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MIX DESIGN INVESTIGATION — ROLLER COMPACTED CONCRETE CONSTRUCTION, UPPER STILLWATER DAM, UTAH

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16. ABSTRACT

RCC (roller compacted concrete) is concrete with zero slump consistency, which is placed in horizontal lifts and consolidated in place with vibratory rollers. It has the same properties as conventional concrete. The research in RCC for Upper Stillwater Dam involved two phases: a laboratory mix program to evaluate different RCC mixes and a test placement at the jobsite followed by an extensive coring program. The laboratory mix design program was initiated to develop and fully evaluate three mixes which were used at the test placement. The three mixes evaluated differed primarily in the percentage of minus 200 material in the sand. Properties studied included compressive and tensile strength, elastic properties, shear strength, thermal properties, permeability, shrinkage, and durability. The test placement evaluated field performance of the mixes and the use of slip-form paving machines with laser control to extrude curbing for the upstream and downstream faces of the dam.

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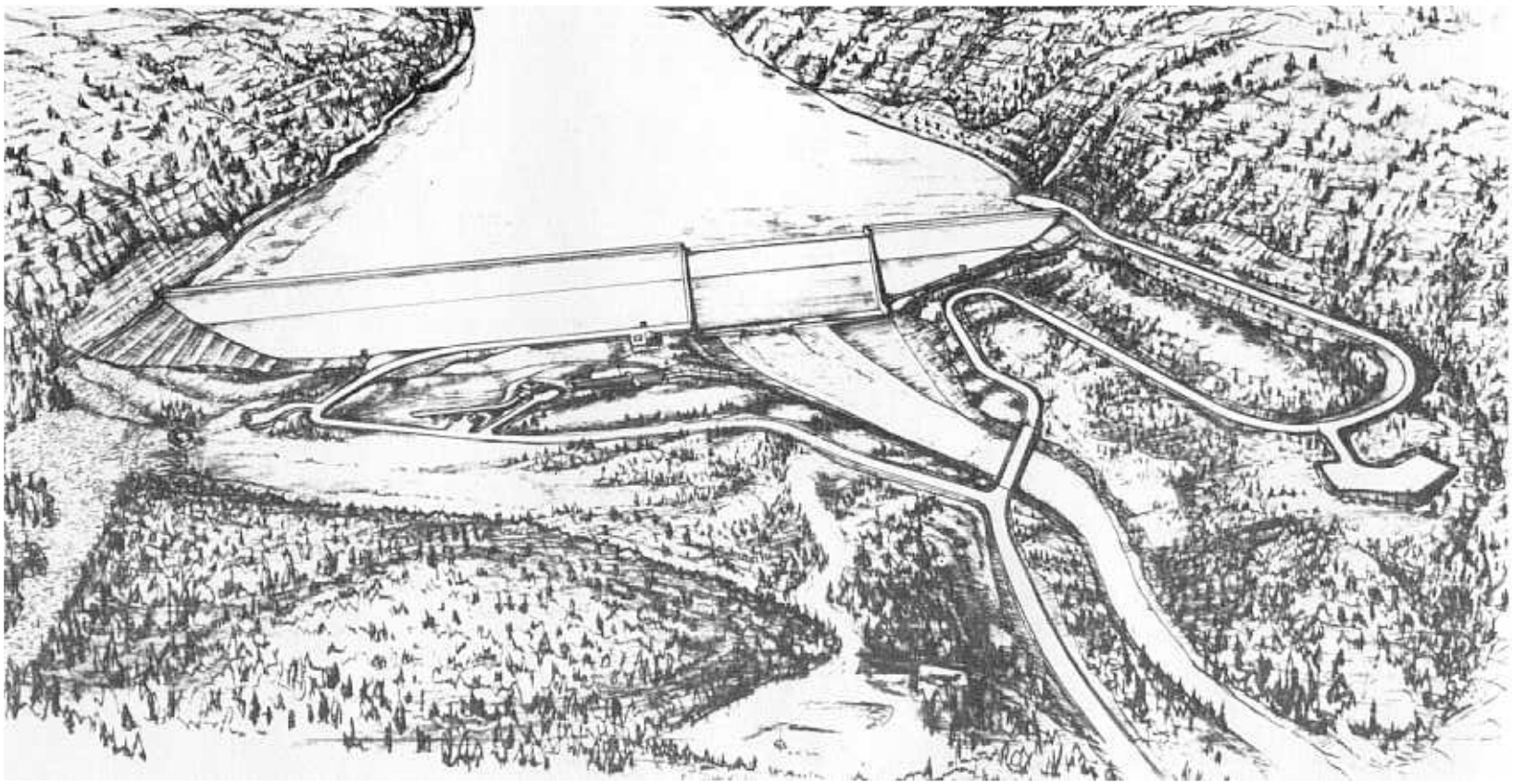

Frontispiece – Artist's conception of Upper Stillwater Dam, Utah. P801-D-80872.

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INTRODUCTION

Upper Stillwater Dam will be the first concrete dam constructed by the Bureau of Reclamation utilizing the RCC (roller compacted concrete) construction technique. RCC is concrete with zero slump consistency, which is deposited in horizontal lifts and consolidated in place by a smooth drum vibrating roller. This method of construction was first proposed in the early 1970's [1, 2, 3]* and was demonstrated successfully in emergency spillway repairs at Tarbela Dam, Pakistan [3], in the construction of Shimajigawa Dam, Japan, and Willow Creek Dam, Oregon.

An extensive RCC mix design investigation was begun in 1980. This report summarizes the mix design program conducted in the laboratories of the Concrete and Structural Branch at the E&R (Engineering and Research) Center and discusses the field RCC test placement constructed at the Upper Stillwater damsite in 1981.

DESIGN CONCEPTS – UPPER STILLWATER DAM

Upper Stillwater Dam will be a gravity dam containing 1.4 million yd^3 of RCC, located approximately 120 miles southeast of Salt Lake City, Utah, in the Uinta Mountains at an elevation of 8000 feet. The dam will be approximately 275 feet high with a 2,600-foot crest and a vertical upstream face; the downstream face will have a 0.6:1.0 slope. Storage capacity of the reservoir will be 30,000 acre-feet, and the dam will have a spillway capacity of 15,000 ft^3/s.

Selecting the Type of Dam for Upper Stillwater

The Upper Stillwater site was first investigated as a potential damsite in the late 1940's. Since that time, the site has been considered for various types of dams including an earth embankment, a concrete-faced rockfill, and three concrete dams – a straight concrete gravity dam, a massive head buttress dam, and a straight RCC gravity dam. Feasibility studies, completed in late 1979, compared the rockfill with the RCC alternative and showed the RCC dam to be the lower cost alternative.

Foundation

Upper Stillwater damsite is located on Rock Creek which flows southward down the south flank of the Uinta Mountains. Rock Creek and glacial action have created a wide, flat valley with steep abutments. The dam will be founded on medium- to massive-bedded quartzose sandstone in the valley floor. Foundation rock in both abutments consists of sandstone, argillite, and siltstone. Surficial deposits covering the foundation rock consist of alluvial fan, talus, and glacial deposits. These surficial deposits range in thickness from 10 to over 100 feet. Several steeply dipping narrow faults cross the foundation running in a north-south direction.

Foundation Treatment

Foundation preparation for placing the concrete dam will require removal of the 1,000,000 yd^3 surficial deposits and excavation of 300,000 yd^3 of the sandstone foundation. Consolidation grouting will be used over much of the foundation to close numerous small joints and tighten up the foundation. Larger shear zones will be excavated and backfilled with dental treatment concrete.

A seepage cutoff curtain will be constructed the full length of the dam by grouting a line of holes from the foundation gallery. Foundation uplift pressures will be relieved by a line of drainage holes drilled downstream from the grout curtain. These drainage holes will also be drilled from the gallery.

Leveling concrete, which consists of internally consolidated low slump concrete, will be placed between the RCC and the foundation rock. This concrete serves several functions, the most important of which is to provide bond between the RCC and the irregular rock foundation, thus controlling seepage and providing sliding stability. Leveling concrete also serves to facilitate placement of the first RCC lifts. In the lowest point of the foundation, referred to as the trough, leveling concrete 15 feet thick will be placed to provide a 100-foot minimum RCC lift length parallel to the dam axis. In this way, placing lifts perpendicular to the dam axis will be avoided which could contribute to seepage through the dam. On nearly level areas of the foundation, leveling concrete will be placed in 50- by 50- by 2-foot-thick blocks. These blocks will initiate the horizontal RCC lifts across the slope and will be repeated on each succeeding RCC lift to eliminate any need for tapering or featheredging lifts.

Both the upstream and downstream faces will be constructed by extruding concrete using a conventional slip-form paver and a side-hung mold. Each pass of the paver will add 2 feet of facing. Following the placement of the facing elements, two successive 1-foot lifts of RCC will be placed as interior concrete and against the facing. It is estimated that this process will add approximately 2 feet of height to the dam per day. RCC will be mixed by either conventional weigh batching or by a continuous mixing operation. Transportation will be by truck or conveyor, and in-place rolling of the 1-foot lifts will be

* Numbers in brackets refer to entries in the Bibliography.

with conventional smooth drum vibratory rollers. This process will be repeated until the dam is completed.

Extruded slip-formed faces provide a form system and a durable concrete face to resist freeze-thaw action. They will not function as a water barrier. As thermal adjustments take place within the dam, cracks will develop in the faces. Therefore, the RCC is designed to be impervious, and the faces will be allowed to crack.

Spillway

A spillway crest length of 600 feet and a design head of 3.5 feet were selected to take advantage of the RCC extruded facing construction method. The long crest length eliminated the need for gates and required only 3.5 feet of surcharge to pass the 15,000 ft^3/s peak of the inflow design flood. Steps added to the extruded facing elements in the spillway chute dissipate 70 percent of the hydraulic energy and require a spillway basin only 30 feet long, or one-sixth of that originally proposed. Spillway chute steps also eliminated possible problems with cavitation common with high-velocity flow.

Outlet Works

The outlet works will release water at a rate of 285 ft^3/s into Stillwater tunnel or up to 220 ft^3/s into Rock Creek. An intake tower in the reservoir is connected to the control valves by a 6-foot-diameter conduit set in the foundation beneath the dam. The entire system is separated from the dam to minimize interference with placement of the RCC. Rock Creek discharges will be released from a submerged 14-inch-diameter jet-flow valve discharging up to 30 ft^3/s into an enclosed compact stilling basin.

Dam

The original design of the dam featured a 15-foot top width and a 0.6:1.0 downstream slope, minimizing excavation and concrete volume. The top width was increased to 30 feet to facilitate construction access of the upper portion; however, the downstream slope was not changed. The dam was analyzed for normal static and dynamic loads using current Bureau criteria for conventional concrete gravity dams. In addition, extensive analyses were conducted to model the construction problems unique to RCC construction. A computer model was developed to compute the thermal gradients produced by heat generation of the RCC and ambient temperatures. These gradients compared well with the 1 year of temperature data obtained from the test section. Stresses within the dam were determined for the thermal gradients for various times up to 5 years. Extensive two- and three-dimensional finite element studies were used to determine stresses.

CONCEPTS OF RCC MIX DESIGN

RCC mixes have been developed using three mix design philosophies, largely determined by specific conditions existing at each damsite. These mix design concepts include:

- The use of "as-dug" aggregate with minimal to no processing, blended with cement, such as that used at Tarbela Dam [5]
- Lean concrete (a dry, lean concrete with low cement content), such as that used at Shimajigawa Dam in Japan and Willow Creek in Oregon
- High fly ash content concrete, such as that proposed for Upper Stillwater Dam in Utah and the Milton Brook Dam in the United Kingdom.

For the above structures, the MSA (maximum-sized aggregate) ranged from ¾-inch to 9 inches, and the cementitious material content ranged from 88 pounds of cement up to 420 pounds of cement and fly ash. Fly-ash contents ranged from 0 to 75 percent of cementitious materials by volume.

Processing of aggregates used in RCC has ranged from minimal or no processing to a fully processed ASTM graded aggregate. The excavated material used at Tarbella, for instance, was a pit-run skip-graded material mixed by rock ladders. Aggregates for Willow Creek Dam were reasonably well-graded material with minus No. 200 sieve-size material varying from 4 to 10 percent of the total materials. Proposed coarse aggregate for Upper Stillwater Dam will be reasonably well-graded from 2-inch to No. 4 material.

In the mix design for an RCC dam, several requirements must be taken into consideration: (1) durability of the material, (2) compressive and tensile strength requirements of the concrete, and (3) required workability. Also, both foundation and seismic safety considerations must be satisfied. Requirements for RCC in Upper Stillwater Dam include the following:

1. Full consolidation of the RCC must be maintained to prevent seepage.
2. Bond between the lifts must be achieved.
3. Concrete must be able to withstand early and long-term thermal movement without jeopardizing the structural stability of the dam.

4. Concrete must have good workability so maximum density can be achieved with vibratory rollers.

5. Concrete must not segregate.

6. The design of the dam must accommodate the poor freeze-thaw durability of RCC by either providing protective air-entrained facing concrete or by overbuilding dam faces to provide sacrificial concrete in anticipation of raveling due to freeze-thaw damage.

RCC is much the same as conventional concrete, and design of RCC dams must be approached the same as any other concrete dam. The properties required for a particular structure govern the mix design: for instance, the purpose of the reservoir (storage, flood control, or both) or the existence of freeze-thaw conditions.

A two-phase program was developed for the evaluation of materials for Upper Stillwater Dam using RCC: (1) a complete mix design study was performed in the laboratory, and (2) a test placement was made using the concepts of construction with RCC and laser guided slip-form pavers. The laboratory mix program evaluated three differently proportioned mixes. The strength development properties investigated were compressive, direct tensile, and shear strength; other properties included thermal expansion, adiabatic temperature rise, diffusivity, creep, drying shrinkage, and permeability.

Upon completion of the test placement, cores were extracted at 28, 90, 180, and 360 days. The test data from the cores were compared to specimens cast in the laboratory mix program.

CONCLUSIONS

1. Concrete from RCC mixes L-1, L-2, and L-3 all met a compressive strength requirement of 3,000 lb/in^2 and tensile strength requirement of 180 lb/in^2 at 1year's age.

2. The use of pit-run materials with a high silt content in the RCC test placement resulted in a reduction in strength and increase in drying shrinkage due to a probable increase in water content. Though acceptable concrete properties can be obtained using partial replacement with pit-run materials, the variations in silt content of available materials preclude their use for Upper Stillwater Dam.

3. Mix L-3 meets all the requirements set forth in the design criteria with the fewest problems in quality control. Mix L-3 also showed greater tensile strength potential than either mix L-1 or L-2, whose strengths apparently were reduced by clumps of silt and clay in the pit-run sand.

4. Creep and thermal property results were as expected, generally in the range of ordinary concrete using a quartzose sandstone for coarse aggregate.

5. Temperature rise is lower than for ordinary concrete and is due to the high fly-ash content and low initial placing temperature of approximately 50 °F.

6. Test results of bonding of the joints of the test section were not as good as anticipated; however, bonding of the horizontal joints can be obtained by control of lift thickness to ±1 inch, adequate compaction, and adjustment of the mortar content for variation in voids of the coarse aggregate. Testing for the voids ratio in accordance with ASTM Designation: C-29 should be a part of the quality control program. Finally, horizontal joints must be clean and maintained in a moist condition.

7. The use of slip-form pavers using laser control of both horizontal and vertical alinement for constructing the upstream and downstream facing elements of the dam is acceptable.

8. The low modulus in the RCC mixes may be attributed to the high mortar content of the 1½-inch MSA as compared to a 4-inch MSA mass concrete, the low modulus of the coarse aggregates influenced by either the bedding planes of the aggregate (quartzose sandstone), or microcracking at the interface of the mortar and coarse aggregate.

9. Additional research is needed in several areas:

a. Methods to entrain air to increase the durability of RCC under freeze-thaw conditions

b. Methods of accelerated strength testing for quality control of high fly-ash concretes

c. Methods to detect voids at horizontal lift lines in construction

d. Investigation of factors influencing temperature rise of high fly-ash concretes, in particular at early ages (1 to 3 days)

e. Factors influencing joint bond potential including time, temperature, and paste content

MATERIALS INVESTIGATIONS

The materials investigations were based on two differing philosophies for the mix design of RCC: (1) unprocessed, "pit-run" materials from the foundation excavation, and (2) processed graded materials produced the same as for conventional concrete. Three mix designs were developed, the first containing 100 percent unprocessed materials, the second, a combination of both processed and pit-run sand, and the third, incorporating 100 percent clean, graded sand.

The coarse aggregate for the laboratory testing program was a crushed sandstone obtained from talus slopes at the jobsite. The sandstone is marginal in quality with specific gravity ranging from 2.49 to 2.54, absorption of 1.7 percent, and LA abrasion loss (at 500 revolutions) of 65.1 percent. Pertinent data on the sandstone coarse aggregate are given in table 1.

Three sand sources were used for the laboratory testing program; pit-run sand from the damsite, crusher fines generated from the aggregate crushing operation, and clean processed sand obtained from a local Denver aggregate source. Pertinent data on these sands are given in table 2.

For the test placement, the coarse aggregate was obtained by crushing talus material at the jobsite. The properties of the coarse aggregate used for RCC and facing concrete are given in table 3. This coarse aggregate is very similar in physical properties to the aggregate used in the laboratory mix program, with the exception of the grading. A high percentage of aggregate in the ⅜- to ¾-inch size caused a 3 to 4 percent increase in the voids ratio content of the aggregate. This led to an increase in segregation and a decrease in the compactability of the RCC, particularly in mixes T-2 and T-3. The voids content of aggregate used in RCC should be monitored closely to ensure that there is sufficient mortar in the mix.

Table 1. – Laboratory mix program – coarse aggregate physical properties – RCC.

