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W. R. Stroman
Rone Engineers, Dallas, Texas

R. R. W. Beene
Consulting Civil Engineer, Fairfax, Virginia

P. D. Thornhill
Espey, Huston & Associated, Austin, Texas

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W.R. Stroman
Civil Engineer, Rone Engineers, Dallas, Texas

R.R.W. Beene
Consulting Civil Engineer, Fairfax, Virginia

P.D. Thornhill
Civil Engineer, Espey, Huston & Associates, Austin, Texas

SYNOPSIS: Kerrville Ponding Dam is a relatively small channel dam in the Guadalupe River in Kerrville, Texas. The dam is 22 feet (6.7M) high and 600 feet (183M) long and provides a water supply for the City of Kerrville. The dam was constructed during 1979-1980. Seepage problems in the abutments were observed during the initial filling of the reservoir. Some minor corrections to the problems were made at that time. In June 1981, after a moderate flood flow passed over the dam, additional seepage problems occurred. The downstream slope protection was displaced to the extent that cracks appeared in the concrete. This damage led to a more significant amount of repair. On December 31, 1984 the dam was overtopped by a flood to a depth of 10.5 feet (3.2M) above the spillway elevation. The dam suffered severe damage including loss of a portion of the concrete cap and significant erosion of the clay core over approximately one-third of the length of the dam, and seepage related damage at both the abutment areas. Figure 1 shows the conditions of the structure in January 1985. The dam suffered a "Type 1 Accident" as defined according to International Commission on Large Dams (ICOLD).

INTRODUCTION

Kerrville Ponding Dam is owned and operated by the Upper Guadalupe River Authority, a functional entity of the State of Texas. The dam forms a lake having an area of 105 acres with a storage capacity of 840 acre-feet at normal pool elevation. The damaged Kerrville Ponding Dam consisted of a rolled clay core with 8-inch thick concrete upstream and downstream facing. The service spillway is 21 feet (6.4M) high and approximately 200 feet (60.9M) long. The overflow spillway is 22 feet (6.7M) high and 400 feet (121.9M) long. A centerline profile, looking upstream, is shown in Figure 2. At the abutments the embankment rises on 3 horizontal to 1 vertical slopes to elevation 1635 msl. Cast-in-place concrete cut-off walls extend from the clay core for a distance of 128 feet (39M) into the north abutment and 183 feet (56M) into the south abutment. It was intended that these cut-off walls would bottom a few inches in the sound rock and extend to the bottom of the impervious clay fill.

SUBSURFACE CONDITIONS

A typical subsurface profile along the axis of the dam, showing the riverbed formation, the natural abutment materials and the rolled filled embankment is depicted in Figure 2. Kerrville Ponding Dam is built almost entirely within the Guadalupe River channel. The river bed exposes a marly clay shale (argillaceous limestone) of the Glen Rose formation, Cretaceous System. The fresh, unweathered limestone is dark bluish-gray in color, is intact, and contains no fissures or fractures. The rock rises approximately 10 feet (3M) above the incised river channel. In the north abutment the upper 5 feet (1.5M) of the limestone is less marly, more earthy, and is highly fissured and jointed with the fissures and joints principally oriented parallel to the

