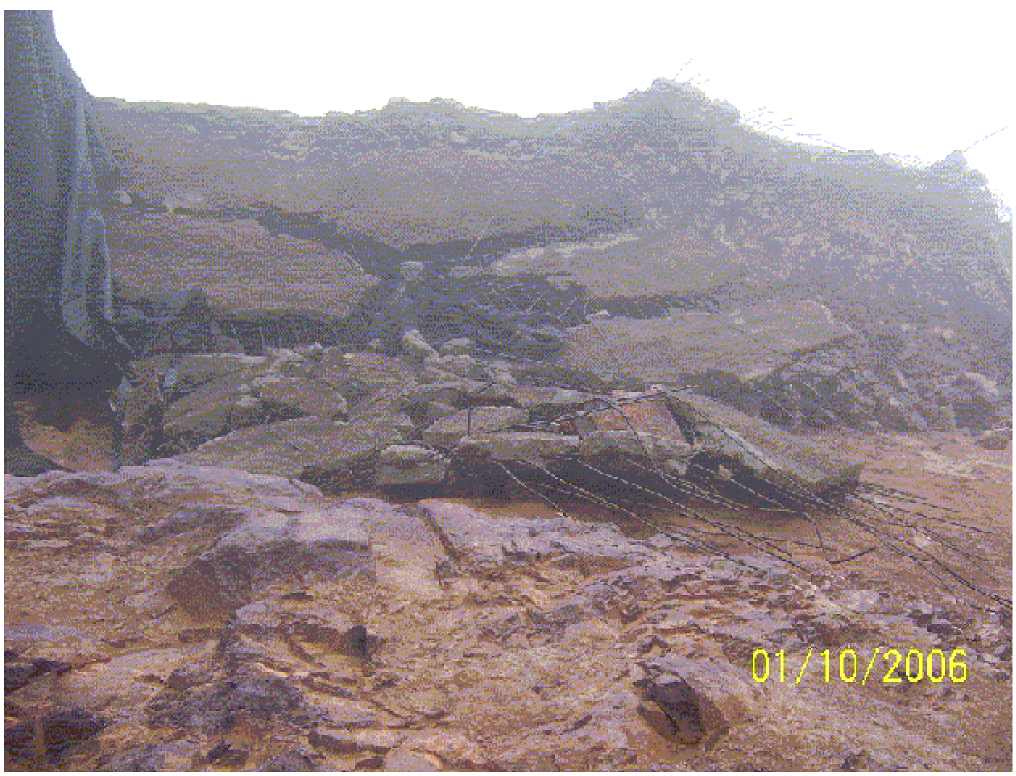


Figure 2.4 - View of bedrock immediately below failed embankment section.
Note weathered clay in lower portions of the exposure.