Sieve designation[1]	Coarse aggregate grading (individual percent retained)	
	Mix L-1	Mix L-2, L-3
1½ in	12	3
¾ in	56	52
⅜ in	21	39
No. 4	11	4
Pan	0	2
Specific gravity	2.54	2.54
Absorption	1.67	1.67
Voids content	38.35	38.35

[1] U.S.A. Alternate Sieve Designations ASTM: E-11

Three sand sources were also used in the test placement mixes. Pit-run sand was similar in quality to the sand used in the laboratory test program. Talus sand was obtained from the aggregate crushing operation; however, the processing did not remove the considerable amount of overburden which led to contamination with silty fines. The clean, graded sand was obtained from a commercial source near Salt Lake City. Pertinent data on these sands are given in table 4.

Because of variations in the quality of the available pit-run sand, a clean, graded sand was required. This led to the use of higher fly-ash contents in the mix design to supply the necessary fines for adequate consolidation and reduction in segregation of the RCC.

Laboratory standard cement and fly ash were used for the three laboratory mix designs. Physical and chemical properties of cement and fly ash are given in table 5. For the test placement, type II, low-alkali cement was obtained from a commercial producer. Class F pozzolan (fly ash) was supplied from a powerplant located in western Wyoming. Physical and chemical properties of cement and fly ash are given in table 6.

PRELIMINARY MIX DESIGN AND MORTAR STUDIES

The initial RCC mix design trials were begun in April 1980. Preliminary mix design investigations were performed in accordance with procedures outlined in ACI (American Concrete Institute) Committee 207 report on RCC. These mix designs generally proved unacceptable because of the poor workability of the mixes.

In an effort to reduce the time for preliminary tests, a series of mortar tests was conducted to establish basic strength and workability parameters of mortar. After developing an "optimal mortar" design, the percentage of coarse aggregate for RCC designs was optimized for the most workable mix.

Six series of mortar mixes were tested to investigate the following mix design parameters:

- $\frac{FA}{C}$ ratio (fly ash:cement)

Table 2. – Laboratory mix program – fine aggregate physical properties – RCC.

Sieve designation No.	Fine aggregate grading (individual percent retained)					
	Individual grading			Combined grading		
	PRS[1]	TS[2]	CC[3]	Mix L-1 40 percent PRS 60 percent TS	Mix L-2 25 percent PRS 75 percent CC	Mix L-3 100 percent CC
4	3.0	9.0	2.0	6.6	2.3	2.0
8	6.7	12.0	2.0	9.9	3.2	2.0
16	12.3	6.0	12.0	8.5	12.1	12.0
30	14.3	10.0	31.0	11.7	26.8	31.0
50	18.5	35.0	34.0	28.4	30.0	34.0
100	17.3	23.0	14.0	20.7	14.8	14.0
200	6.8	3.8	5.0	5.0	5.5	5.0
–200	21.1	1.2	–	9.2	5.3	–
Specific gravity	2.40	2.62	2.65			
Absorption	5.78	0.81	0.84			
Voids content	35.97	35.25	32.69			

[1] PRS – Pit-run sand – Upper Stillwater damsite.
[2] TS – Crushed talus sand – Upper Stillwater damsite.
[3] CC – Clear Creek sand – Denver laboratory standard.

- $\frac{W}{C + FA}$ ratio (water:cement plus fly ash)
- $\frac{C + FA}{sand}$ ratio (cement plus fly ash:sand)
- Sand content (various blends of processed sand, pit-run sand, and crusher fines)

Flow of mortar and compressive strength testing of 2-inch cubes were performed in accordance with ASTM: C 109-80 (Compressive Strength of Hydraulic Cement Mortars). The results of these tests are shown in table 7 and on figure 1. Five trial mixes were made with a class F fly ash, and one trial was conducted using a class C fly ash. The concept of these mortar mixes was to develop a stiff mortar (as compared to conventional mortar) with the required strength parameters, while utilizing as high a percentage of fly ash as possible in an effort to minimize heat generation.

Mortar design mix No. 52, which used a pit-run sand content of 40 percent plus 60 percent crusher fines and fly ash to cement ratio of 60:40, was selected for development of laboratory mix No. L-1. Following the construction of the test placement, additional mortar tests were conducted using a clean processed sand (mixes No. 53 to 58) which was used in laboratory mix No. L-3.

Upon completing the mortar studies, a series of trial mixes was conducted to evaluate the strength and workability of RCC mixes at varying percentages of No. 4 to 1½-inch MSA. The 1½-inch MSA was an early decision based upon the problems of segregation as reported by others and has since been changed to 2-inch MSA. In addition, test procedures for mixing and testing RCC and methods of casting test specimens were evaluated. The surcharge weight for the Vebe test was increased to 50 pounds. Test cylinders were cast on the Vebe table in three lifts with a 20-pound surcharge. Based on workability, compression, and tension tests, a coarse aggregate content of 55 percent (by volume) was established as the optimal aggregate percentage. This mix design was then selected for complete physical properties testing as mix L-1.

FINAL MIX DESIGN STUDIES

Development of Laboratory Mix Designs

Three mix designs were developed for complete physical properties testing in the laboratory. Tests

Table 3. – Upper Stillwater Dam concrete test placement – coarse aggregate physical properties – individual grading (percent retained).

RCC

Sieve designation	Mix T-1 (Lifts 1-3)	Mix T-2 (Lifts 4-7)	Mix T-3 (Lifts 8-11)
	(individual percent retained)		
1¾ inch	0.0	0.0	0.0
1½ inch	2.3	1.5	1.2
¾ inch	49.9	40.1	41.4
⅜ inch	33.9	43.5	43.8
No. 4	9.7	8.9	11.8
Pan	3.7	6.0	1.8
Specific gravity		2.49	
Absorption		2.8 percent	

Facing concrete

Sieve designation	Mix T-4 and T-5
	(individual percent retained)
1 inch	0.0
½ inch	63.7
No. 4	33.1
No. 8	1.9
Pan	1.3
Specific gravity	2.49
Absorption	2.8

and test ages are shown in table 8. Though there were some adjustments in these mix designs due to variations in aggregate gradations and workability requirements, the primary mix design parameters were held constant. The three mix designs used in the laboratory studies are shown in table 9.

As discussed in the section "Preliminary Mix Design and Mortar Studies," mix L-1 was developed from a series of mortar studies conducted earlier in the testing program. The state-of-the-art with respect to RCC mix design suggested that pit-run or "as-dug" materials could be utilized in the RCC mixes. Mix L-1 incorporated the use of pit-run sand, combined with "crusher fines" generated by the processing of coarse aggregate. The quantity of coarse aggregate was determined by substituting different percentages of aggregate with mortar until a workable mix was found.

Mix L-2 was developed to reduce the total heat generated by the RCC. The adiabatic temperature rise of mix design L-1 was considered too high for thermal design considerations. In this mix design, approximately 60 pounds of cement were replaced with fly ash, which increased the FA:C ratio to 75:25. This reduced the total amount of heat evolved and the rate of heat evolution. Because of variations in the quality of pit-run sand used at the damsite, a change in sand source was required. Mix L-2 incorporated 75 percent clean, processed sand and only 25 percent unprocessed, pit-run sand. The 25 percent pit-run sand provided the fines required for optimal compaction.

Mix L-3 is based upon the mix design procedures developed during the Milton Brook Dam studies in the United Kingdom. This mix design was based upon the use of 100 percent clean sand with the fines required for optimal compaction supplied by the addition of fly ash. This mix design was deemed necessary because of the wide variation in quality of pit-run sand in mixes used at the Upper Stillwater Dam site. Though it is possible to design an RCC mix with pit-run materials, the variation in quality as well as the percentage of fines in this material would make quality control extremely difficult.

Two additional mix designs were investigated to a limited extent in the laboratory. Mixes L-4 and L-5 were designed with a higher paste content to provide additional joint bonding capabilities if the need arises during a construction delay or shutdown. These mix designs are shown in table 9.

Mixing and Testing Procedures

The RCC was mixed in the laboratory in a 9-ft^3 drum mixer. For mix L-1, aggregates and 75 percent of the water were added and mixed for 1 minute. This was necessary to break up lumps of pit-run sand. Following this mixing, cementitious materials and the remaining water were added and mixed for 4 additional minutes. Mixing procedures were altered for mixes L-2 and L-3 with the addition of all materials and enough water to bring the workability to a range of 35 to 45 seconds as determined by the Vebe test.

Fresh Concrete

Workability and density of the fresh concrete were determined with the Vebe vibrating table apparatus. Tests were conducted to determine workability, expressed as the time required to consolidate 28 pounds of RCC in a cylinder clamped to the vibrating table. Tests were performed with and without a 50-pound surcharge weight. The Vebe time was expressed as the time required for paste to migrate

Table 4. – Upper Stillwater Dam concrete test placement – fine aggregate physical properties – individual grading (percent retained).

Sieve designation No.	Pit-run sand			Talus sand			Clean sand		
	Fine	Coarse	Average	Fine	Coarse	Average	Fine	Coarse	Average
	(individual percent retained)								
4	0.0	22.7	9.6	9.0	37.2	19.5	4.6	5.0	4.8
8	4.1	5.0	6.0	7.2	17.3	10.0	9.7	10.8	10.2
16	14.1	13.0	16.2	2.9	7.7	6.8	8.0	10.8	9.4
30	15.0	24.3	18.7	16.3	3.8	7.1	17.5	15.7	16.6
50	15.8	18.6	17.2	7.1	4.4	15.9	31.2	31.4	31.3
100	16.2	11.0	16.1	25.2	22.0	25.2	21.9	22.1	22.0
200	6.4	2.9	5.3	9.6	3.7	6.3	5.2	2.3	3.8
−200	28.4	2.5	10.9	22.7	3.9	9.2	1.9	1.9	1.9
FM	1.70	3.34	2.54	1.90	3.83	2.73	2.45	2.59	2.52
Specific gravity	-	-	2.44	-	-	2.63	-	-	2.58
Absorption	-	-	5.70	-	-	2.30	-	-	0.80

Combined grading

Sieve designation No.	Mix 1 40 percent PRS[1] 60 percent TS[2]	Mix 2 25 percent TS 75 percent CS[3]	Mix 3 100 percent CS
	(individual percent retained)		
4	15.5	8.5	4.8
8	8.4	10.2	10.2
16	10.6	8.8	9.4
30	11.7	14.2	16.6
	16.4	27.4	31.3
100	21.6	22.8	22.0
200	5.9	4.4	3.8
−100	15.8	8.1	5.7
−200	9.9	3.7	1.9

[1] PRS – Pitrun sand
[2] TS – Talus sand
[3] CS – Clean sand

completely around the surcharge or around the periphery of the cylinder (similar to the Cannon test). [3 (pg. 17-18)] The drier mix L-1 had a Vebe time of approximately 50 to 60 seconds with the surcharge weight. The wetter mixes L-2 and L-3 had Vebe times of approximately 25 to 35 seconds with the surcharge weight and 35 to 45 seconds without the surcharge weight. Because this test involves some estimation as to when the paste fills all the voids under the surcharge or around the rim of the cylinder, the variation in Vebe time is relatively high (standard deviation of 5 to 20 seconds). The presence of small rock pockets within the sample further complicates the test. Research needs to be performed on test

Table 5. – Laboratory mix program – physical and chemical properties of cement and pozzolan – RCC.

Physical properties		Chemical analysis		Percent
Type II, LA cement – M-7120				
Loss on ignition (percent)	1.3	Silica	SiO_2	21.6
Specific surface (m^2/kg)	392	Alumina	Al_2O_3	4.8
Specific gravity	3.16	Iron	Fe_2O_3	3.9
Compressive strength:		Calcium	CaO	62.3
3 days (lb/in^2)	2,340	Magnesium	MgO	3.0
7 days (lb/in^2)	3,700	Total sulfates	SO_3	2.7
Initial set (h:min)	3:30	Sodium	Na_2O	.3
Final set (h:min)	5:25	Potassium	K_2O	.4
		Insoluble residue		.8
Class F – Pozzolan – M-6498				
Loss on ignition (percent)	2.0	Silica	SiO_2	46.1
Fineness (percent retained	-	Alumina	Al_2O_3	19.0
on No. 325 sieve)	2.44	Iron	Fe_2O_3	18.6
Specific gravity	−0.004	Magnesium	MgO	1.3
Autoclave expansion (percent)	96	Total sulfates	SO_3	1.6
Water requirement (percent)		Moisture content		.03
Compressive strength:				
28 days with cement (percent)	82			
7-day lime pozzolan (lb/in^2)	960			

Table 6. – Upper Stillwater Dam concrete test placement – physical and chemical properties of cement and pozzolan.

Physical properties		Chemical analysis		Percent
Type II, LA cement				
Loss on ignition (percent)	0.8	Silica	SiO_2	22.1
Specific surface (m^2/kg)	420	Alumina	Al_2O_3	3.7
Specific gravity	3.20	Iron	Fe_2O_3	3.0
Compressive strength:		Calcium	CaO	63.4
3 days (lb/in^2)	3,660	Magnesium	MgO	2.6
7 days (lb/in^2)	4,710	Total sulfates	SO_3	2.9
Initial set (h:min)	3:20	Sodium	Na_2O	.3
Final set (h:min)	5:05	Potassium	K_2O	.4
		Insoluble residue		.4
Class F – Pozzolan				
Loss on ignition (percent)	0.3	Silica	SiO_2	
Fineness (percent retained	25	Alumina	Al_2O_3	84.0
on No. 325 sieve)	2.34-2.41	Iron	Fe_2O_3	
Specific gravity	−0.02	Magnesium	MgO	2.9
Autoclave expansion (percent)	91	Total sulfates	SO_3	.6
Water requirement (percent)		Moisture content		.0
Compressive strength:				
28 days with cement (percent)	93			
7-day lime pozzolan (lb/in^2)	1,250			

Table 7. – Laboratory mix program – mix design summaries – mortar mixes – RCC.

Mix	Cement/ pozzolan ratio	Cement weight, g	Fly ash weight, g	W/C+P	Cement-pozzolan/ sand ratio	Sand weight, g	Flow, mm	Compressive strength, lb/in²				
								7-day	28-day	90-day	180-day	365-day
51	40:60	289	335	0.47	1:275	1677	60	1218	2010	3148		
52	40:60	289	335	.47	1:275	1687	82	1276	2118	3307		
53	30:70	150*	259	.47	1:275	1186	105	1059	1982	3191	4371	5615
54	30:70	150*	259	.43	1:275	1186	84	1395	2565	3977	4617	6219
55	25:75	125*	278	.47	1:275	1168	129	921	1666	2881	3694	5222
56	25:75	125*	278	.43	1:275	1168	113	1100	1975	3142	4069	5468
57	20:80	100*	298	.47	1:275	1153	128	586	1145	2095	3134	3851
58	20:80	100*	298	.43	1:275	1153	112	767	1314	2530	3394	4442

* Clear Creek sand, ½ batches

Note: Mix 51 made with a blend of 60 percent pit-run sand and 40 percent crushed talus sand.
Mix 52 made with a blend of 40 percent pit-run sand and 60 percent crushed talus sand.

methods for RCC to reduce the visual error and subjectivity of this test.

The density of fresh concrete was determined using the weight of the vibrated sample and its calculated volume. The volume of the sample vibrated was determined by filling the Vebe cylinder above the sample with water and subtracting the volume of the water from the known volume of the cylinder. When the surcharge weight was used with the Vebe test, the density of fresh concrete was determined immediately after completion of the test. When the Vebe test was used without the surcharge, the sample was vibrated following the initial test up to a total time of 2 minutes prior to determining the density. A summary of the properties of fresh concrete is shown in table 10.

Laboratory test specimens (6 by 12 inches) were cast on the Vebe table. Steel cylinder molds were

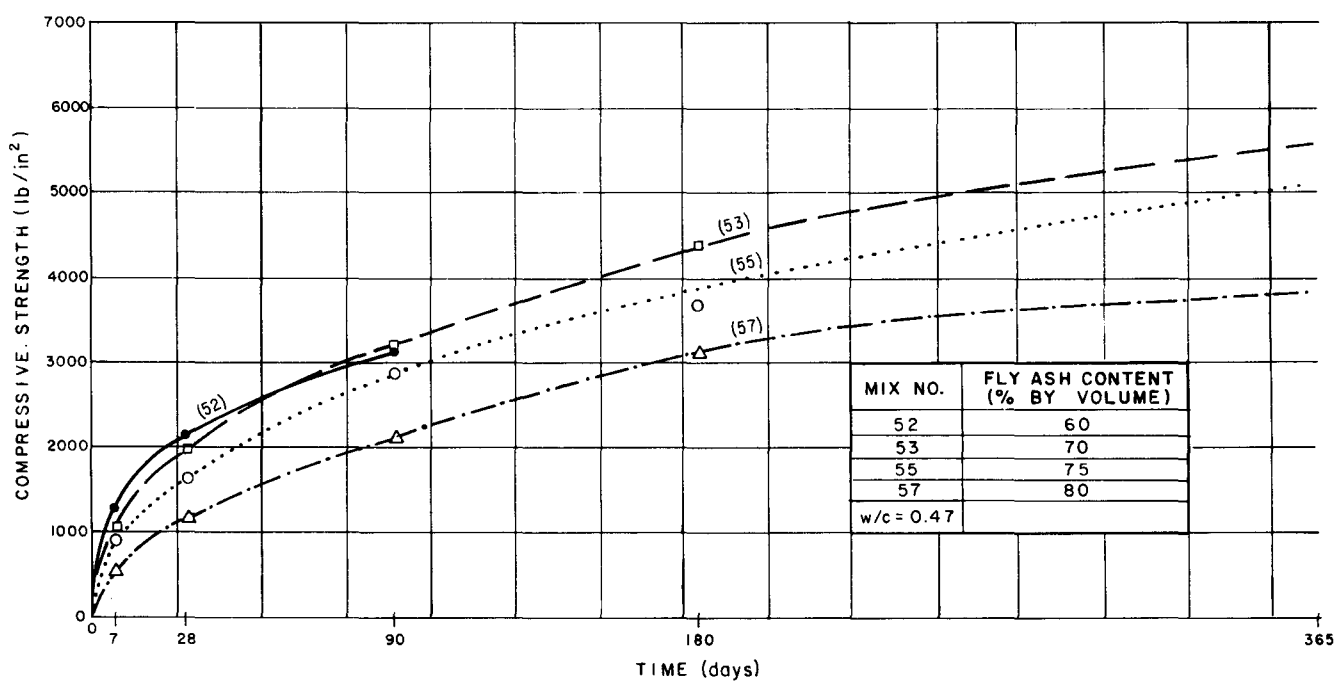


Figure 1. – Laboratory mix program, compressive strength of mortar, RCC.