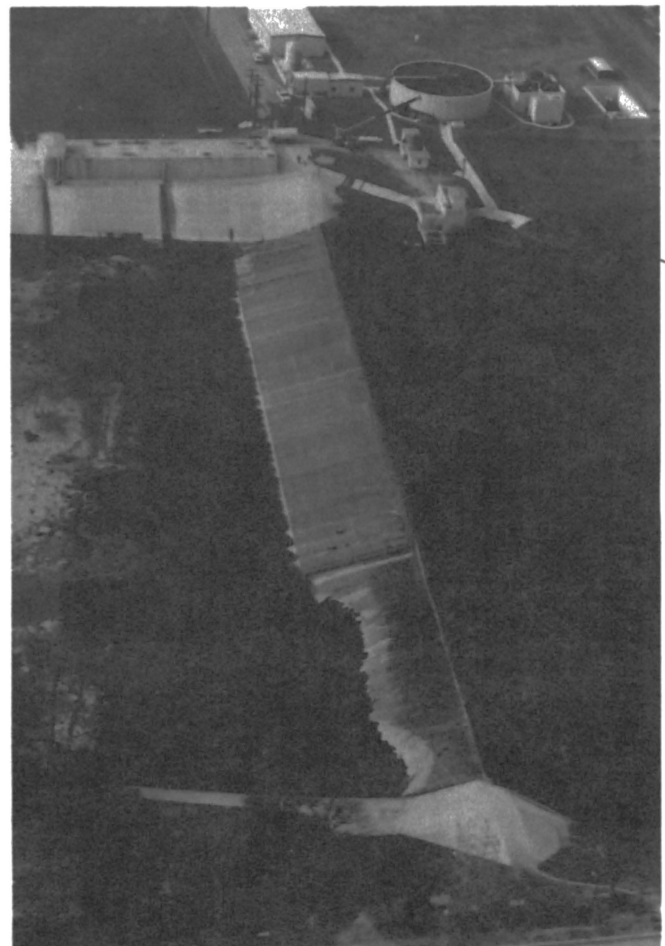


Fig. 1 Kerrville Ponding Dam, Jan. 1985

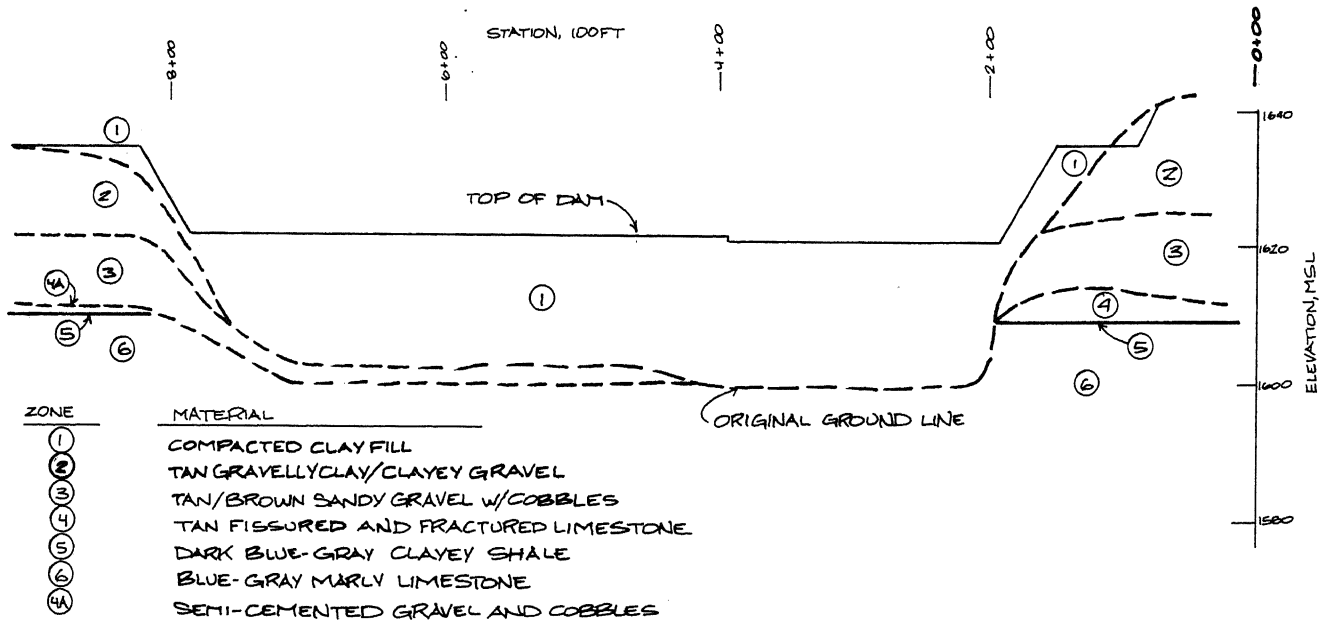


Fig. 2 Subsurface Profile

river flow. These fissures and joints are as much as 3 inches (76mm) wide and extend entirely through the upper limestone bed. On the south abutment, the sound marly limestone is overlain by a poorly cemented, highly vuggy and porous, gravelly and cobbly conglomerate which varies in thickness from 1 to 3 feet (0.3 to 0.9M). A thin bed of high plasticity clay, varying in thickness from 3 to 6 inches (25 to 150mm), lies between the sound rock and fissured rock in the north abutment and between the sound rock and the conglomerate in the south abutment.

On both abutments and the fissured limestone and conglomerate overlain by a loosely compacted silty and/or sandy gravel containing particles up to cobble size. This bed is approximately 10 feet (3.0M) in thickness and becomes more gravelly with depth. Overlying the gravel bed is an approximate 10-foot (3.0M) thick bed of clayey gravel/gravelly clay containing fines of moderate plasticity characteristics.

ORIGINAL PROJECT

Typical embankment sections are shown in Figure 3. The earth embankment consisted of moderate plasticity clay having liquid limit values ranging from 50 to 60. The clay was compacted to not less than 95% Standard Proctor density.