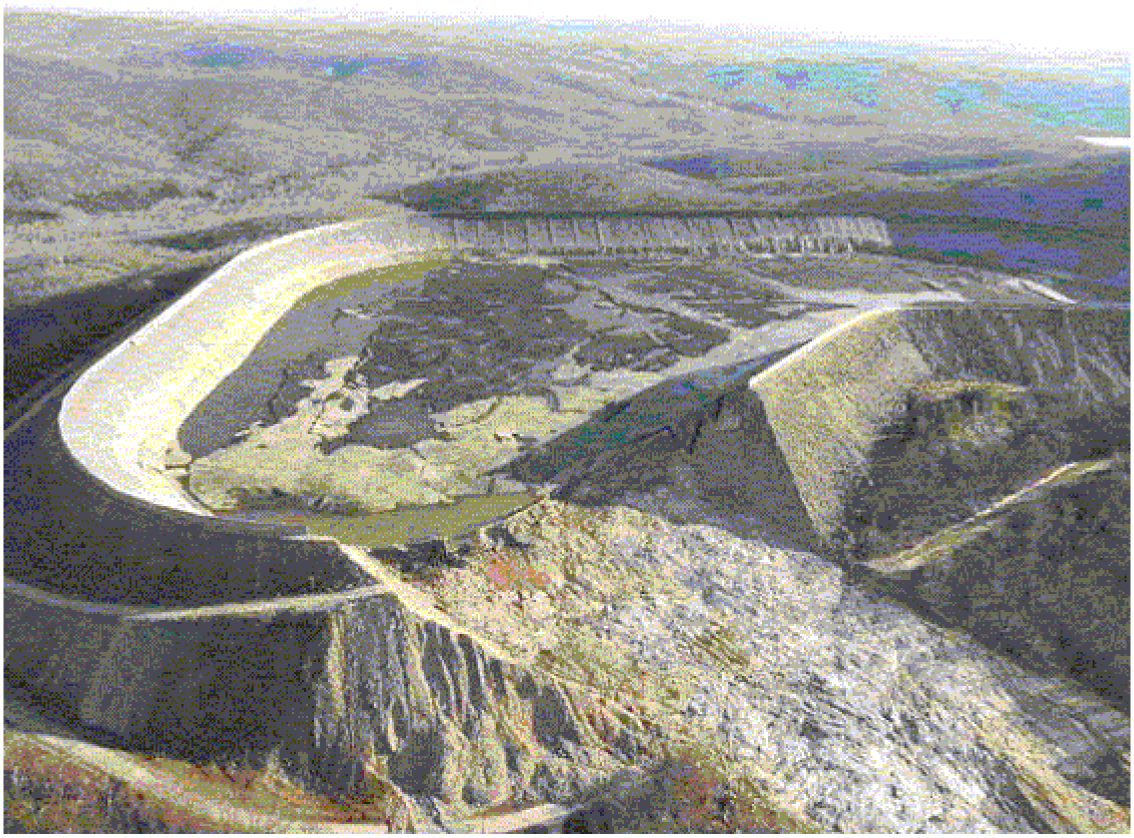


Figure 2.5 - View of bedrock within, and immediately below failed embankment section. Note distribution of weathered rock (red-brown color) and topsoil (green-brown color). The "fish-pond" area contains water and is immediately upstream of the breach.

2.2.3 Breach Foundation Geology

An area of the foundation in the breach section contained clay with low-moderate plasticity and weathered rock in the area just beneath the breach. The clay and weathered rock zone could have provided a failure surface for embankment sliding. The clay appears to be a residual weathering product of the bedrock (Figure 2.6), and in areas, relict bedrock structure can be observed in partially and completely decomposed clay remnants. No records were made available that indicate the extent of clay that was left in the breach area. However it could be conjectured that the settlement of the parapet wall in this area may have been accentuated due to consolidation of this clay deposit if it was of substantial thickness. The clay appeared saturated and contained groove marks from debris (Figures 2.7-2.9). This area was over-excavated in the footprint of the reservoir due to the clay foundation conditions, forming the "fish-pond" area of the reservoir floor (Figure 2.10). There was also some remnant soils found in the breach area (Figure 2.11 and 2.12).

According to the 1964 Union Electric Company memo on "Leakage From Upper Pond" (pages 1-2) "…the exposed rhyolite at levels uncovered still contain fingers of weathered rock.…on the west side a deeper zone of weathering was excavated near drill hole #18 about where considerable spring flow is found at the outer toe. An inclined clay band at Sta. 6+00 on the west side apparently crossed the floor of the pond and occurs on the opposite side of the basin. The clay band was trenched and back-filled with concrete before material in the rockfill or seal cover was placed. These geologic zones are reflected in response of pond by seepage that collect upon reaching the underlying rock and by air bubbles near the west bank along the clay band, following initial filling of the pond. Most of the exploratory drill holes in rhyolite had substantial loss of water… …that led to asphalt lining of the pond floor… …indicat(ing) that joints are communicative. At the north end (Panels 90-95) a sudden increase in losses between January 8-10 (1964) was caused by open channels (under the asphalt lining and) in bedrock under the dam where eroded material had been removed by gradual piping. It was necessary to add concrete cutoff in this section, fill the visible channels and attempt to control water movement along bedrock joints by means of a shallow grout curtain across the floor at the northern end of the pond. The work was largely successful but should be watched for further aggravated losses beyond the section that was repaired."