Table 8. – Laboratory mix program – tests performed – RCC.

Test	Test age (if applicable)						Design criteria
	2	7	28	90	180	365	
	days						
Static uniaxial compressive strength	X	X	X	X	X	*	3,000 lb/in²
Modulus of elasticity and Poisson's ratio (compression)		X	X	X	X	X	-
Static uniaxial tensile strength			X	X	X	*	180 lb/in²
Modulus of elasticity and Poisson's ratio (tension)			X	X	X	X	-
Uniaxial creep in compression			X	X	X	X	-
Static cohesion and coefficient of internal friction			X	X	X	*	300 lb/in², 0.5
Coefficient of thermal expansion			X				-
Adiabatic temperature rise (50 °F)			X				-
Diffusivity			X				-
Density		X	X	X	X	*	145 lb/ft³
Permeability			X	X			-
Freeze-thaw resistance			X				25 percent maximum weight loss at 500 cycles
Drying shrinkage							-

* Design criteria age.

Table 9. – Laboratory mix program – mix design summaries – RCC.

Mix designation	L-1		L-2		L-3		L-4		L-5	
	Weight, lb	Volume, ft³	Weight, lb	Volume, ft³	Weight, lb	Volume, ft³	Weight, lb	Volume, ft³	Weight, lb	Volume, ft³
Air (1 percent)	-	0.27	-	0.27	-	0.27	-	0.27	-	0.27
Water (W)	183.0	2.96	175.0	2.82	180.0	2.89	186.0	2.98	194.0	3.11
Cement (C)	182.0	.93	121.0	.61	129.0	.67	132.0	.67	156.0	.61
Fly ash (FA)	210.0	1.38	269.0	1.77	286.0	1.88	295.0	1.94	344.0	2.26
Fine aggregate:										
Pit run	421.0	2.71	286.0	1.85	-	-	-	-	-	-
Crushed talus	661.0	4.05	-	-	-	-	-	-	-	-
Clean – Clear Creek	-	-	867.0	5.25	1,134.0	6.87	1,171.0	7.09	1,101.0	6.67
Coarse aggregate: No. 4 to 1½ inch	2,324.0	14.70	2,283.0	14.43	2,285.0	14.44	2,223.0	14.05	2,200.0	13.90
Total	3,981.0	27.00	4,001.0	27.00	4,014.0	27.00	4,007.0	27.00	3,995.0	27.00
$\frac{W}{C + FA}$ (percent weight)	0.47		0.45		0.43		0.44		0.39	
FA (percent C+FA volume)	59.7		74.4		73.7		74.3		74.1	
Theoretical density	147.40		148.19		148.67		148.41		147.96	

rigidly clamped to the vibrating table. Test cylinders were cast in three equal lifts under a 20-pound surcharge weight. Paste filled around the edge of the surcharge before the next lift was placed. Jointed test specimens were fabricated by placing and compacting one-half of the cylinder, and placing the top half following a specified cure (generally 24 hours). After the test program, a new method of casting 6- by 12-inch cylinders was developed. A standard 6- by 12-inch plastic mold was set inside the steel cylinder and clamped tightly, but without deforming the plastic mold. The RCC was then vibrated in place as before, and the entire specimen in the mold was removed. This method is advantageous over casting in the steel molds because of the time savings involved.

Hardened Concrete

A summary of tests performed on RCC is shown in table 8. Unless otherwise noted, tests were performed on specimens fog-cured at 73 °F.

Compressive strength testing was performed on 6- by 12-inch cylinders in accordance with ASTM Designation: C 39-80 (Compressive Strength of Cylindrical Concrete Specimens). Modulus of elasticity and Poisson's ratio in compression were initially determined with an extensometer-compressometer frame at early ages (28 days or less). Automation of testing apparatus made it possible to test stress versus strain to failure with epoxied strain gages. During the process of epoxying strain gages, test specimens were kept moist by wrapping them in plastic. The modulus of elasticity was determined in a stress range between 100 and 1,000 lb/in^2 unless excessive strain was experienced (generally early age tests), in which case the upper value was reduced to 600 lb/in^2. Direct tensile strength testing was performed on 6- by 12-inch cylinders epoxied to steel end plates. Tests were performed on both jointed and unjointed specimens. Only one failure was attributed to failure across the joint; thus, joint efficiency would be considered close to 100 percent. In some cases, bond failure occurred in the epoxy at the end plate. These failures, when not occurring in the test specimen, are not included in average tensile strength values reported herein. The modulus of elasticity and Poisson's ratio in tension were computed from stress versus strain readings recorded from epoxied strain gages. These values were calculated over a stress range from 50 to 150 lb/in^2 unless excessive strain was recorded, in which case the upper stress limit was reduced to 100 lb/in^2.

Creep parameters were determined from tests of sealed concrete specimens under constant compressive load. Test specimens were cast in copper sleeves, with gage point inserts embedded in the concrete. Specimens were fog-cured, lapped plane, and sealed prior to testing. Two specimens were loaded at each test age, with one companion specimen retained without load under the same test conditions. Axial strain measurements were taken at third points around the circumference of the cylinders prior to loading, immediately after loading, and at regular intervals thereafter. Measurements were taken with a fulcrum plate strain gage. The applied load was less than 20 percent of the ultimate strength of the concrete at each test age. The magnitudes of the creep parameters $1/E$ and $F(K)$ are based upon the empirical logarithmic function

$$\varepsilon = 1/E + F(K)\ln(t+1)$$

where:

ε = elastic plus creep strain per lb/in^2
E = modulus of elasticity at loading age K, lb/in^2
$F(K)$ = creep parameter, a constant for any particular loading age K
$\ln(t+1)$ = natural logarithm of the time after loading plus one.

These values can be determined from a semilogarithmic plot of time plus one as the abscissa and elastic plus creep strain as the ordinate on the linear scale. The intersection of this line with the coordinate $(t+1) = 1$ gives the value of $1/E$, and the $F(K)$ is derived from the slope of the straight line curve.

Static cohesion and coefficient of internal friction were determined from shear break-bond and sliding friction tests of jointed concrete specimens. The intact specimen was subjected to a biaxial stress until shear failure occurred across the horizontal joint. The failure plane across the joint is predetermined by casting the hardened concrete specimen into separate holding rings approximately one-fourth of an inch apart. Normal loads are applied to the specimen at stresses varying from 50 to 100 lb/in^2, and the shear load is applied to the lower ring until failure occurs across the joint. Following the break-bond test, sliding friction tests are performed on the open joint at normal stresses varying from 50 to 300 lb/in^2. Cohesion and coefficient of internal friction are determined by plotting the normal and shear stresses for individual break-bond tests. Similar values are determined for the results of sliding friction tests.

Diffusivity of RCC was determined from time-temperature readings of 6- by 12-inch specimens brought to a constant temperature and then immersed into a high-temperature water bath. The difference between the interior and exterior surface of the test specimen is plotted on a logarithmic scale versus time on a linear scale, and the diffusivity is

Table 10. – Laboratory mix program – properties of fresh concrete – RCC.

Mix No.	Vebe test					
	With surcharge			Without surcharge		
	Time, s	Unit weight, lb/ft³	No. of tests	Time, s	Unit weight, lb/ft³	No. of tests
L-1	61.0	147.1	22	-	-	-
*σ	18.8	.6		-	-	-
**CV (percent)	30.6	.4		-	-	-
L-2	33.0	146.6	13	36.2	147.1	9
σ	5.3	.7		11.1	1.4	
CV (percent)	16.1	.5		30.5	.9	
L-3	32.4	147.6	15	40.7	147.7	15
σ	5.9	.7		13.9	.6	
CV (percent)	18.2	.5		34.3	.4	

* σ – Standard deviation
** CV – Coefficient of variation

computed when the rate of change of temperature is constant with time. The interior temperature was determined with a thermocouple embedded in the center of the cylinder.

The temperature rise of mass concrete was determined over a 28-day period in an adiabatic calorimeter room on 650-pound specimens sealed in a 21.5- by 21.5-inch metal container in accordance with USBR standard laboratory procedures. Throughout the test, the room temperature was maintained at the same temperature of the concrete to ensure the adiabatic condition. The initial temperature of the concrete was determined immediately after mixing. Generally, a 2 to 4 °F increase in temperature was recorded between mixing and final sealing of the test container due to initial hydration and energy imparted to the concrete during consolidation. The initial placing temperature ranged from 45 to 58 °F.

Coefficient of thermal expansion of hardened concrete was determined by direct measurement of a series of prisms over a temperature range from 35 to 90 °F. Test specimens were saw cut from cylinders into 2- by 2- by 4-inch lengths. The cumulative length change of three prisms was recorded to avoid the error incorporated with the small measurements. Tests were conducted in the saturated moisture condition for all mixes and in the oven-dry and 75 percent moisture condition for mix L-1. Because the local Denver area sand was used for mixes L-2 and L-3, the values are lower than mix L-1 and lower than what would be expected for the dam. This was confirmed by tests of RCC when project sand was used in mix L-3.

The density of hardened concrete in the saturated, surface-dry moisture condition was determined from its weight and displaced volume when immersed in water.

Permeability of hardened concrete was determined from samples of RCC subjected to a hydrostatic pressure of 400 lb/in². The rate of flow through the specimen was measured over a 1-month period.

Freeze-thaw resistance was determined on 3- by 6-inch cylinders of RCC wet-screened to ¾-inch MSA. Following a 28-day fog cure, samples were subjected to alternating cycles of freezing and thawing until a 25-percent weight loss was recorded.

Drying shrinkage was determined from length change measurements of RCC fog cured specimens which were then placed in laboratory air maintained at 73 °F and 50 percent humidity.

RESULTS OF LABORATORY MIX DESIGNS

Compressive Strength

The results of compressive strength are given in table 11. As shown on figure 2, the strength and rate of

Table 11. – Laboratory mix program – average strength and elastic properties summary – RCC.

Mix No.[1]	Age days	Compressive strength, lb/in²	Compressive strength, No. of tests	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in	Tensile strength, lb/in²	Tensile strength, No. of tests	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in
L-1	2	520	3	-	-	-	-	-	-
L-1	7	1,360	25	-	-	-	-	-	-
L-1	28	2,130	26	1.03	0.13	110	5	-	-
L-1	90	3,510	8	1.32	.14	150	5	-	-
L-1	180	4,720	3	1.58	.15	-	-	-	-
L-1	365	5,220	7	1.71	.17	205	5	1.10	0.13
L-2	2	220	3	-	-	-	-	-	-
L-2	7	770	10	-	-	50	4	-	-
L-2	28	1,220	14	.82	.13	80	4	-	-
L-2	90	2,150	18	-	-	130	5	-	-
L-2	180	3,240	6	1.26	.17	135	5	1.28	.08
L-2	365	4,780	6	1.59	.20	200	3	1.43	.14
L-3	2	350	3	-	-	-	-	-	-
L-3	7	1,110	8	-	-	55	4	-	-
L-3	28	1,620	15	.92	.13	110	4	-	-
L-3	[2]28	2,030	3	1.49	.13	-	-	-	-
L-3	90	2,770	17	-	-	130	5	-	-
L-3	180	4,025	4	1.69	.17	185	3	1.19	.13
L-3	365	4,960	6	1.76	.18	220	5	1.60	.19

[1] All mixes using a set-retarding WRA.
[2] Using a nonset-retarding WRA and different sand source.

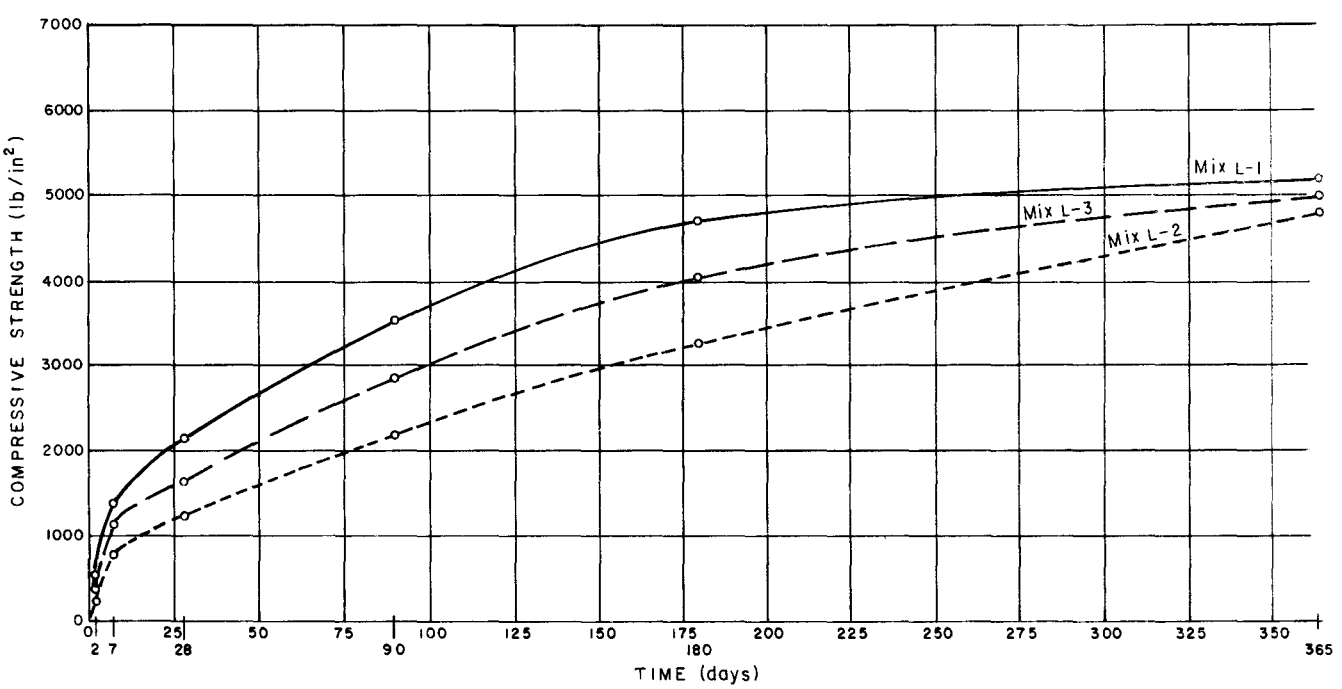


Figure 2. – Laboratory mix program, compressive strength development, RCC.

strength development are influenced by the FA:C ratio and the W:C+FA ratio. Through 1 year's age, the effect of the FA:C ratio outweighs the effect of the W:C+FA ratio. Mix L-1, with a water-cement ratio greater than both mixes L-2 and L-3 (0.47 versus 0.45 and 0.43, respectively), has a higher compressive strength through 1 year. This can be attributed to the higher percentage of cement in mix L-1 than in mixes L-2 and L-3 (40 percent by volume versus 25 percent). It is anticipated that the compressive strength of mixes L-2 and L-3 may exceed the strength of mix L-1 at ages beyond 1 year's age. Mix L-2 has a higher rate of strength gain after 180 days' age than mix L-3, which was not anticipated with equal FA:C ratios; however, this may be attributed to within batch variations of individual mix designs.

The strength development of all mix designs beyond 28 days' age is greater than expected, with 28-day strengths varying from 25 to 40 percent of the 1-year strengths. This is directly related to the high percentages of fly ash and may pose a problem for the early age quality control testing of field cylinders at the damsite.

An accelerated strength test method needs to be developed to provide a quicker means of determining the ultimate strength potential of the mixes. The strength development at early ages may be influenced by the use of a set-retarding WRA (water reducing admixture) in the mix, particularly during the first 3 days. When combined with the fly-ash contents, the strength development at early ages can be retarded. This may be advantageous for better joint strength development; however, the disadvantage would be the trapping of heat in the mass which may have been able to escape prior to the placement of the next lift of RCC.

Modulus of Elasticity and Poisson's Ratio in Compression

Perhaps the most significant deviation from properties of conventional concrete lies with the elastic properties of Upper Stillwater RCC mixes. As shown in table 12, the modulus of elasticity in compression is significantly lower than that of conventional concrete at equal strength levels. Figure 3 shows the variation in compressive strength and modulus of elasticity for other Bureau mass concrete dams. The RCC test results indicate moduli ranging from 25 to 40 percent of the modulus of conventional concrete at equal strengths. There are three possible explanations for this phenomenon: lower modulus of the aggregate, lower modulus of the mortar (or paste),

Table 12. – Laboratory mix program – creep function parameters – RCC.

Mix No.	Age at load days	Compressive strength[2], lb/in²	Modulus of elasticity, lb/in² x 10^6	Refer to creep curves[1]		
				1/E, 10^{-6}/(lb/in²)	$F(K)$	Modulus of elasticity[3], lb/in² x 10^6
L-1	28	2,150	[4]1.03	1.05	0.106	0.95
L-1	90	3,410	[4]1.32	.84	.057	1.19
L-1	180	4,120	[4]1.58	.67	.032	1.49
L-1	365	4,990	[2]1.75	.57	.018	1.75
L-2	180	3,220	1.26	.62	.019	1.61
L-2	365	4,870	[2]1.63	.57	.013	1.75
L-3	28	[5]2,030	[5]1.49	.66	.044	1.52
L-3	180	4,170	1.69	.57	.013	1.75
L-3	365	5,140	[2]1.82	.53	.021	1.89

[1] Creep equation $\varepsilon = 1/E + F(K) \ln (t + 1)$
ε = elastic + creep strain in millionths inches per inch per lb/in²
E = modulus of elasticity at loading age K
K = age of concrete when load is initially applied
$F(K)$ = a constant for any loading age representing the rate of creep deformation with time
t = age in days after loading.
[2] Values computed by companion 6- x 12-in cylinder.
[3] Values from creep specimens.
[4] Values taken from average of all cylinders of mix at that age.
[5] Different sand source used in mix.