A 12-inch thick layer of filter drainage material was placed both beneath the bottom of the embankment, between the embankment and rock surface, and on top of the embankment, between the embankment and the concrete slope paving. The filter materials were obtained from natural sandy gravel deposits upstream of the dam. The upper filter blanket extended from the toe of the dam to the upstream edge of the crest beam as shown in Figure 3. In the river bed section the lower filter blanket extended from the toe to the centerline of the embankment. At the abutments the lower filter blanket extended

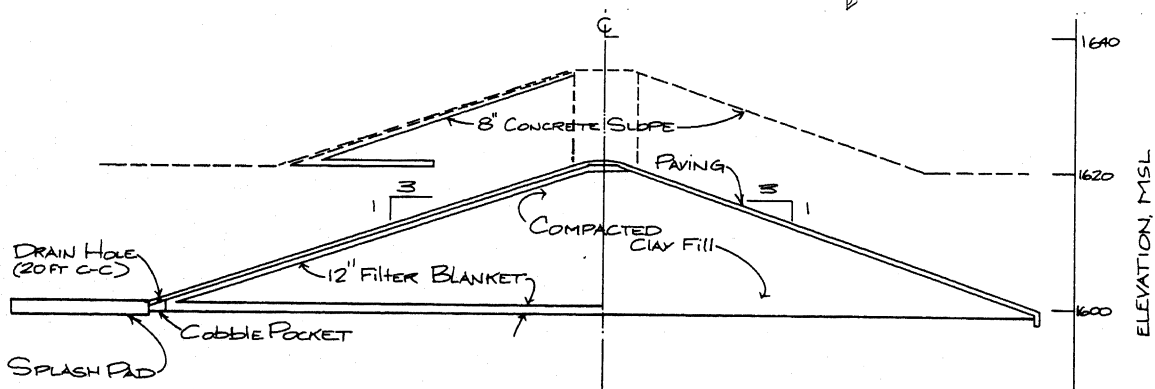


Fig. 3 Typical Embankment Section

from the toe to a point one-third the base of the embankment. The embankment was protected from erosion and scour by a lightly reinforced 8-inch thick concrete slab. A 2-foot thick by 20 feet wide concrete splash pad was placed along the downstream toe. The concrete slope paving was dowelled into the splash pad. Ten-foot high training walls were placed adjacent the steep river banks downstream of the embankment and extended approximately half-way into the downstream slope.