According to the August 1968 Union Electric Company memo on "Review of Safety Report – Upper Reservoir" (page 4) "…the rhyolite porphyry…is generally fresh, dense, moderately to abundantly jointed… …Overburden ran from a few feet thickness to as much as 65 feet. Several significant clay seams, gently dipping, and up to 4 feet in thickness were encountered. Under the rockfill these seems were either excavated and plugged with concrete or covered with small compact rock. Weathered rock was left in place wherever its competence was judged equivalent to the rockfill. However, within the inside 70 feet of the base of the rockfill all weathered material was stripped to sound rock. A filter zone and several layers of compacted rock were placed over questionable areas where piping of the foundation might be possible. Low areas or depressions in the natural topography were filled with compacted rock."

According to the August 1967 Union Electric Company memo on "Taum Sauk Upper Reservoir Report on Safety" (page 2) "The… …rhyolite porphyry is an excellent high compressive strength rock that should have stabilized in its settlement. However, the formation contained frequent zones of soft weathered rock, all of which could not have been selectively wasted. The frequent cycling of the water load should not cause continued adjustment of competent rock but would affect poor rock. Actually, there is no other experience with such frequent cycling of load on a dumped rockfill, and whether a dumped rockfill of all sound rock would have stabilized by this time (1967) is not known. I believe a fill of 100% competent rock would have stabilized and that the percentage of weathered rock in the Taum Sauk is the cause."

Figure 2.6 - Foundation area composed of weathered rhyolite with some clay, located in breach area.

Figure 2.7 - Grooves cut into an area of clay-seam foundation by the floodwaters. This area is located immediately downstream and in the center of the breach.

Figure 2.8 - Overview of area shown in previous photograph. Note washed out access road (circled).

Figure 2.9 - Overview of area to the right of that shown in previous photograph. (See also Figure 2.4.) Note breach in background and toe of breach slope on left edge of photograph.

Figure 2.10 - Breached area. Note "fish pond" area immediately upstream of the breach. This area was over-excavated during original construction due to poor foundation conditions.

Figure 2.11 - Weathered rock and discontinuous clay seam foundation just downstream of fish-pond area.

Figure 2.12 - Root-laden remnant native soil in breach area, resting on fresh rhyolite.

During a geologic inspection conducted on April 12-13, 2006, it was noted that there is a shear zone that cuts through the breach area. There appeared to be a component of left lateral displacement across the shear zone. The exposed rhyolite beds in the breach area were composed of three to four descrete flows, separated by very thin to thin clay rich seams. The rhyolite complex rests on a saprolitic soil that was interpreted to be heavily weathered granite. The underlying granite appeared moderately weathered with alteration of the feldspars near the overlying contact with the saprolite, and was less weathered deeper in the profile. The rhyolite sequence varied from dark red-brown to purple brown flows with occasional lineation of the phenocrysts. The rhyolite resting on the saprolite was black and contained veins of clay near the base. The contact between the rhyolite and granite was water bearing. The saprolite varied from several inches to as much as 10-feet in thickness. Based on construction documentation, the saprolite appears to be present in the shaft and therefore may extent beneath the entire reservoir at unknown depths. Rock outcrops on the southwest side of the reservoir indicate the contact between the granite and rhyolite passes beneath the reservoir foot print, possibly in the area of Panel 60-75. However, more site work is needed to define the location of this contact. Boring information taken in preparation for reconstruction of the Upper Reservoir indicates that there is as much as 200 feet of relief on the granite surface, within the immediate reservoir area.

2.2.4 Geology of Johnson's Shut-Ins State Park

The Johnson's Shut-Ins State Park is located on the East Fork of the Black River which carved through fractures in hard volcanic rock to form a natural water park. The area is home to waterslides, waterfalls, a small underwater shelter, whirlpools, and much more. The rocks vary in color from pink to black depending on the nature of the volcanic eruption that led to their creation. The rocks at Johnson's Shut-ins consist of welded tuffs and ignimbrites, rocks formed from extremely violent volcanic eruptions. These rocks form when clouds of hot volcanic ash roar down a mountainside and settle to the ground. The residual heat melts the ash and 'welds' it together to form a rock. These rocks are very hard and form shut-ins where rivers have tried to erode. Shut-ins form when a stream down cutting softer rocks runs into harder rocks. These harder rocks channel the river into a smaller area and make for interesting scenery.