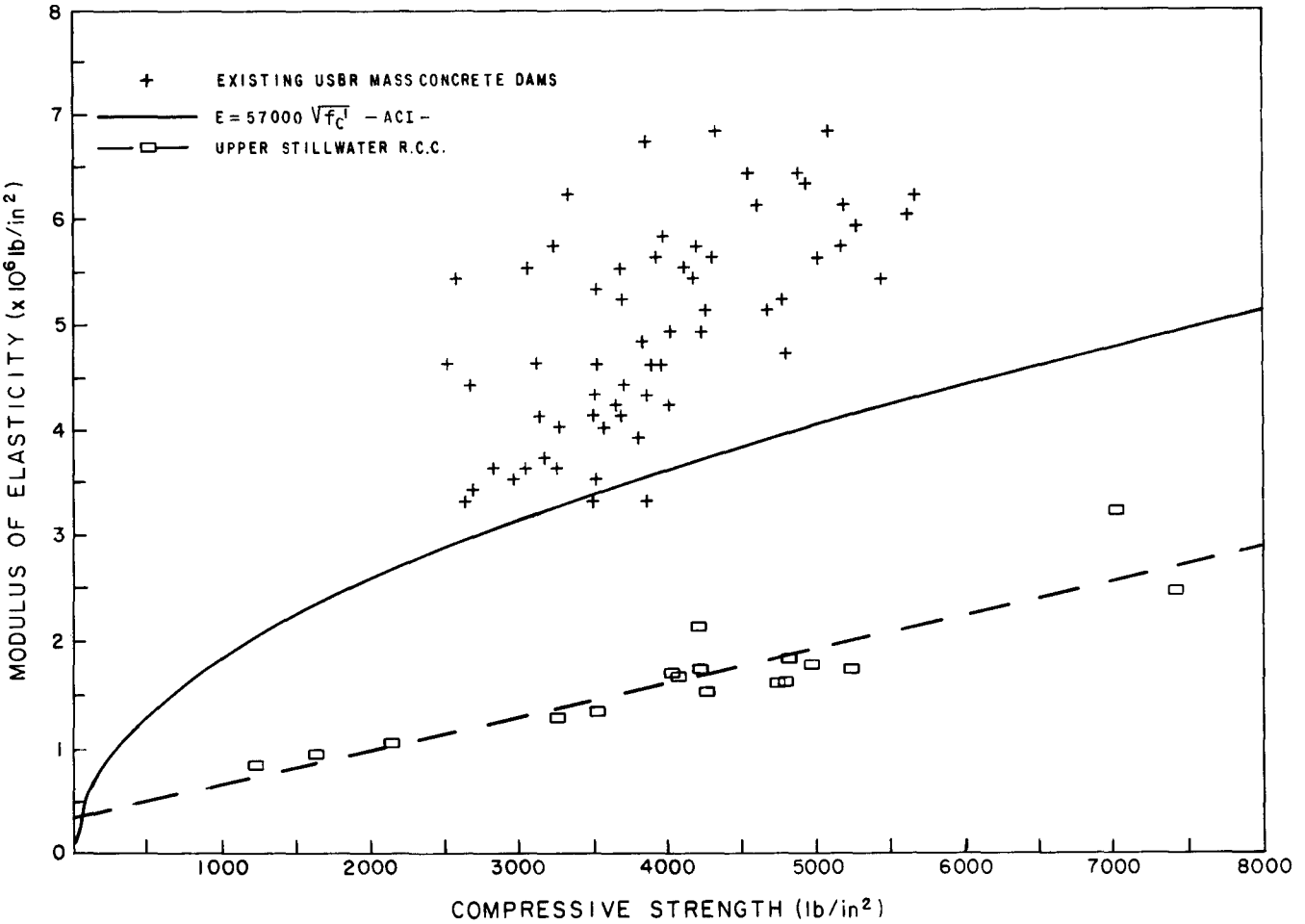


Figure 3. – Laboratory mix program, elastic properties of concrete versus compressive strength, RCC.

or microcracking in the paste aggregate interface. According to Neville [6]:

> "The two components of concrete, cement paste and aggregate, when individually subjected to stress, exhibit a sensibly linear stress-strain relation. The reason for the curved relation in the composite material – concrete lies in the presence of interfaces between the cement paste and the aggregate and in the development of microcracking at those interfaces."

As shown on figure 4, the typical stress – strain relationship of RCC in compression shows higher strain values at equal stress than for conventional concrete. As stated before, three possible causes are the aggregate, paste, or their interface. Since the aggregate used in the mixes is in itself a "cemented" material, being a quartzose sandstone, there is a possibility that the aggregate may experience higher strain and thus have a lower modulus of elasticity. This could, therefore, induce higher strain in the RCC stress – strain relationship or lower its modulus of elasticity or both.

The effect of bedding planes in the coarse aggregate could also contribute to lowering the elastic modulus.

Figure 4 shows two tests of cores of foundation sandstone similar in physical properties to the coarse aggregate. When the load was applied perpendicular (90°) to the bedding planes, the strength of the sandstone exceeded 20,000 lb/in². When load was applied at a 60° angle from the bedding plane, the strength was reduced to less than 7,000 lb/in², and the specimen exhibited more than triple the strain. Though this type of bedding plane would not normally be found in a crushed coarse aggregate, there may be some influence of the bedding of the sandstone in aggregate particles which could lower the modulus of the concrete.

The high fly-ash content of the paste could be responsible for the high strain and lower modulus. The low strength of the paste could also result in a low

modulus, which would be in turn lower the RCC modulus. Studies are not available which show the influence of high fly-ash contents on modulus of either paste or concrete. The paste-aggregate interface may also be responsible for the lower modulus of elasticity and high strain of the stress – strain relationship. Following Neville's hypothesis [6], microcracking at the paste-aggregate interface causes local stress intensity, increasing the magnitude of the strain, thus causing strain to increase faster than the applied stress resulting in the stress-strain "curve." The sandstone used for these mix designs contained individual sand grains which could be "rubbed off" the aggregate surface. This "rubbing off" could also be occurring during the application of stress, causing microcracking.

The low modulus of the aggregate will probably be revealed as the factor influencing the low modulus of the RCC. This is not considered detrimental to the RCC. Rather, high-strength, low-modulus concrete would probably be considered a benefit for this structure, which is subjected to significant differential temperature stresses during and after the cooling of the dam.

Poisson's ratio in compression compares with conventional concrete in the established range of 0.10 to 0.20 in/in, as shown in table 6. This should be expected, as the ratio of strain should not differ significantly with respect to lateral versus longitudinal deformation.

Tensile Strength

The results of tensile strength testing are shown in table 11 and on figure 5. As with compressive strength, the rate of tensile strength development after 28 days' age is significant, again attributed to the high percentages of fly ash. All these mix designs exceed the 180-lb/in^2 tensile strength design requirement, but not to the same degree as the compressive strength. Mix L-3 has the highest tensile strength at 1 year's age; however, mix L-1 and L-2 pit-run sand contained some small clumps of silt and clay which were evident in most breaks. This may be responsible for the fact that mix L-1 had a lower tensile strength expressed as a percent of the compressive strength. There was only one occurrence of joint bond failure throughout the test program, indicating that with a clean joint and proper consolidation adequate joint bonding can be achieved.

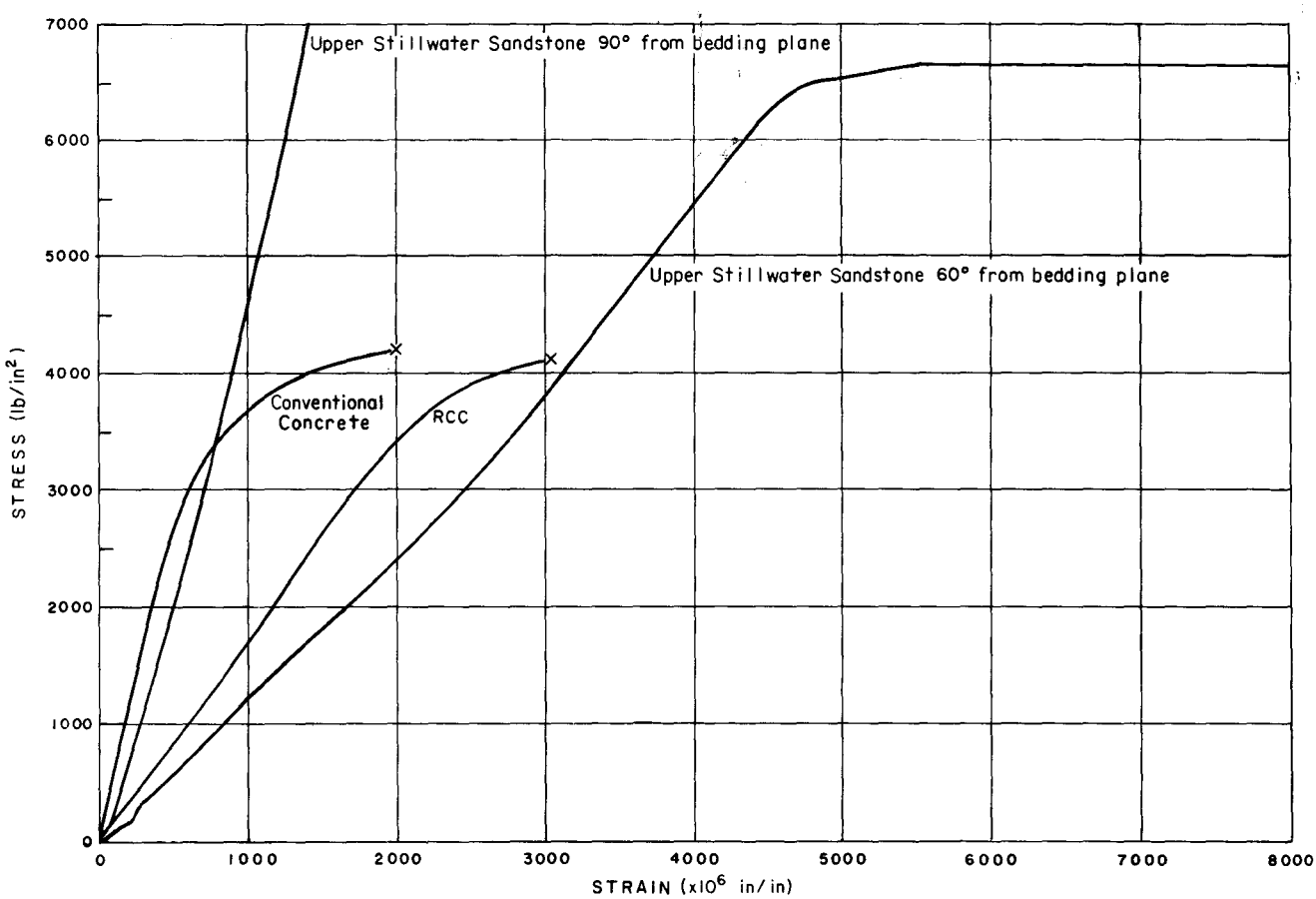


Figure 4. – Laboratory mix program, typical stress versus strain curves, RCC.

Modulus of Elasticity and Poisson's Ratio in Tension

Test results for modulus of elasticity and Poisson's ratio in tension are given in table 11. Though the number of tests is limited, the results do indicate that the modulus of elasticity is lower than what is normally found with conventional concrete. Again, as with compression, the stress-strain relationship indicates higher strain than is normally found in conventional concrete.

Creep

The creep test results are given in table 12. As shown on figures 6 through 8, the instantaneous deformation, $1/E$ (elastic plus creep strain), is higher than that of conventional concrete. This difference is related to the low modulus of elasticity of RCC. The rate of creep deformation, $F(K)$, is similar to that of conventional concrete at equal strength levels. Referring to figures 6 and 8, mix L-3 has a lower instantaneous deformation than mix L-1 at 28 days' age, which can be attributed to a higher modulus of elasticity for this particular mix (with Moon and White sand source). There is some inconsistency associated with the creep lines generated for mix L-3 at 180 and 365 days' loading age. This apparently is due to the inaccuracy of measurements of deformation. The test specimens loaded at 180 days had smaller deflections after loading and should be considered less accurate than the recorded values at 1 year, which shows excellent correlation of data points.

Cohesion and Coefficient of Internal Friction

Test results for cohesion and coefficient of internal friction are given in table 13 and on figures 9 through 19. All three mix designs exceed the design requirement of 300 lb/in² and 0.5 for cohesion and coefficient of internal friction, respectively.

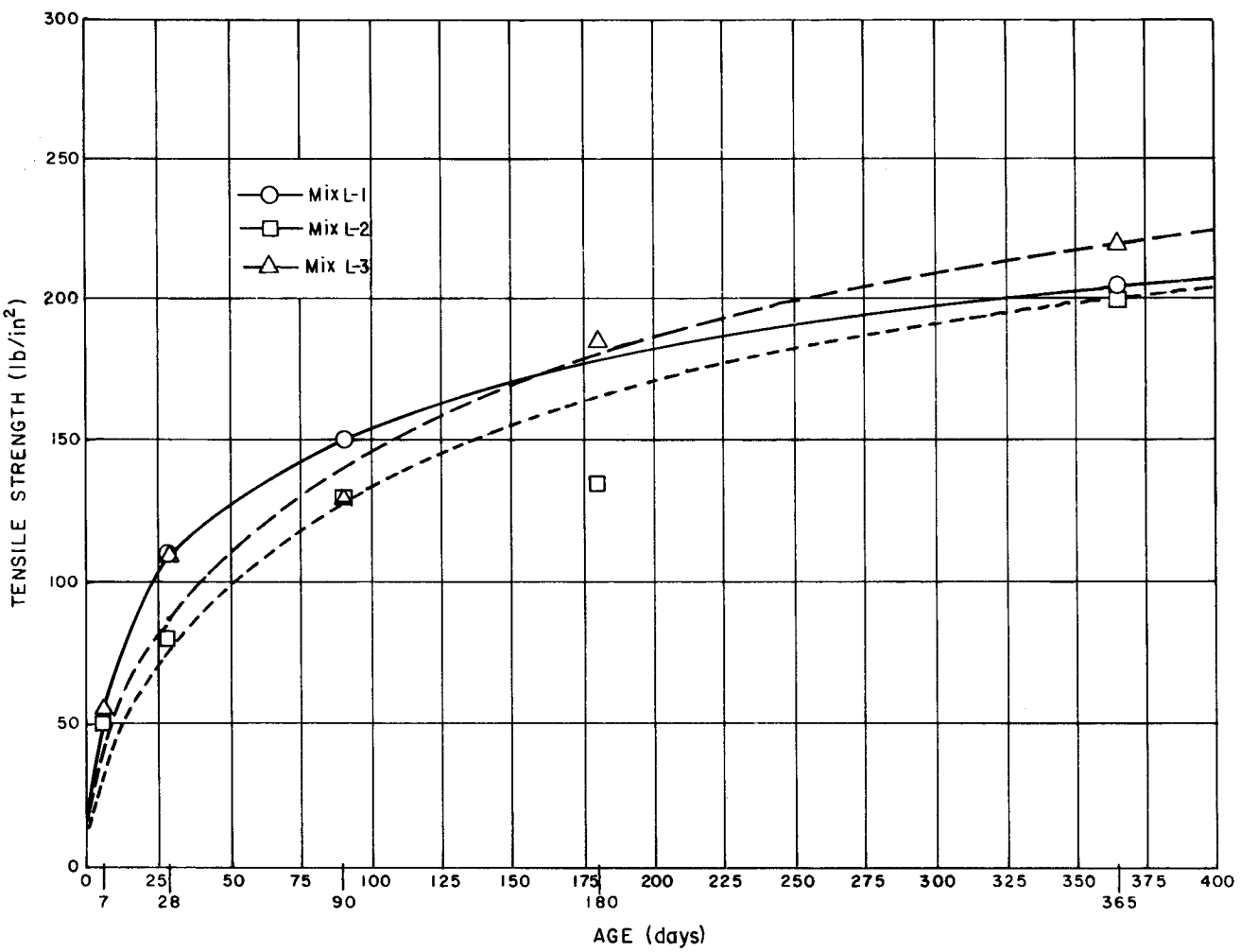


Figure 5. – Laboratory mix program, tensile strength development, RCC.