Four-inch diameter weep holes were placed on 20-foot (6.1M) centers at the juncture of the downstream slope protection and the splash pad and along the downstream training walls. To prevent loss of filter materials through the weep holes, one cubic foot pockets of cobbles ranging in size from 4 to 12 inches were placed between the filter zone and the weep holes.

Slope paving extended up the abutments to the crest of the non-overflow section as shown by the dashed-line embankment in Figure 3. A crest width of 10 feet (3M) was established for the non-overflow section. Slope paving was extended land-ward until its elevation of 1635 msl met natural ground surface. The cast-in-place concrete cut-off was constructed in open ditches excavated with a back-hoe through the cohesive and non-cohesive abutment materials.

Construction of the dam began in the Fall of 1979 and was essentially complete in the Fall of 1980. Impoundment began in Aug. 1980 and the reservoir was essentially full by October 1980.

EARLY DEFICIENCIES

During construction of the north abutment in the spring of 1980 it was discovered that the concrete cut-off wall had been bottomed on top of the fissured and fractured limestone. When excavations were being made for the left downstream training wall, water was observed flowing from the fissures. An additional cut-off wall upstream and adjacent the initial wall and bottoming a minimum of 1 foot (300mm) into the sound rock was installed. The top of this additional wall abutted and overlapped the bottom of the initial wall for a distance of 1 foot (300mm).

In September 1980, when the reservoir was first filling, excess seepage was observed in the south (right) abutment area. Significant seepage was observed coming through construction joints in the downstream slope paving slab. At one location a stream of water was observed jetting approximately 8 inches (200mm) above the slope paving at a point some 9 feet (2.7M) below the then existing reservoir elevation. A portion of the slope paving adjacent the right downstream training wall was removed and a 2-foot by 2-foot (500 x 500mm) aggregate filter surrounding a 4-inch (100mm) perforated pipe was installed. The concrete slope paving was then replaced. The exit point for the pipe was established at the downstream end of the right training wall.

Kerrville Ponding Dam was dedicated in June 1981. During the Spring and Summer of 1981 additional problems related to performance of

the embankment structure appeared. A storm had produced a flow 5.6 feet (1.7M) passing over the spillway. After the flow had subsided, a "rooster-tail", i.e. a fan-like spray of water, appeared on the downstream slope. This "rooster-tail" was actually produced by a crack in the concrete slope paving with an attendant 3/8-inch (9.5mm) vertical displacement. When the flow had further receded and the downstream slope paving could be more closely examined, several cracks in the slope paving could be seen. Most notable of these cracks was a continuous crack approximately mid-way down the downstream slope that extended from abutment to abutment. The condition beneath the concrete slope paving was investigated by coring 4-inch (100mm) holes through the concrete. It was found that a void space having a depth of as much as 3-inches in thickness (75mm) had formed beneath the slope paving. The surface area defining the lateral extent of the void covered a large percentage of the northern one-third of the embankment. This condition was "repaired" by drilling several holes through the concrete paving into which a mixture of concrete sand, water and "Revert" (TM) was placed; mostly by gravity flow but in some instances by pumping. Although no surveys were made to establish relative elevation of the slab, observers seem to think that the slab was in a "bowed" position at this time. The holes drilled through the concrete were filled with concrete and cracks in the slab were sealed with an elastomeric compound.

FAILURE OF DAM

During the night of December 31, 1984 the dam was overtopped by a flood to a depth of approximately 10.5 feet (3.2M) over the spillway section. The dam suffered extreme damage, including complete destruction of the downstream concrete slope protection. Approximately one-half of the concrete was completely removed from the left spillway section. The clay core eroded in some places as far as 10 feet (3.0M) beneath the remaining concrete paving slab on the left one-third of the dam. The concrete paving slab adjacent the south (right) abutment was cracked and displaced upwards as much as 1 foot (300mm). A significant amount of filter material was eroded from beneath the slab for a distance of 60 feet (18M) from the south abutment. A photograph showing the scope of damages is shown in Figure 1. The dam was not breached, so the incident could be classed as an ICOLD TYPE 1(1A) Accident. The reservoir was drawn down to elevation 1613 msl to alleviate further damage and yet maintain a water supply source for the City of Kerrville and, except for occasional rises of the river during the investigation and repair phase, was maintained at that level until repairs to the structure were completed.