2.3 Design and Stability

2.3.1 Embankment Design and Stability Analysis

Stability of the embankment (slopes 1.3H:1V, 37.56º) was evaluated in the 1988, assuming fully drained conditions. The consultant performed an infinite slope stability analysis (using two phi angles) and compared this with a "SLOPE" stability analysis performed for a concrete faced rockfill dam with similar section geometry and fill properties. Zero cohesion and a phi angle of 45 degrees were assumed in the SLOPE computer analysis. Zero cohesion and a phi angle of 50 and 47 degrees were assumed in the infinite slope method. The infinite slope method yielded the lower factors of safety. The rockfill dike was also evaluated for seismic loading using a pseudostatic seismic coefficient of 0.14g for the SLOPE analysis and 0.10g for the infinite slope analysis. Estimated factors of safety for the dam for these various loading conditions are summarized below:

Method	Static Φ (50º/47º)	Seismic
Infinite Slope	1.55/1.39	1.26/1.14
SLOPE	1.7[1]	1.3

[1]A factor of safety of 1.5 was obtained for shallow sloughing failure surfaces. The factors of safety increase proportionally to the depth of the failure circle for the assumed homogeneous and drained materials.

Following breach of the dam, core material observed in the breach area contains a much larger percentage of finer materials than the outer shells. The processed materials, including the sand sizes, should be highly angular in nature and be similar to the values used in 1988.

2.3.2 Parapet Wall Design and Stability

A 10-foot-high, 1-foot-thick reinforced concrete parapet wall sits atop the crest of the Upper Reservoir Dam. The wall is comprised of 111 panels, each approximately 60 feet in length. The design crest elevation of the wall was 1599 feet. Originally, the project was to operate with a maximum operating level of 1597 feet, which would have stored 8 feet of water on the 10-foot-high panels. Since construction the crest elevation of the parapet wall has decreased and varied due to settlement of the dam (See Section 3.1 – Settlement).

The majority of the panels were placed using conventional forms and concrete. Figure 2.13 is a typical cross section of the walls, with a load diagram and overturning analysis from Drawing 8304-X-26157:

The initial portion of the concrete parapet wall, panels 3-20, were pneumatically placed concrete similar to the concrete face slab. However, this section was not accepted due to poor quality concrete. Based on a 1963 design drawing (8304-X-26122), parapet panels 3-20 were modified to have a sloping reinforced-concrete wall placed upstream of the parapet stem, with the area between the two walls filled with 3,000 psi concrete (see Figure 2.14).