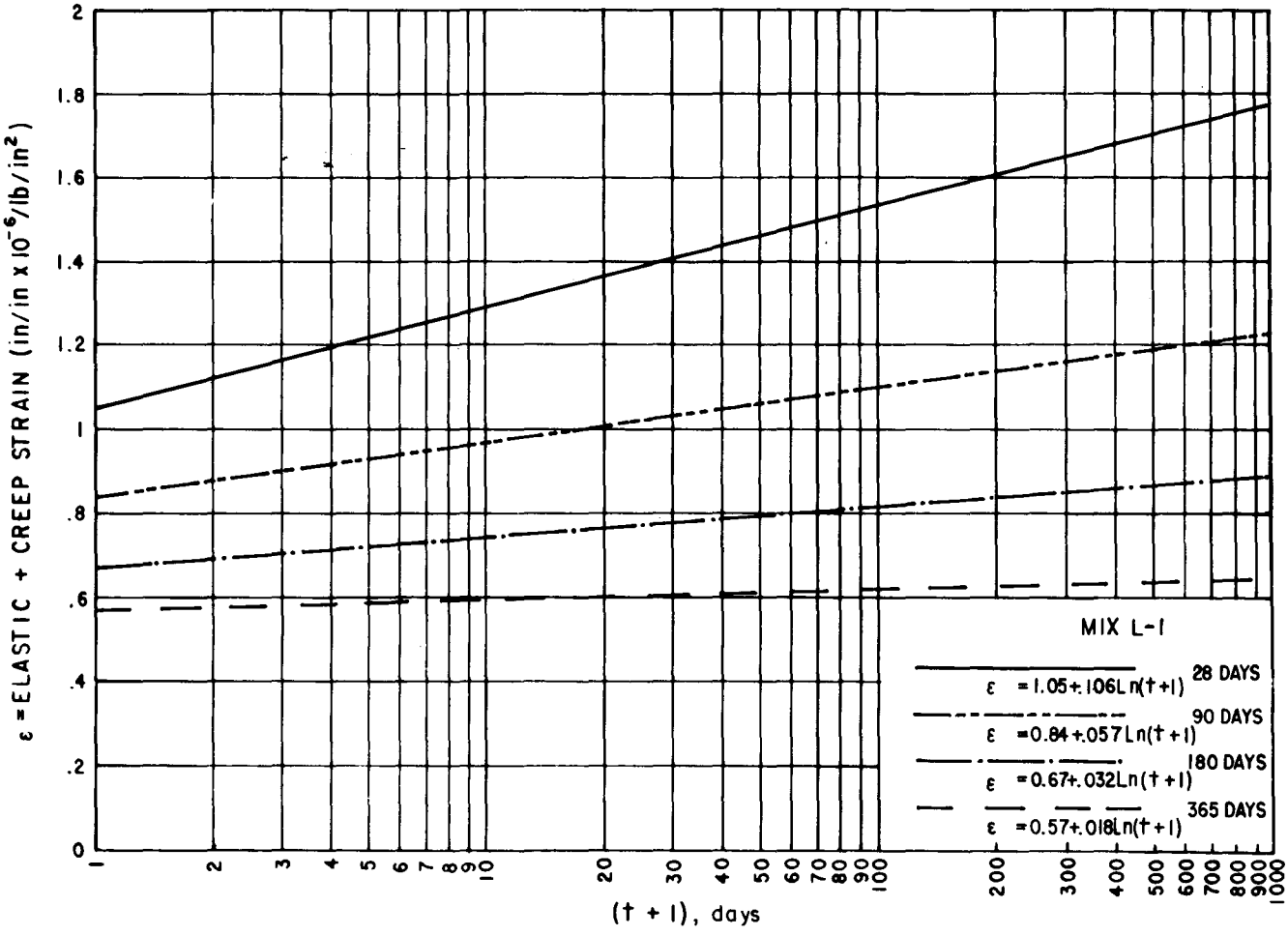


Figure 6. – Laboratory mix program, creep of mix L-1, RCC.

Coefficient of Thermal Expansion

Values for coefficient of thermal expansion are listed in table 14. Initial testing of mixes L-2 and L-3 was performed with concrete specimens which contained a local Denver area sand and which had lower coefficients of thermal expansion than was expected. When mix L-3 was retested with a project quartz-sand, the value increased from 3.20 to 4.96 x 10^{-6} °F. These values are similar to mix L-1 and within the range of quartzitic aggregates which have an above average thermal coefficient of expansion. Values agree with test results from samples obtained from the test section as shown in table 15.

Adiabatic Temperature Rise

Results of adiabatic temperature rise tests are shown in table 16 and on figures 20 through 23. These results indicate a substantial delay in heat generation during the 24 to 36 hours following placing. There are three probable causes for this delay or retardation: (1) the use of a set-retarding WRA, (2) the low initial placing temperature, and (3) the high percentage of fly ash. Tests were performed with and without the set-retarding WRA on laboratory mix L-3, as shown on figure 21. The delay in heat generation was reduced by approximately 12 hours; however, little temperature rise occurred during the first 24 hours. In previous temperature rise studies, it was found that when higher percentages of fly ash were combined with low placing temperatures and retarders, the total delay in setting time was greater than the sum of the delay attributed to each of the three components individually. This also appears to be the case occurring with the RCC. The total temperature rise at 28 days reflects the percentage of fly ash as well as the total cementitious materials content with mix L-1 having the highest total temperature rise, followed by mixes L-3 and L-2. This is because mix L-1 had approximately 50 pounds more cement than mix L-3. The temperature rise of mix L-1 at 45.5 °F in 28 days is relatively high for a mass concrete dam; however, with 1½-inch MSA, the total cementitious materials content must be increased above that for a conventional 3-inch MSA concrete. Temperature

rise for mixes L-2 and L-3 is low even for a mass concrete dam with a lower cementitious materials content.

Two additional temperature rise tests were performed, one on mix L-5, a richer mix proposed for use when delays in construction exceed 2 days, and one on the facing element mix design (FE-1). These results are shown in table 16 and on figures 22 and 23. These test results are consistent with results from earlier mixes with the exception of the facing element mix after 14 days. Here, a decrease in concrete temperature occurred rather than a continuing increase. The cause for this occurrence has not been determined.

Density

The results of density testing are given in tables 10 and 11. All mix designs meet the design density requirement of 145 lb/ft^3, which is somewhat lower than conventional, non-air-entrained concrete. This can be attributed to the low specific gravities of the coarse aggregate (2.54) and fly ash when compared to average specific gravities of aggregate and cement, respectively. The density of fresh concrete, determined by the Vebe test, compares favorably with the theoretical density, but is lower than the density of cast hardened concrete cylinder specimens. It appears that the densities of test cylinders exceed what would be achieved in the Vebe test, due to a possible over vibration of the sample.

Permeability

The results of permeability tests are given in table 17. These values are equal to or lower than those associated with conventional mass concrete. This can be attributed to the high fines content supplied by the fly ash and the low water to cementitious materials ratio. Failure of sealant material prevented the measurement of permeability for mix L-3. However, similar mixes were tested, and value of 4.0 x 10^{-4} ft/yr can be estimated. The standard test pro-

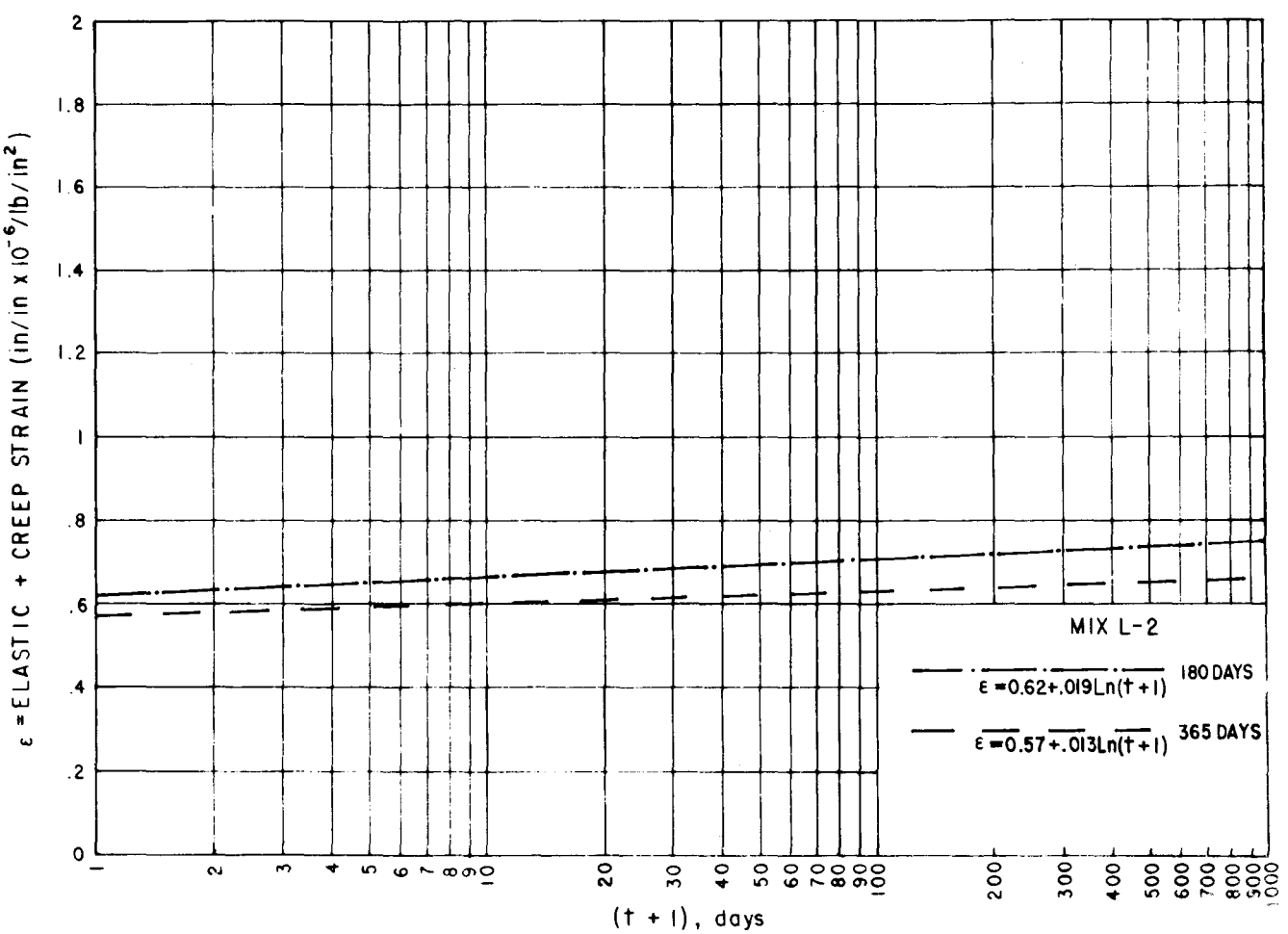


Figure 7. – Laboratory mix program, creep of mix L-2, RCC.

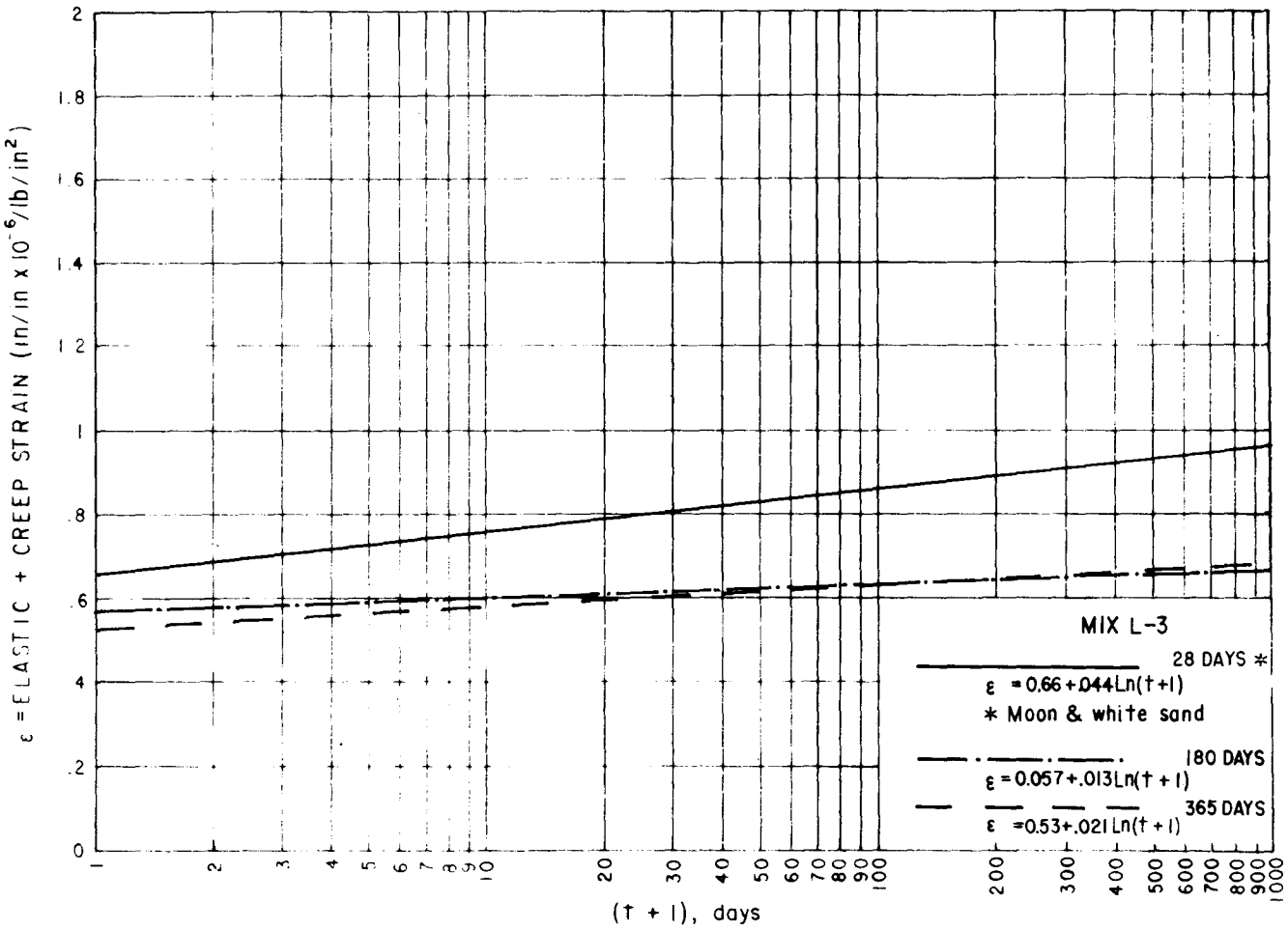


Figure 8. – Laboratory mix program, creep of mix L-3, RCC.

cedure for permeability involves correcting values to a standard test age of 60 days of curing. This was not useful because these procedures assume a higher degree of hydration of cementitious materials at that age than is achieved with RCC. The permeability of mix L-1, which was tested at 28 days' age, would probably be 25 to 50 percent lower if tested at a later age. Mixes L-2, L-4, and L-5 were tested when their compressive strengths were approximately 4,000 lb/in². These tests reflect a decrease in permeability at equal strengths with an increase in cementitious materials content, as shown on figure 24.

Diffusivity

The results of testing for diffusivity are shown in table 18 and on figures 25 through 27. These test results fall within the normal ranges of conventional concrete, with the values primarily governed by the aggregates. Mixes L-2 and L-3 incorporated the use of local Denver sand. Mix L-3 was retested with sand obtained from the Moon and White sand source.

Drying Shrinkage

Drying-shrinkage results are given in tables 19 and 20 and on figures 28 through 33. The average drying shrinkage values for laboratory mix designs L-1, L-2, and L-3 are higher than for conventional mass concrete which generally has a shrinkage of approximately 0.00500 inch after 1 year of drying. However, this should be expected with only 1½-inch MSA and relatively high mortar content. In addition, an aggregate with a low modulus of elasticity, such as the sandstone used at Upper Stillwater, offers less restraint to the potential shrinkage of the paste and also contributes to increased drying shrinkage. Drying shrinkage results from the test placement specimens are considerably different than from the laboratory specimens. Drying shrinkage from lift 3 (mix T-1)* is much higher than from mix L-1. This is probably due to a much higher water requirement

* For mix designation description, see "Concrete Mix Designs" section which follows.

Table 13. – Laboratory mix program – shear and sliding friction summary – RCC.

Mix. No.	Age, days	Break bond summary[1,2]			Sliding friction summary[1,3]		
		Φ_B	tan Φ_B	C_B, lb/in²	Φ_S	tan Φ_S	C_S, lb/in²
L-1	28	46.6	1.06	219.6	42.0	0.90	44.1
L-1	90	49.1	1.15	376.3	47.4	1.09	44.0
L-1	365	46.8	1.06	497.2	47.9	1.11	48.1
L-2	28	46.5	1.05	136.3	38.8	.80	38.1
L-2	90	58.0	1.60	240.6	45.0	1.00	38.5
L-2	180	54.9	1.42	350.1	43.9	.96	39.2
L-2	365	71.1	2.92	354.7	40.5	.85	31.9
L-3	28	42.1	.90	234.7	45.9	1.03	58.1
L-3	90	59.3	1.68	281.1	45.0	1.00	38.5
L-3	180	63.9	2.04	310.1	43.2	.94	44.9
L-3	365	45.6	1.02	575.4	42.9	.93	40.9

[1] General equation for shear: $\tau = C + \tan\Phi\,(\sigma)$
[2] $_B$ denotes break bond tests
[3] $_S$ denotes sliding friction tests

τ_B = shear strength (lb/in²)
τ_S = shear resistance (lb/in²)
C = cohesion (lb/in²)
Φ = friction angle (°)
tan Φ = coefficient of internal friction
σ = normal stress (lb/in²)

associated with 100 percent unprocessed pit-run sand for mix T-1 instead of 40 percent unprocessed pit-run sand which was in mix L-1 (see section "Materials Investigations"). Test reports indicate that the silt content of pit-run sand was as high as 34 percent minus No. 200 material. Lower strengths in field cylinders for mix T-1 support this conclusion. Drying shrinkages for lifts 6 and 8 (mixes T-2 and T-3, respectively) are lower than for laboratory mixes L-2 and L-3. This is probably the result of a lower water content, which is indicated in some batch scale weight readings and supported by higher strengths in field test cylinders. Most of the test specimens showed higher expansion after the continuous fog cure than what would be expected with conventional mass concrete. This could be due the result of the highly absorptive coarse aggregate and the higher paste and mortar content associated with the smaller MSA.

UPPER STILLWATER DAM – RCC TEST PLACEMENT

In August 1981, the RCC test placement was constructed at the proposed Upper Stillwater Dam site (see app. A). The 100-foot-long placement had a cross section similar to the upper 11 feet of the dam, with the downstream face similar to the spillway portion of the dam. The test placement was constructed to evaluate the construction procedures proposed for the dam, including the laser-guided slip forming of facing concrete and compaction of concrete by vibratory rollers (fig. 34). A coring program followed construction to determine additional properties of the RCC. Test cylinders were cast at the jobsite for additional testing.