INVESTIGATION OF FAILURE, FINDINGS

Several theories as to the cause of the damage were propounded by various personalities. Among the causes put forth was the effect of strong eddy currents and turbulence developing negative pressures against the concrete cover. Another cause considered was the beating and pounding action of logs and other debris caught in the roller waves downstream of the dam. However, careful observations of the condition of the dam

and its abutments, review of pre-construction geotechnical investigations together with construction quality control records, the incidences of seepage during construction and first filling and the remedial procedures used to mitigate these conditions, coupled with results of a fairly extensive subsurface investigation made after the failure gave strong indications that the basic problem dealt with the design of the dam; its internal hydrostatic control and drainage system and the communication established between the drainage system and the naturally occurring pervious materials found in the abutments.

Samples of the undisturbed filter materials used in constructing the dam were obtained for testing. Grain size analyses showed percent fines (-No. 200 sieve) varying from 5 to 18 percent. Atterberg limits tests indicated plasticity index values on the order of 8 to 15. A typical grain size curve for the filter material is shown in Figure 4.

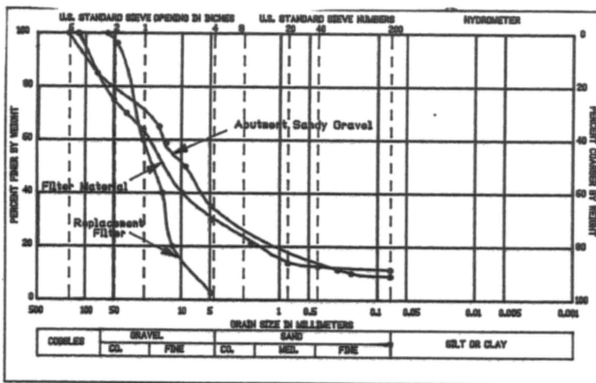


Fig. 4 Grain-Size Curves, Natural and Filter Materials

Bulk samples of the natural gravel stratum were obtained from large diameter test holes dug into the abutments both upstream and downstream of the embankment. Grain size analyses typical of the materials are also shown in Figure 4. It is obvious that these materials have very similar grain size characteristics. Analyses of grain size curves indicate that the materials are of a suffusion character. The D_{60}/D_{10} ratio are well in excess of 20 which according to U.S. Army Engineer Waterways Experiment Station Report "Design and Construction of Granular Filters for Embankment Dams" by Edward B. Perry, indicate the materials are internally unstable and susceptible to internal migration and possible loss of the finer particles.

Trenches were excavated into both abutments at locations where the cut-off trench could be exposed for inspection. On the south (right) abutment it was found that the concrete cut-off bottomed some 4 to 8 inches (100 to 200mm) above the sound rock. Very loose coarse gravel was found in the void between the concrete cut-off and the rock. In the north (left) abutment a void ranging from 2 to 6 inches (51 to 152mm) was found between the concrete and the fissured rock. Additionally, it was found that the concrete had been cast in three separate sections with coarse gravel intruding into the

Joints in the concrete. The triangular joint in the center of the two right pieces of wall is about 6x8 inches (152-203mm) in size. This condition is depicted in Figure 5. A 36-inch (0.9M) boring was made upstream of the cut-off to inspect the juncture between the original cut-off and the extended cut-off. It was found that an approximate 2-inch (51mm) vertical gap existed between the two walls. The gap was filled with loose coarse gravel and tree roots up to 1-1/2 inches (38mm) in diameter.