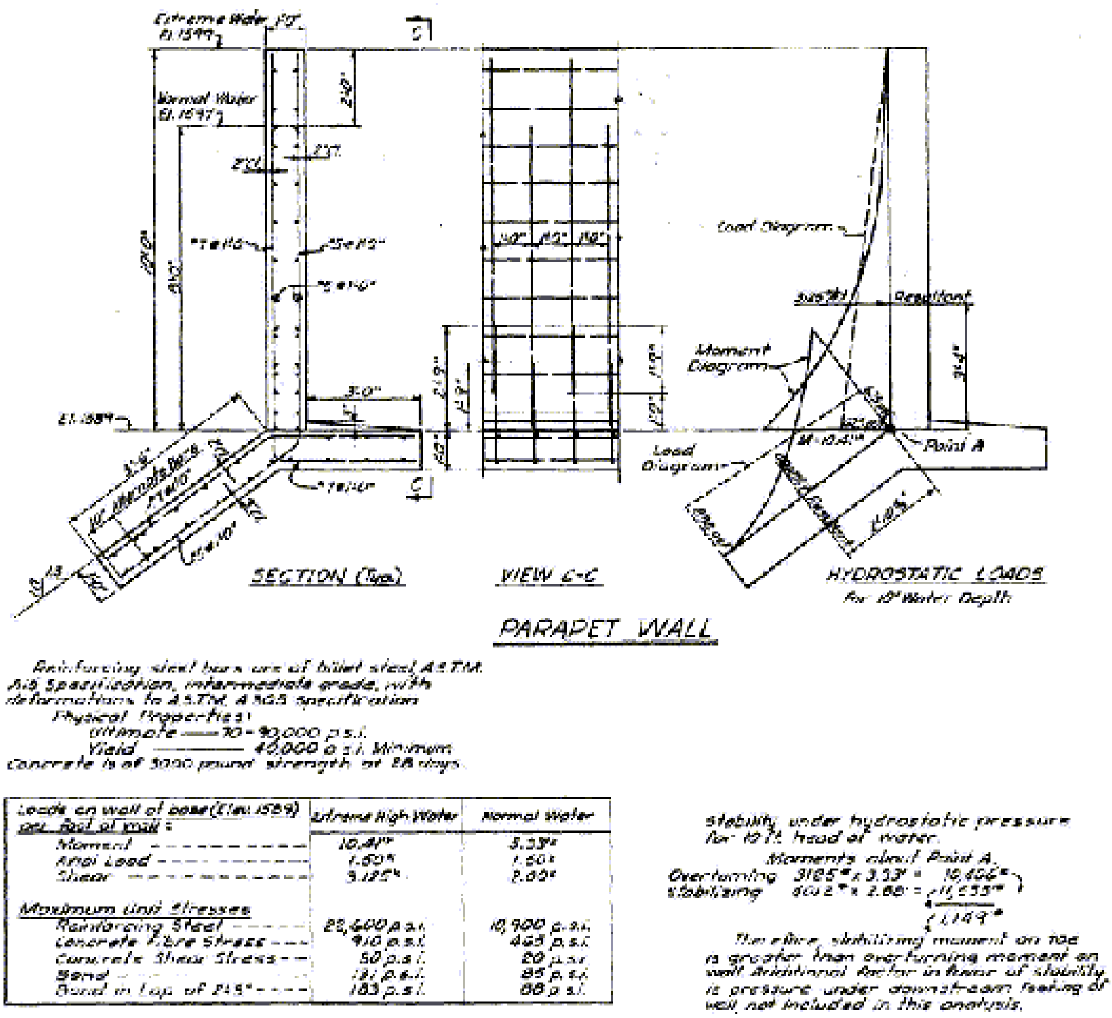


Figure 2.13

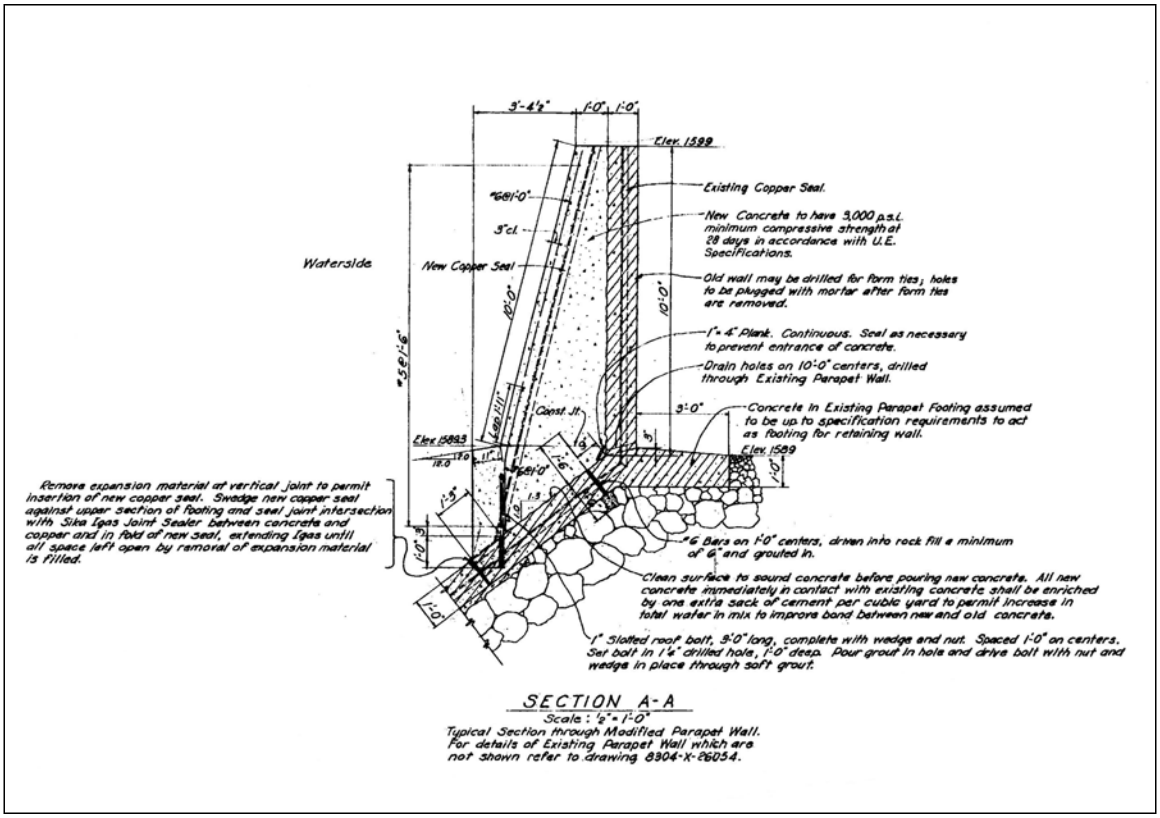


Figure 2.14

<u>Parapet Wall Stability</u>

Design Drawing 8304-X-26157 (Figure 2.13) shows the following results from an overturning stability analysis, taken about the point where upstream vertical surface intersects the sloping base of the parapet wall:

Overturning Moment: 10,406 Kip-feet
Stabilizing Moment: 11,155 Kip-feet

Taking moments about the downstream heel of the wall (Pt. A on Figure 2.15) yields an overturning factor of safety of 2.45. This calculations neglect uplift on the base of the wall, assuming a free draining foundation.

The Design Drawing includes a table with the maximum loads and unit stresses in the parapet wall under the normal (elevation 1597 ft) and extreme (elevation 1599 ft) water levels. The following table provides the maximum stresses as well as a comparison to allowable stress based on the ACI-Alternative Design Method, which would have been commonly used at the time of the dam design.

	Maximum Unit Stress (psi) (water level at 2 feet below top of wall)	Maximum Unit Stress (psi) (water level at top of wall)	Allowable Stress (psi) (ACI – Alternate Design Method)