The test placement (fig. 35) consisted of 11 lifts of RCC, 5 upstream and 5 downstream facing elements. The lifts of RCC are numbered 1 through 11, with 1 beginning at the lowest lift. Odd-numbered facing elements are upstream, and even-numbered ones are downstream.

Concrete Mix Designs

The three RCC mix designs used in the test placement, designated T-1 through T-3, are given in table 21. Though similar to the corresponding laboratory mix designs, two significant differences in the material properties were the high silt content of the sand that was obtained from the coarse aggregate crushing operation and an increase in voids content of the

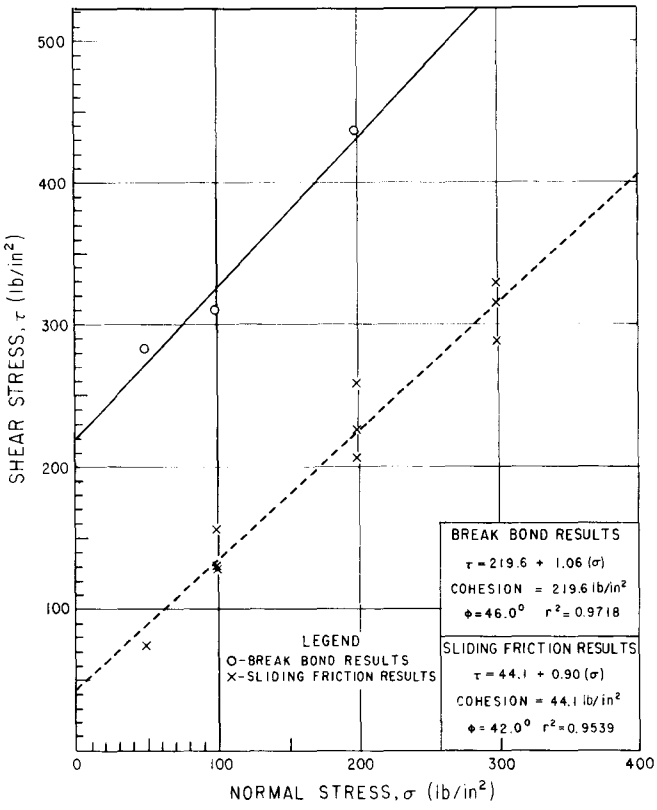


Figure 9. – Laboratory mix program, direct shear test mix L-1, 28 days, RCC.

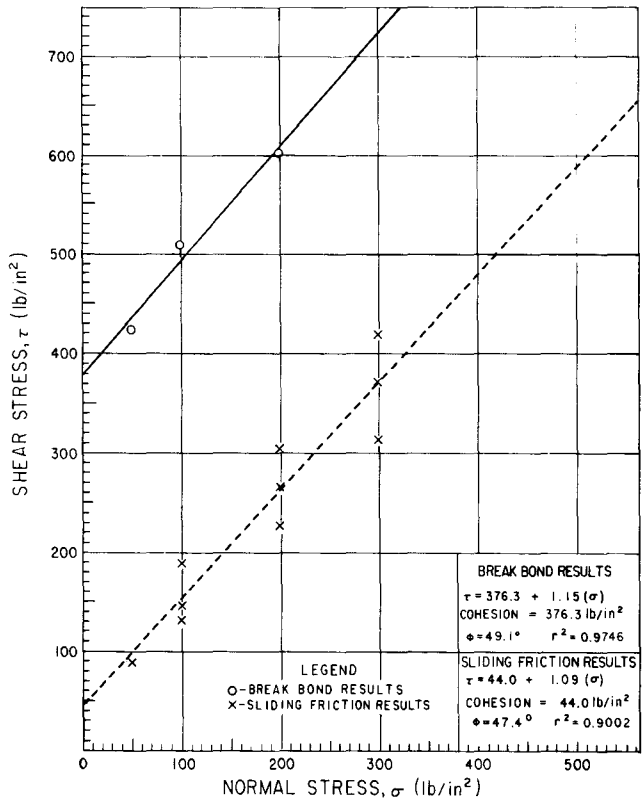


Figure 10. – Laboratory mix program, direct shear test mix L-1, 90 days, RCC.

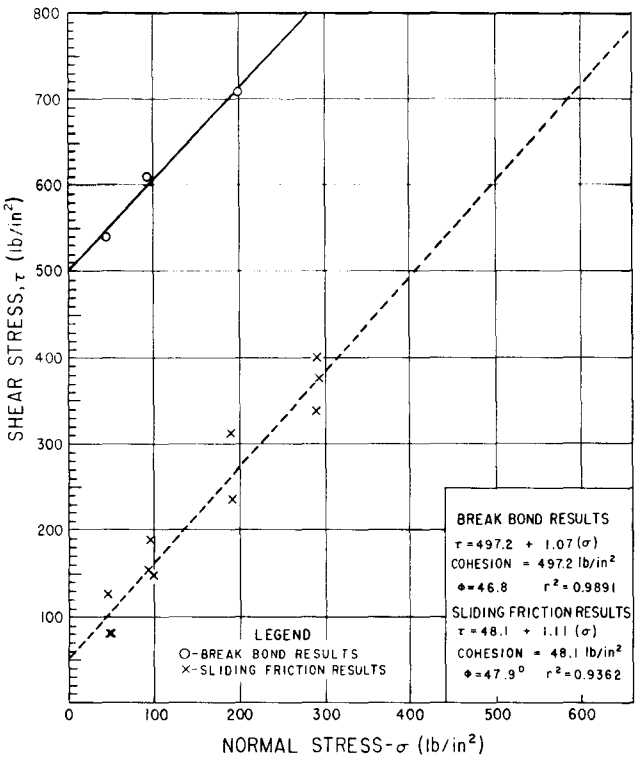


Figure 11. – Laboratory mix program, direct shear test mix L-1, 365 days, RCC.

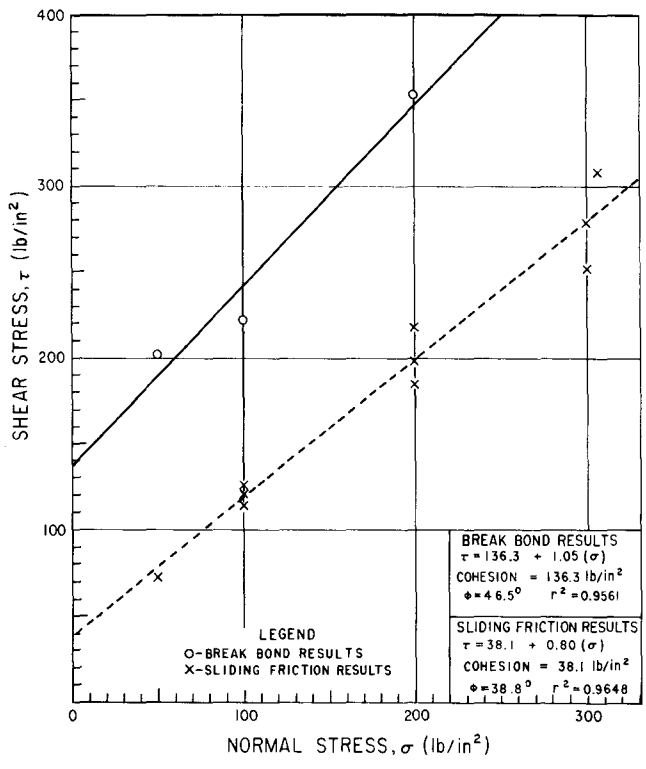


Figure 12. – Laboratory mix program, direct shear test mix L-2, 28 days, RCC.

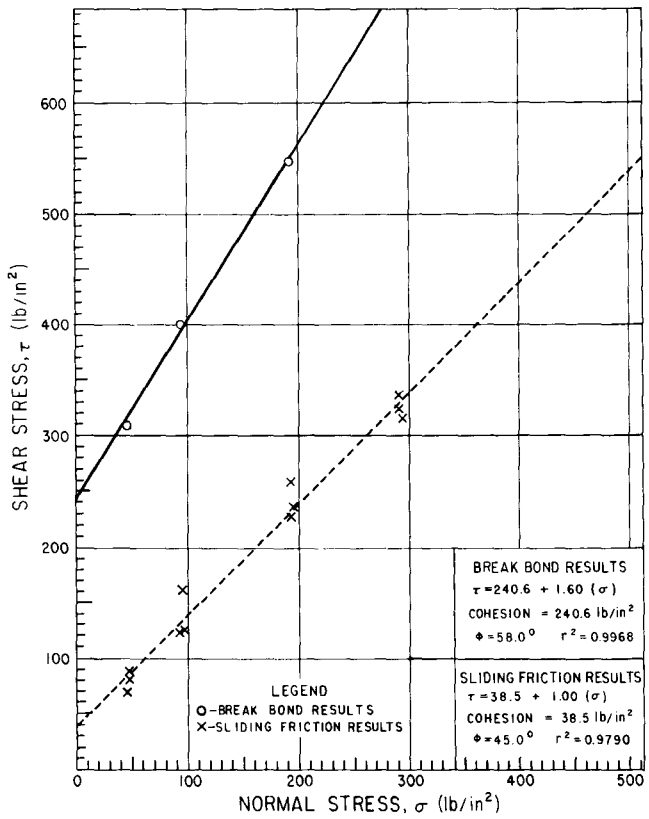


Figure 13. – Laboratory mix program, direct shear test mix L-2, 90 days, RCC.

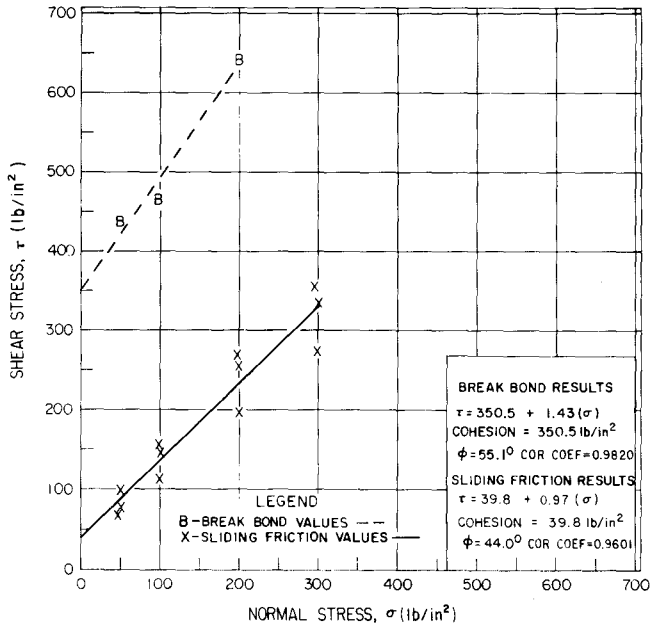


Figure 14. – Laboratory mix program, direct shear test mix L-2, 180 days, RCC.

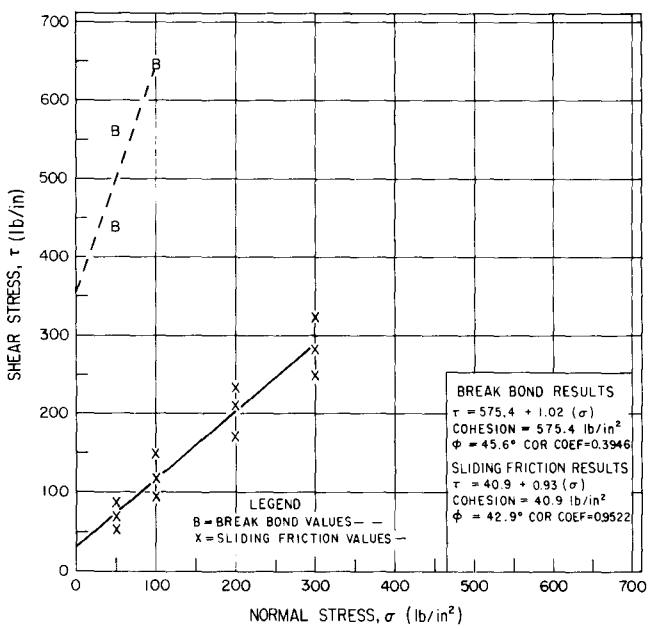


Figure 15. – Laboratory mix program, direct shear test mix L-2, 365 days, RCC.

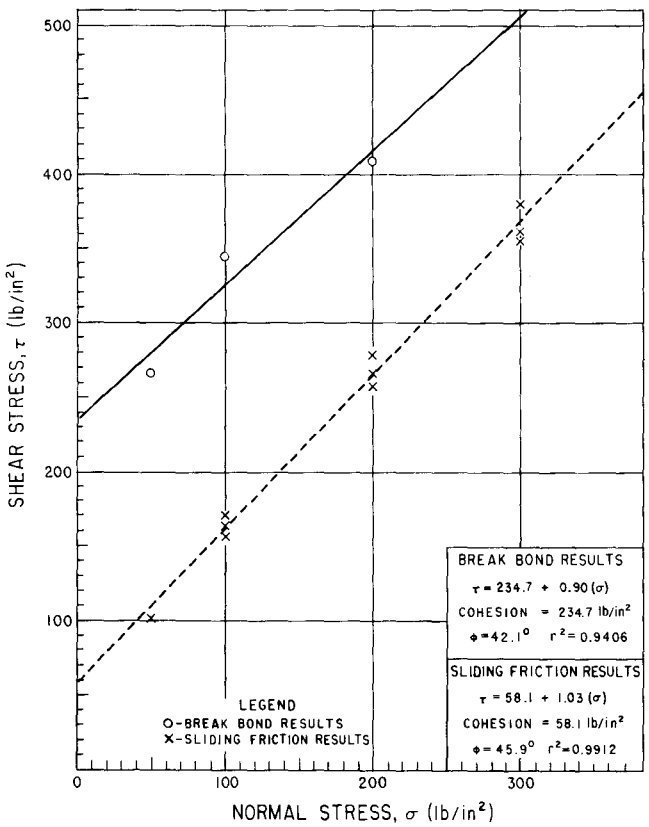


Figure 16. – Laboratory mix program, direct shear test mix L-3, 28 days, RCC.

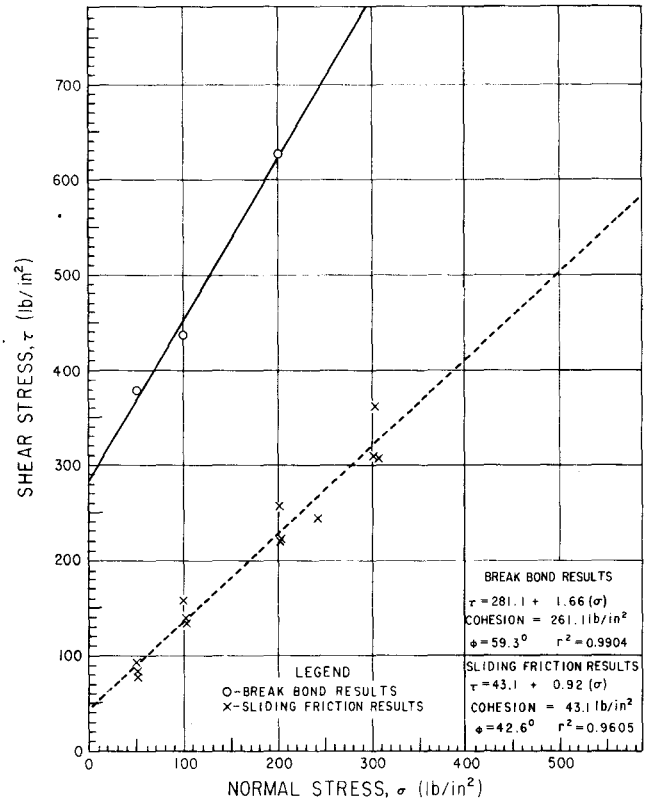


Figure 17. – Laboratory mix program, direct shear test mix L-3, 90 days, RCC.

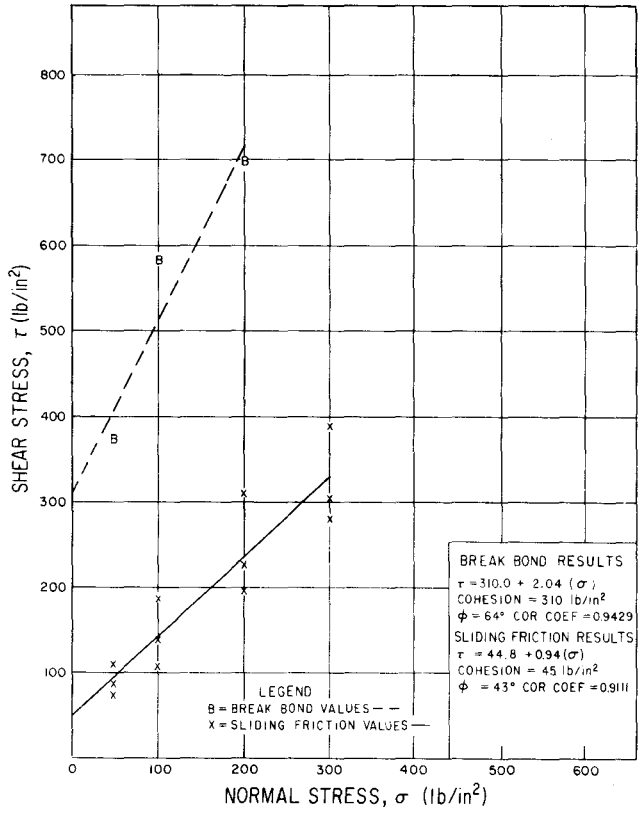


Figure 18. – Laboratory mix program, direct shear test mix L-3, 180 days, RCC.