Fig. 5 Defects in North Cut-off Wall

BASIC CAUSE OF FAILURE

Based on visual observation, exploratory borings, and laboratory testing it was concluded that the distress to the dam was the result of three basic flaws:

1. The defects in the cut-off walls provided direct access of significant quantities of water under excess head from the reservoir into the filter materials in the area downstream of the crest of the dam. The sum of areas providing relief for the water flows was significantly less than the sum of inlet areas. Consequently the filter materials became overloaded, high hydrostatic pressures developed beneath the paving slabs, thereby lifting and disrupting the slab. Once the slabs were displaced and broken, the rush of water over the crest completed the destruction shown in Figure 1.
2. It is noted that the so-called filter material violated several of the recognized filter criteria when using the gravel as the base material. It can be said that the structure did not have a filtration or pressure reducing system in-so-far-as the more permeable base materials were concerned. Also, the filter materials were quite variable as to percent fines. This produced a great variation in filter permeability. Much of the filter contained significantly more fines than are usually permitted in a well designed filter material.

- The "cobble pockets" at each of the weep holes were without any direct control as to particle size or size of voids produced. The size of the drain hole with respect to the size of the cobbles were such that one cobble could completely block a drain hole. Close examination indicated that perhaps only 4 of the 20 drain holes in the dam that remained after the accident were actually working.

REPLACEMENT DAM

Constraints placed on repair to the dam included that the repair structure should fit within the footprint of the original dam. Also, the reservoir remaining behind the damaged structure could not be lowered more than 10 feet (3.0M) without adversely impacting the City of Kerrville's water supply. Six basic options were considered: (1) remove the existing dam and replace it; (2) repair and replace the damaged portion in accordance with original plans; (3) construct a vertical concrete buttress dam; (4) replace the clay core dam with one having a much thicker concrete cap; (5) move the dam to a new location; and (6) replace the dam with a roller compacted concrete (RCC) dam.

Whatever of these solutions were adopted, it was considered imperative that the downstream pressure relief filter system would require replacement and that a new, continuous cut-off system would be necessary.

It was determined that a roller compacted concrete (RCC) dam offered several advantages to the other considerations. Not the least of these was the consideration that a RCC dam could withstand overtopping with a minimal damage to work in place.

Along the Guadalupe River and within easy haul distance of the dam site, several gravel bar deposits were being mined commercially to produce concrete, asphalt and base course aggregate. Generally, the sand and gravel is processed by washing and screening and/or crushing and blending to meet American Society of Testing Materials or Texas Highway Department specification requirements. Samples of the processed materials and pit-run materials were obtained from several of these pits to evaluate their potential use in producing roller

compacted concrete. Specification limits for grain size distribution were established as shown in Figure 6. Also shown are the average results of grain size analyses tests conducted on 52 samples taken from the mix plant during construction.

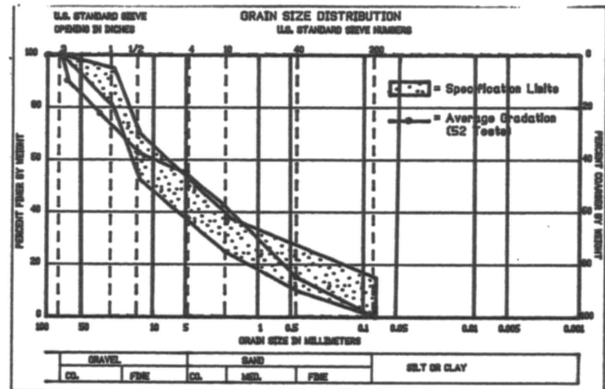


Fig. 6 Roller Compacted Concrete Gradation

Grain-size analyses of pit-run materials, and visual examination, showed that the various producers were all mining essentially the same materials. Grain-size distribution curves of materials "scalped" over a 3-inch screen were compared with curves reported by other RCC users and were found to conform quite well. Trial mixes showed that adequate strength, and durability could be obtained with moderate amounts of cement. Both Wetting-and-Drying and Freezing-and-Thawing (ASTM D559 and ASTM D560) on laboratory molded specimens showed losses ranging from 6 to 10 percent. Tests also indicated that the molded materials would develop compressive strength ranging from 1100 to 2000 psi at 28 days age on materials containing from 5 to 10 percent portland cement, by weight. The durability and quality of the aggregates themselves were well established by their use in numerous projects throughout the area.