Reinforcing Steel	10,900	22,600	20,000
Concrete, Fibre Stress (Flexure)	465	910	1,350
Concrete, Shear	20	30	60.2
Bond	85	131	

The table indicates the maximum unit tensile stresses in the reinforcement would meet the Alternate Design Method code for water levels two feet below the top of wall. For the extreme case of water levels at the top of the wall, the maximum unit tensile stress is higher than the allowable stress.

FERC Staff also compared the wall design to the 2005 Load Factor Design code:

Loading	Applied Moment (kip-ft)	Mu (kip-ft)
Water Level 2 Feet Below Top of the Parapet Wall	5.32	7.45
Water Level at the Top of the Parapet Wall	10.41	14.57

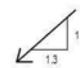
ΦMn: 15.20364 (assuming full development length)

Check development length
d: 10 in.
Cover: 2 in.
Dia: 0.875 in.
As: 0.60132 si
Fy: 40 ksi
Fc’: 3 ksi
ld: 21 in.
Splice required: 27 in. (greater than the 21 in. provided)
Class B: 1.3 - Divide ΦMn by 1.3

ΦMn/1.3: 11.7 k-ft

The wall design meets Load Factor Design code when water levels are two feet below the top of the parapet wall. For the extreme condition with water levels at the top of the wall, Mu exceeds ΦMn/1.3.