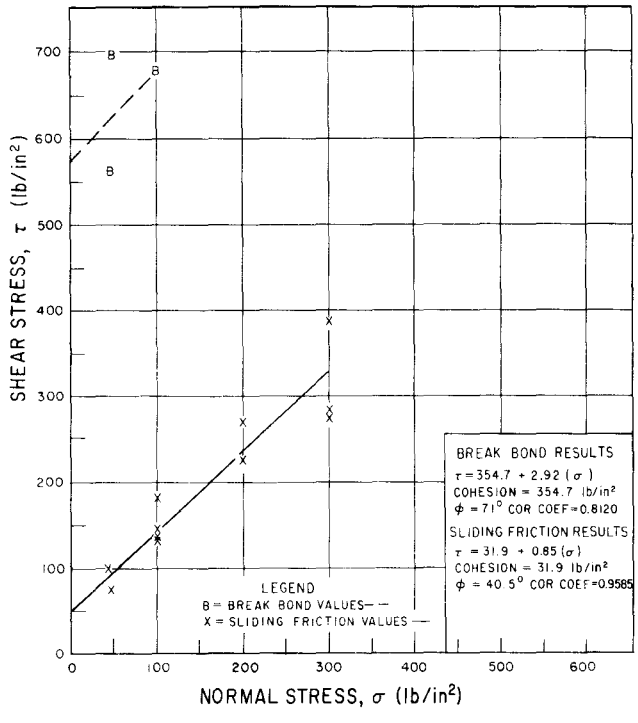


Figure 19. – Laboratory mix program, direct shear test mix L-3, 365 days, RCC.

Table 14. – Laboratory mix program – thermal expansion summary – RCC.

Mix No.	Fine aggregate source[1]	Moisture condition (percent saturation	Coefficient of thermal expansion[2], in/in x 10^{-6} / °F	No. of tests
L-1	40% PRS 60% TS	75 100	5.18 4.88	6 6
L-2	75% CC 25% PRS	100	3.97	12
L-3	100% CC	100	3.20	12
L-3	100% PFA	100	4.92	6

[1] All test mixes included project coarse aggregate (quartzose sandstone). Fine aggregate designations:

PRS – project pit-run sand (quartzitic sand, silt)
TS – crushed talus sand (quartzitic)
DFA – Denver area fine aggregate (granitic)
PFA – project area processed fine aggregate (quartzitic)

[2] Average of tests performed on 2- x 2- x 4-inch prisms.

coarse aggregate. Silt and organic materials, which were deposited throughout the talus source, were not removed prior to crushing and contaminated the sand which was generated during this process. Though the average percentage of minus No. 200 fines was similar to that used in the laboratory mix designs L-1 and L-2, variations of up to 22 percent minus No. 200 in the crushed talus sand and 34 percent minus No. 200 in the pit-run sand were common. This significantly affected the workability, water content, and strength of mixes T-1 and T-2, particularly mix T-1, which utilized 100 percent of this material. Mix T-2 was not as seriously affected since pit-run sand comprised only 25 percent of the total sand content.

Table 15. – Upper Stillwater Dam concrete test placement core program – thermal expansion summary[1].

Mix No.	Coefficient of thermal expansion, in/in x 10^{-6}/°F	No. of tests[2]
T-1	4.82	6
T-2	4.87	6
T-3	4.74	6
Facing concrete	4.56	6

[1] Test specimens were vacuum saturated to assure 100 percent saturation.
[2] Test specimens were 2- x 2- x 4-inch prisms.

An increase in the voids content of the coarse aggregate was caused by a high percentage of aggregate in the ⅜- to ¾-inch size range. Dry-rodded unit weight studies have shown that an increase in the No. 4 to ⅜-inch size fraction is necessary to produce a lower voids ratio, possibly due to the bedding characteristics of the sandstone. The increase in voids was not accounted for the test placement mix designs, resulting in a net decrease in the available free mortar (above minimum voids). This significantly reduced the compactibility of the in-place lifts of RCC, resulting in voids at the bottom of many lifts, particularly mix T-3, causing reduced bond at the construction joints.

The two mix designs used for facing element concrete, designated T-4 and T-5, are given in table 22. These mixes differed primarily in the amount of cement replaced with fly ash. The primary mix design on which the bulk of tests was performed utilized 50 percent replacement of cement by volume. Two facing elements (FE-5 and -6) were constructed with mix T-5 which utilized 60 percent replacement by volume. This mix was deemed unacceptable and was discontinued.

Coring Program

The coring program was designed to provide information on bonding between lifts and compressive

Table 16. – Laboratory mix program – adiabatic temperature rise summary – RCC.

Mix designation No.	Initial placement temperature, °F	Adiabatic temperature rise, °F								
		6 h	12 h	24 h	36 h	3 d	7 d	14 d	21 d	28 d
[1]L-1	59.8	0.5	1.0	2.5	6.9	25.0	33.7	40.7	44.5	45.5
[1]L-2	46.5	3.7	4.0	4.7	5.5	15.3	25.5	29.3	30.7	32.5
[1]L-3	44.5	1.8	2.0	2.5	2.9	3.8	20.0	29.5	31.7	34.3
[2]L-3	49.0	2.9	3.3	3.9	6.2	15.9	27.8	32.5	35.2	37.3
[2]L-5	53.5	4.4	4.8	6.4	11.8	24.3	36.3	43.5	46.7	48.3
[2,3]FE-1	60.5	3.6	4.5	18.7	37.3	56.0	67.7	69.5	69.0	67.9

[1] ASTM type D WRA (water reducing admixture) used for mix.
[2] ASTM type A WRA used for mix.
[3] 1-inch MSA aggregate represents similar proportions to facing element mixes used in Upper Stillwater Dam test placement.

Mix. No.	Cement, lb/yd³	Fly ash, lb/yd³	C/FA ratio by volume
L-1	180.4	208.3	40/60
L-2	121.0	269.0	25/75
L-3	129.0	286.0	25/75
L-5	156.0	344.0	26/74
FE-1	352.9	272.1	50/50

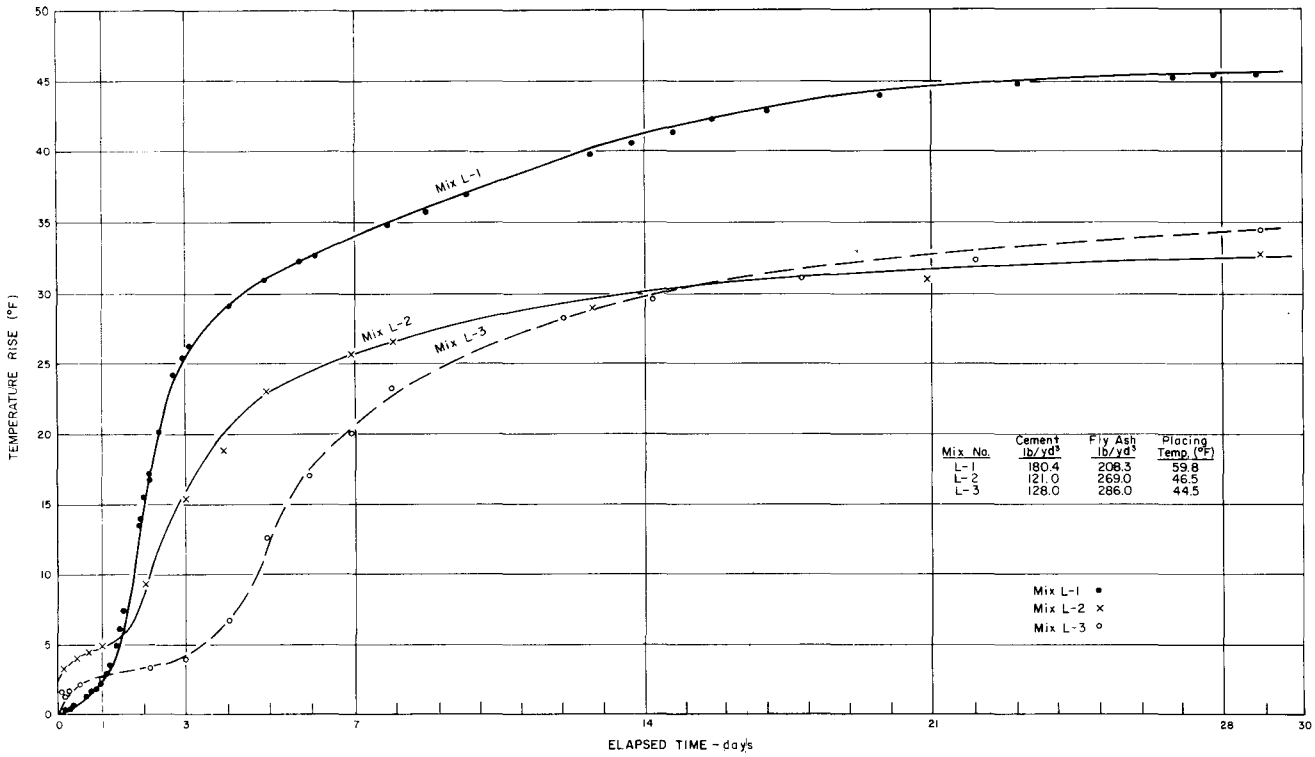


Figure 20. – Laboratory mix program, adiabatic temperature rise, mixes L-1, L-2, and L-3, RCC.

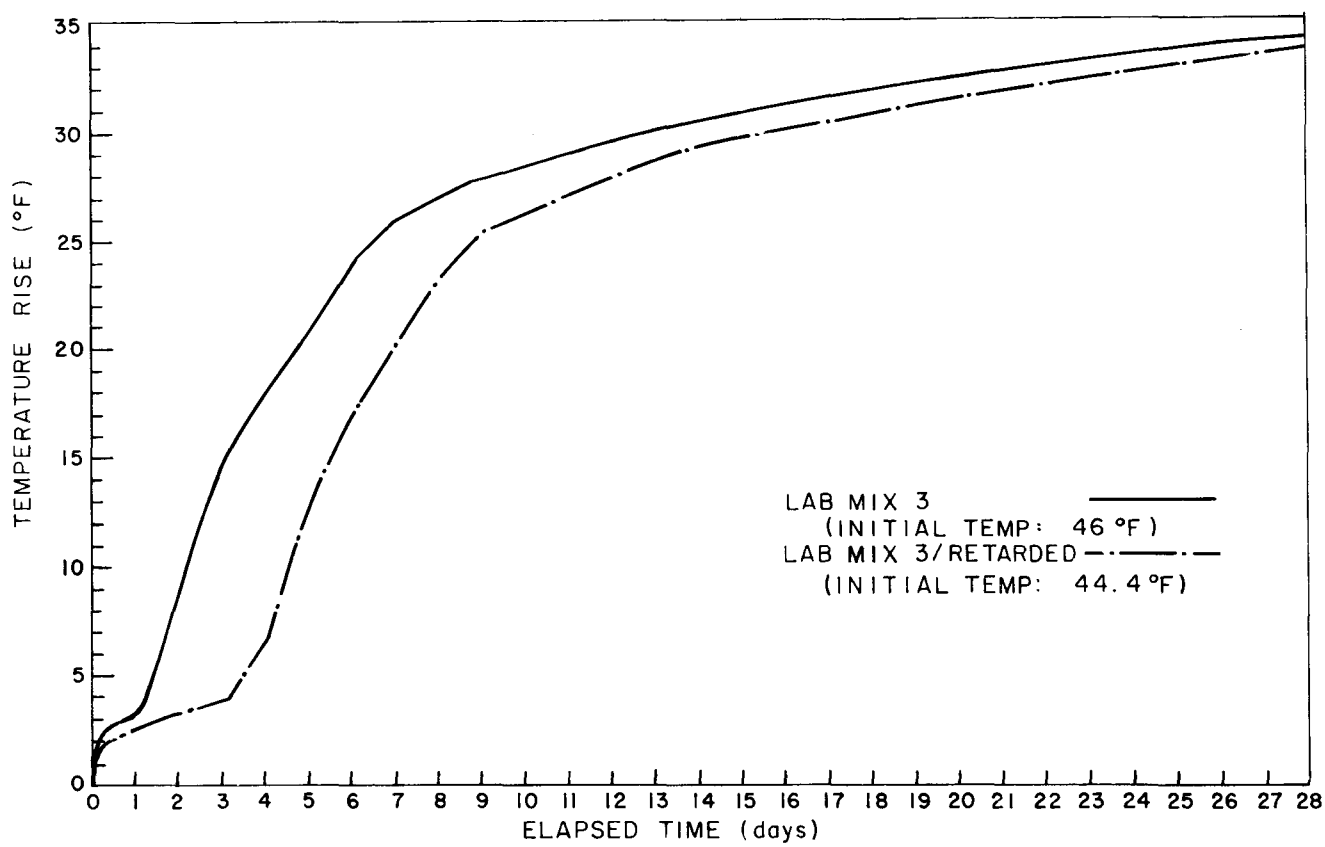


Figure 21. – Laboratory mix program, adiabatic temperature rise, mix L-3 with and without set retarder, RCC.

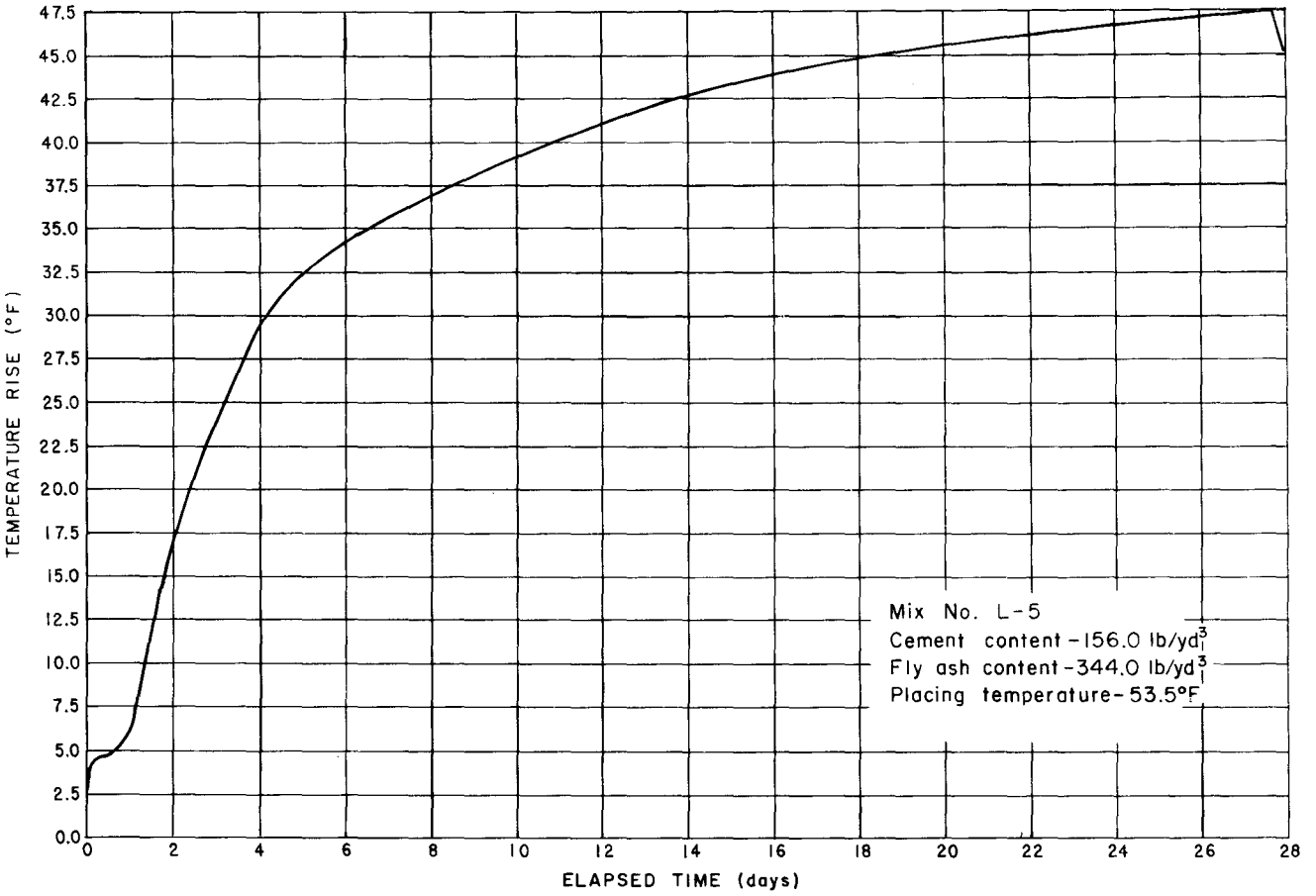


Figure 22. – Laboratory mix program, adiabatic temperature rise, mix L-5, RCC.

and tensile strength at various ages of the concrete test placement. Cores were extracted at approximately 28, 90, 180, and 365 days, sealed in boxes with moist sawdust, and shipped to the laboratory in Denver for testing. When received, the cores were unboxed, logged and photographed, marked, and prepared for testing. Figure 36 shows typical concrete cores extracted from the test placement. Except for the 28- and 90-day cores, specimens were kept moist by covering with damp rags and plastic sheets until cutting, after which they were stored in a 100 percent humidity room until testing. Some drying occurred in the 28- and 90-day cores between unboxing and cutting. It was necessary, in some cases, to surface dry the specimens in preparation for testing since the epoxy used for attaching strain gages had to be applied to a dry surface. A different epoxy used for the 180- and 365-day cores eliminated the necessity of surface drying test specimens.

Seven cores with a diameter of 5.75 inches were vertically drilled at 28 days' age, labeled 1-B and 2 through 7. At 90 days' age, 11 cores were extracted: cores 8 through 15 and 17 were drilled vertically; core 16 was drilled horizontally; and core 18 was drilled at a 31° angle from the vertical. The diameters of these cores ranged between 5.8 and 5.9 inches. Cores 19 through 28 and 29 through 38, with diameters of 5.75 to 5.81 inches, were vertically and horizontally drilled at 180 days' age. Lastly, the 365-day cores included eight vertically drilled cores, 39 through 46, and four horizontally drilled cores, 47 through 50, with an average diameter of 5.8 inches.

All test specimens were saw cut, with cut lengths varying from 7 to 12 inches. Only 10 specimens had a length less than 8 inches.