The final embankment section for the RCC dam is shown in Figure 7. The RCC structure was designed to not only resist the forces generated by the reservoir but also to support the

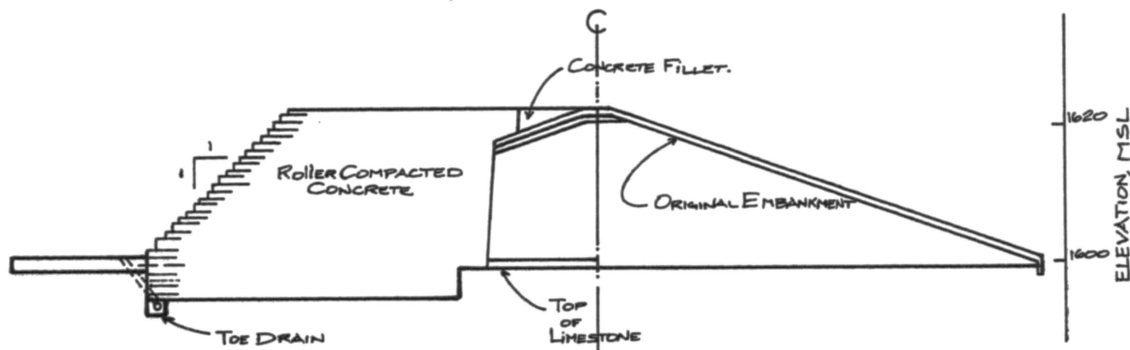


Fig. 7 Repaired Embankment Section

embankment that was to remain in place once the dam was completed. Based on Culman's method to determine the stability of near-vertical slopes composed of clay soils, it was decided that a vertical slope of 17 feet (5.2M) would be safe against failure for some reasonable period of time. Consequently, the original downstream slope was removed to a point some 16 feet (4.9M) downstream of the axis of the dam and a vertical slope established. The slope stood throughout the construction period of some four months with only minor sloughing even though it was overtopped by flood flows on two occasions. The portion identified as "original embankment" in

Figure 7 served to contain a reservoir during the construction period."

Roller compacted concrete was placed in 1-foot (300mm) thick lifts and rolled with a 10-ton vibratory steel wheel roller. It was decided that the bottom seven lifts and top five lifts of RCC would contain 10 percent portland cement by weight and that the interior lifts would contain 5 percent cement. This decision was based on the desire to provide a higher strength material in the base of the dam where tensile forces would be greatest and to provide more erosion resistance at the top of the dam.

Moisture and density control were conducted on a daily basis. Additionally, laboratory molded specimens were made from the materials being placed. Results of strength tests of laboratory molded specimens are shown in Figure 8. After the dam was completed both 6-inch (152mm) and 10-inch (250mm) specimens of in-place RCC were obtained with a diamond core barrel. Strength tests were conducted on specimens of the core. Results of these tests are shown in Figure 8 for comparison with laboratory molded specimens. Down-hole permeability tests were conducted in the core holes. Permeability values obtained from these tests ranged from 1×10^{-5} to 1×10^{-7} cm/sec.