Stability calculations were also included in the January 3, 2002 Design Report for the liner installation. Included with the anchorage calculations are stability checks of the existing parapet wall with imposed liner anchor loads. The design indicates there would be two anchors on the parapet wall:

Anchor	Location	Force (kips/linear foot)	Force Direction
Upper	One foot from the top of the wall	0.12	Down ↓
Lower	On the sloping base about one foot below the intersection with the vertical wall.	2.05	Along sloping wall into the reservoir. 

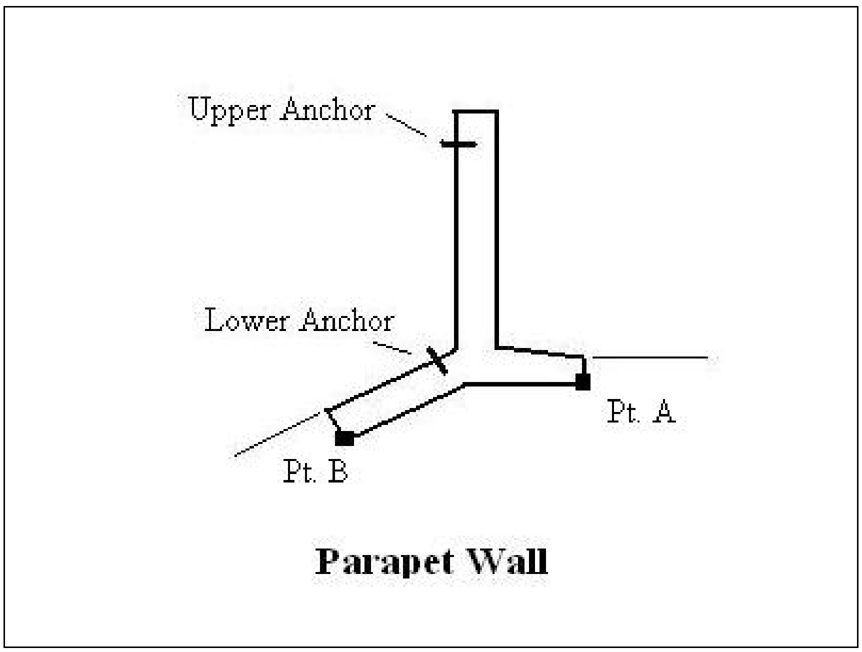


Figure 2.15

These forces result in a net increase in the stabilizing moment about the downstream heel of the wall (Pt. A on Figure 2.15). The Design Report includes calculations showing the effects of the anchor on overturning about the upstream toe of the sloping wall slab (Pt. B on Figure 2.15). This was done to determine if the anchors would pull the wall into the reservoir, especially when the water level was just below the parapet wall. The factor of safety for overturning about the upstream toe was determined to be 5.64.

2.4 Geomembrane Design

Repairs for the cracked and leaking concrete face of the dam consisted of an 80 mil liner manufactured by GSE Lining Technology, The original design (2001) included an underlayment consisting of a nonwoven polypropylene fabric and a HDPE geogrid drainage mat at the toe of the liner along the perimeter toe of the upper reservoir. The final design (2004) underlayment was a geogrid drainage mat with both sides bonded to a non-woven geotextile. The underlayment was placed from the toe block to the bottom of the vertical section of the parapet wall. A concrete anchor block with an embedded anchor section compatible with the geomembrane was installed at the base of the toe block of the concrete panels. The liner was anchored at an elevation approximately 1 ft below the top of the wall with Hilti type anchor bolts and near the top of the upstream footing. The original liner specifications call for concrete infilling of surface irregularities prior to placement of the geotextile/drainage underlayment and the geomenbrane. A geofoam filler was also used at the modified section of the parapet wall.

The December 2001 reservoir lining plans submitted to FERC on January 2, 2003 included notes that covered the demolition and removal of the original reservoir monitoring system, supporting concrete and staff gauges. Removal of the original instrumentation can be seen in the 2004 FERC Operation Inspection Report. A photograph of the new system was shown in the licensee's November 30, 2004 construction report.