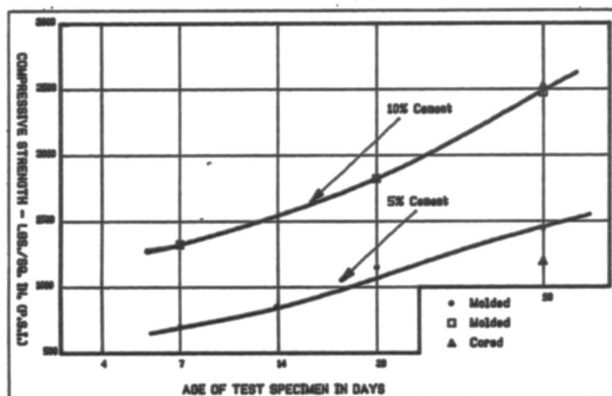


Fig. 8 Strength of RCC

The seepage through the abutments was controlled by constructing overlapping drilled cassions to distances of 130 feet (39.6M) into the abutments. The shafts extended from top of dam surface to at least 3 feet (0.9M) into the unweathered rock. The drilled shaft cut-off wall consisted of two rows of shafts. Shafts

forming the downstream row were 36 inches (914mm) in diameter spaced on 48-inch centers. Approximately 3 days after the concrete was cast in these shafts an 18-inch (460mm) shaft was drilled in the intervening space. Construction of the shafts were closely monitored to insure vertical continuity. The performance of the cut-off walls, when the embankment was overtopped near the end of construction, indicated perfect performance of this control feature.

The downstream pressure relief system (filter material) was redesigned using ASTM Standard Concrete Aggregate, 2-inch to 3/4-inch size. Estimated permeability of this material is quoted to be greater than 50 feet per minute (25 cm/sec). Surfaces of the gravel stratum in the abutments were carefully cleaned and this filter material was placed adjacent the gravel. The filter was carried along the interior of the downstream training walls and terminated at a concrete encased mass of cobbles similar to a gabion. A minimum cross-sectional area equal to 75 square feet (7 square meters) drainage area was provided. It is estimated that a gradient of less than 1/100 would develop in the filter system under maximum reservoir head. construction of the embankment began in the latter part of June 1985 and, except for downstream slope paving, was complete on September 29, 1985.

SERVICE HISTORY OF REPLACEMENT DAM

On October 19, 1985 after the embankment, cut-off walls and abutment pavement were complete but before the slope paving above the downstream training walls was complete, the Guadalupe River experienced a flood which overtopped the spillway by 13.4 feet (4.1M). No damage was experienced by the embankment structure. Parts of the downstream face of the drilled shaft cut-off walls were exposed but there was no evidence of through-seepage. Partially completed downstream filter zones and some of the concrete paving above the training walls were washed away. After the floodwaters receded no unusual seepages were observed that could indicate that the dam and cut-off were not functioning satisfactorily. The minor damages were corrected, filter materials were replaced and slope paving above the downstream training walls were completed.

On July 17, 1987 the dam was overtopped to a depth of 17.2 feet (5.24M). This was reported to be the largest flood in the past 40 years in this reach of the Guadalupe River. At the dam the flood reached elevation 1638.2 msl or 3.2 feet (1.0M) above the paved abutment sections. No damage was experienced by the embankment nor its downstream training walls. The only damage suffered by any of the Upper Guadalupe River Authorities' facilities was erosion of soil above the slope paving on the north (left) abutment and the loss of a head-wall and one section storm drainage pipe downstream of the south (right) training wall. Photograph of the dam taken after the July 17, 1987 flood is shown in Figure 9.



Fig. 9 Completed Dam, July 1987

CONCLUSION

It is concluded that the accident was caused principally by inadequate cut-off and filter design and construction and substandard hydrostatic relief systems in the downstream portion of the dam and abutments.

Roller Compacted Concrete proved to be an excellent choice for the replacement. Although the work-in-progress was overtopped by river flows several times during construction, loss of work-in-progress was minimal and clean-up and restart of construction seldom required more than one day.

By using roller-compacted concrete to rehabilitate the facility it was possible to maintain a pool of water sufficient to satisfy the City of Kerrville's needs without rationing the use of water.

Design strengths for the RCC mix were achieved and exceeded.

The continuous flow of water plus several large floods have not produced any scour or deterioration of the compacted concrete mass.

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