Core Test Program

Tests performed on cores from the test placement included compressive strength and elastic properties, tensile strength and elastic properties, unit weight, thermal expansion, cohesion and coefficient of internal friction, and drying shrinkage (see "Mixing and Testing Procedures" section).

Tests were performed on both vertical and horizontal cores, from RCC and facing element concrete, and

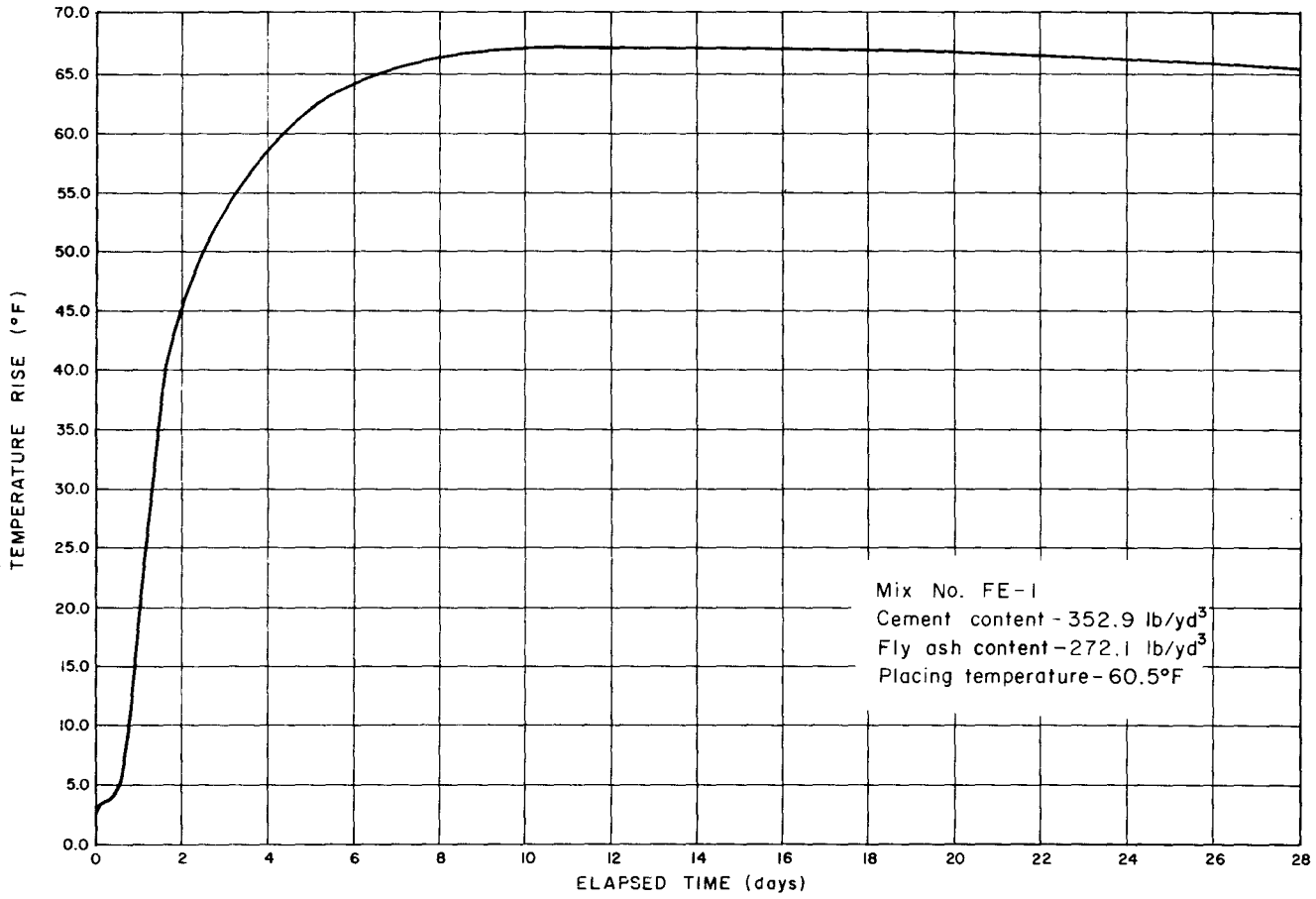


Figure 23. – Laboratory mix program, adiabatic temperature rise, mix FE-1, RCC.

results reported separately. Tensile tests were performed on both jointed and unjointed material, and are also reported separately.

Table 17. – Laboratory mix program – permeability summary – RCC.

Mix No.	Cementitious materials content, lb/yd³		Permeability, 10^{-4} ft/yr
	Cement	Fly ash	
L-1	182	210	14.5
[1]L-1	182	210	9.0
L-2	121	269	18.8
L-2	121	269	13.5
[2]L-3	129	286	4.0
[3]L-4	132	295	3.1
[3]L-5	156	344	1.5

[1] Estimated value at 4,000-lb/in² compressive strength.
[2] Estimated value – see figure 24.
[3] Mix designs developed as joint bonding mixes.

Test of Hardened Concrete Cores

Compressive strength tests were performed on core specimens in accordance with ASTM Designation: C 39-80 (Compressive Strength of Cylindrical Concrete Specimens). Many of the core specimens tested exceeded the scheduled test ages of 28, 90, 180, and 365 days due to delays in shipping and in specimen preparation. The ages listed in tables are actual test ages. Compressive strength results are corrected for cores with an *L/D* (length to diameter) ratio other than 2. Elastic properties testing (modulus of elasticity and Poisson's ratio) in compression were not performed at early ages due to low strength levels. An extensometer-compressometer frame was used to obtain 90-day core elastic properties, and epoxied strain gages were used for 180- and 365-day cores.

Direct tension tests were performed on specimens with and without construction joints using epoxied steel end plates, as in the laboratory mix program. Elastic properties testing was performed on unjointed specimens drilled at 180 and 365 days using epoxied strain gages.

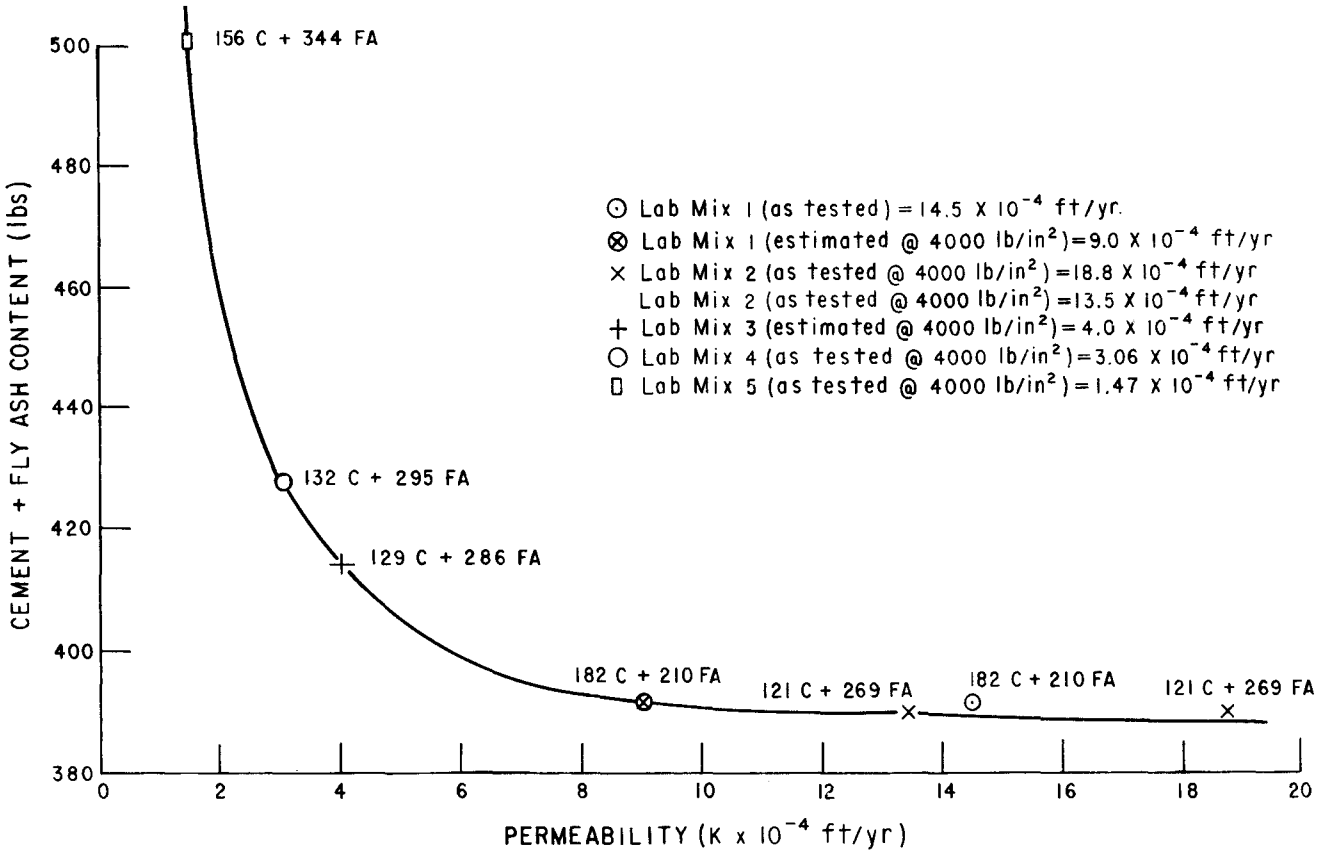


Figure 24. – Laboratory mix program, cementitious materials content versus permeability, RCC.

Table 18. – Laboratory mix program – diffusivity summary – RCC.

Mix No.	Average temperature, °F	Diffusivity, ft^2/h
L-1	71	0.060
L-1	86	.064
L-1	120	.061
L-1	193	.049
L-2	70	.059
L-2	87	.056
L-2	120	.057
L-2	195	.047
L-3	73	.067
L-3	93	.067
L-3	118	.057
L-3	189	.048
[1]L-3	95	.059
[1]L-3	186	.048

[1] Laboratory mix with sand from Moon and White aggregate source.

Compression and tension tests were performed similarly for horizontally drilled cores of facing concrete and RCC and are reported separately.

Static cohesion and coefficient of internal friction were determined for intact construction joints from shear break bond tests; sliding friction tests were then conducted on sheared joints. Test procedures were identical to those used for laboratory test specimens. A general equation for shear was determined for joints at specific ages where three or more break bond tests were performed; otherwise, a sliding friction equation was determined. Tests were performed on joints from both horizontally and vertically drilled cores. Test specimens were obtained from horizontally drilled cores by drilling 3-inch-diameter cores perpendicular to the construction joints.

Coefficient of thermal expansion was determined using 2- by 2- by 4-inch specimens saw cut from cores and saturated prior to testing. Test procedures are identical to those in the laboratory mix program.

The unit weight of hardened concrete specimens was determined by immersion in water to calculate the displaced volume.

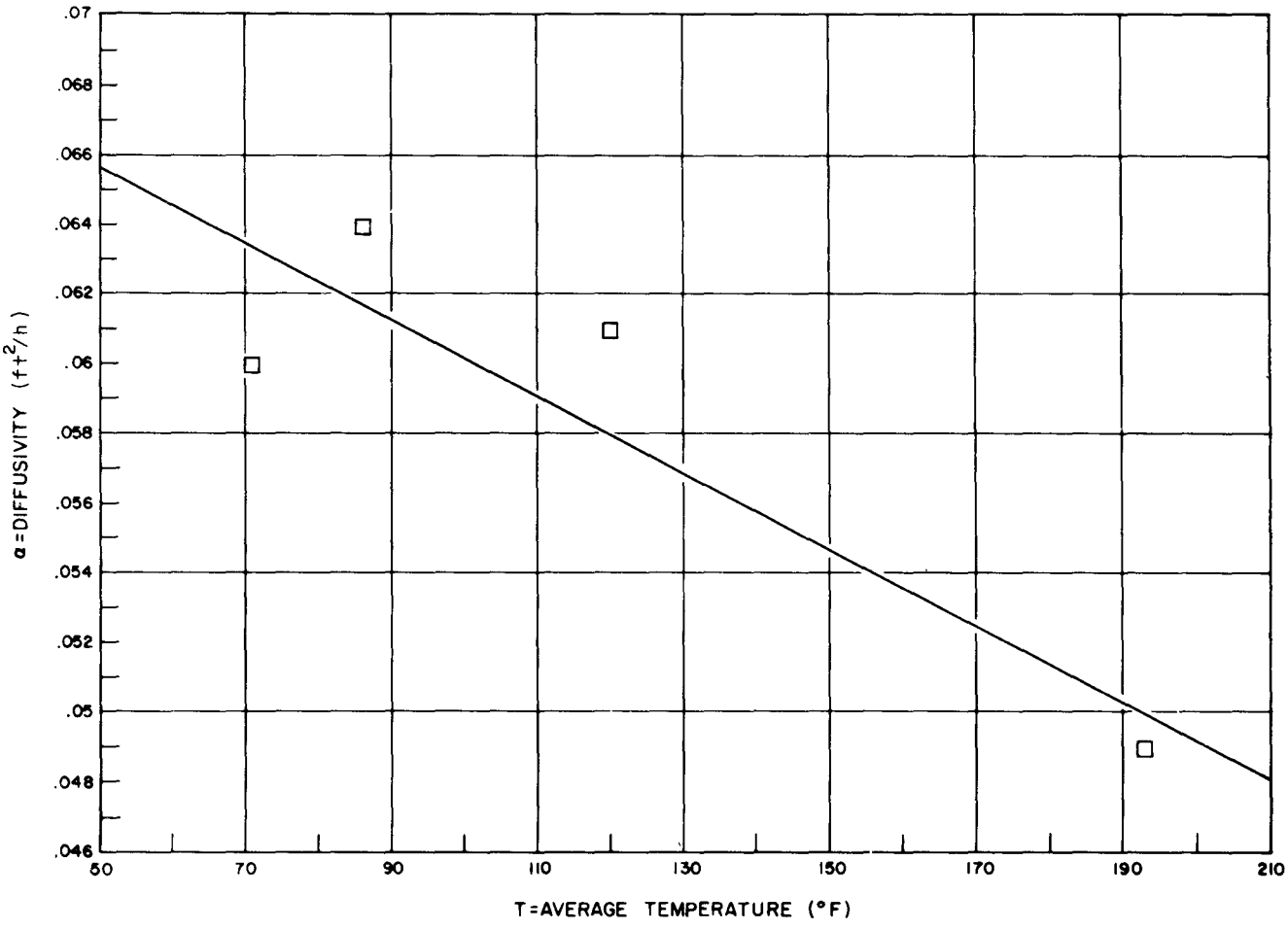


Figure 25. – Laboratory mix program, diffusivity mix L-1, RCC.

Compression, tension, unit weight, and thermal expansion tests were performed on 180- and 365-day horizontally drilled cores of facing concrete. The primary mix design, utilizing 50 percent fly ash by volume of cementitious materials, was used for the facing concrete and represents the majority of the test specimens. Two facing elements were constructed using a mix design with 60 percent fly ash by volume of cementitious material, but these elements were not selected for testing because of the decision that the mix was not suitable for the dam.

Core Test Program Results

Results of compressive strength testing are summarized in table 23. Individual test results are given in tables 24 and 25. The number of tests listed in the tables refers to compressive strength tests only; both the modulus of elasticity and Poisson's ratio are based on fewer tests. The average compressive strength of mix T-3 is higher than mixes T-1 and T-2 at all ages. This agrees with data from field control cylinders cast during construction of the test placement and is attributed to the use of 100 percent clean sand in the mix, as opposed to mixes T-1 and T-2, which used silty sands. Since mixes T-1 through T-3 were based upon laboratory mixes L-1 through L-3, similar strength relationships between mixes should have been expected. However, this was not the case. Mix T-1 showed significantly lower strengths than would have been expected from earlier laboratory studies. This is attributed to the use of crusher fines which were contaminated with silt. In the laboratory mix program, 60 percent clean crusher fines were blended with the pit-run sand. The strength of the 28-day cores from all three mixes (tested at approximately 50 to 55 days) is higher than anticipated when compared with results from later ages. This is not in agreement with laboratory or field control cylinder test results; however, the RCC in the test placement was initially cured at a higher temperature, followed by much cooler temperatures during the winter months, whereas the laboratory and field cylinders were cured at 73 °F. No precautions were taken to prevent the test placement from freezing during the winter, and the strength development

during that period was reduced if not completely stopped.

Correlations between W/C+FA (water to cement plus fly-ash ratio) and strength in both compression and tension cannot be developed because of the inaccuracy of readings of scale weights at the batch plant. It appears that the water content of mix T-1 was increased by the high silt content of the sand and decreased in mix T-3 by the use of clean sand. This conclusion is supported by a decrease in strength of field control cylinders and an increase in drying shrinkage for mix T-1, while the opposite is true for mix T-3 when each is compared to laboratory test results.

Test results of modulus of elasticity and Poisson's ratio in compression are given in tables 23 and 24. As with the laboratory mix program, the modulus of elasticity in compression is lower than what is normally found in conventional concrete. Again, this is most likely due to the lower modulus of the coarse aggregate. Results of Poisson's ratio fall within the normal ranges of conventional concrete and agree with laboratory results.

Tensile strength and elastic properties of cores are summarized in table 23. Individual test results of unjointed specimens are given in table 25 and jointed specimens in table 26. The results indicate that mix T-3 has the highest tensile strength which agrees with laboratory and field specimens. The lack of tensile strength development between the 90- and 365-day cores can be at least partially attributed to the low curing temperature during the winter months. Other factors contributing to poor tensile strength include poor consolidation with inconsistent lift thickness and rock pockets due to segregation. Some of the horizontal cores drilled parallel to the joints verify the existence of these conditions. Lifts 8 through 11 (mix T-3) were particularly deficient in this area. Later analysis revealed that this was caused by an increase in the voids content of the coarse aggregate, resulting in an insufficient volume of mortar in the mix design. Test results of elastic properties in tension are limited; however, they compare favorably with lab-

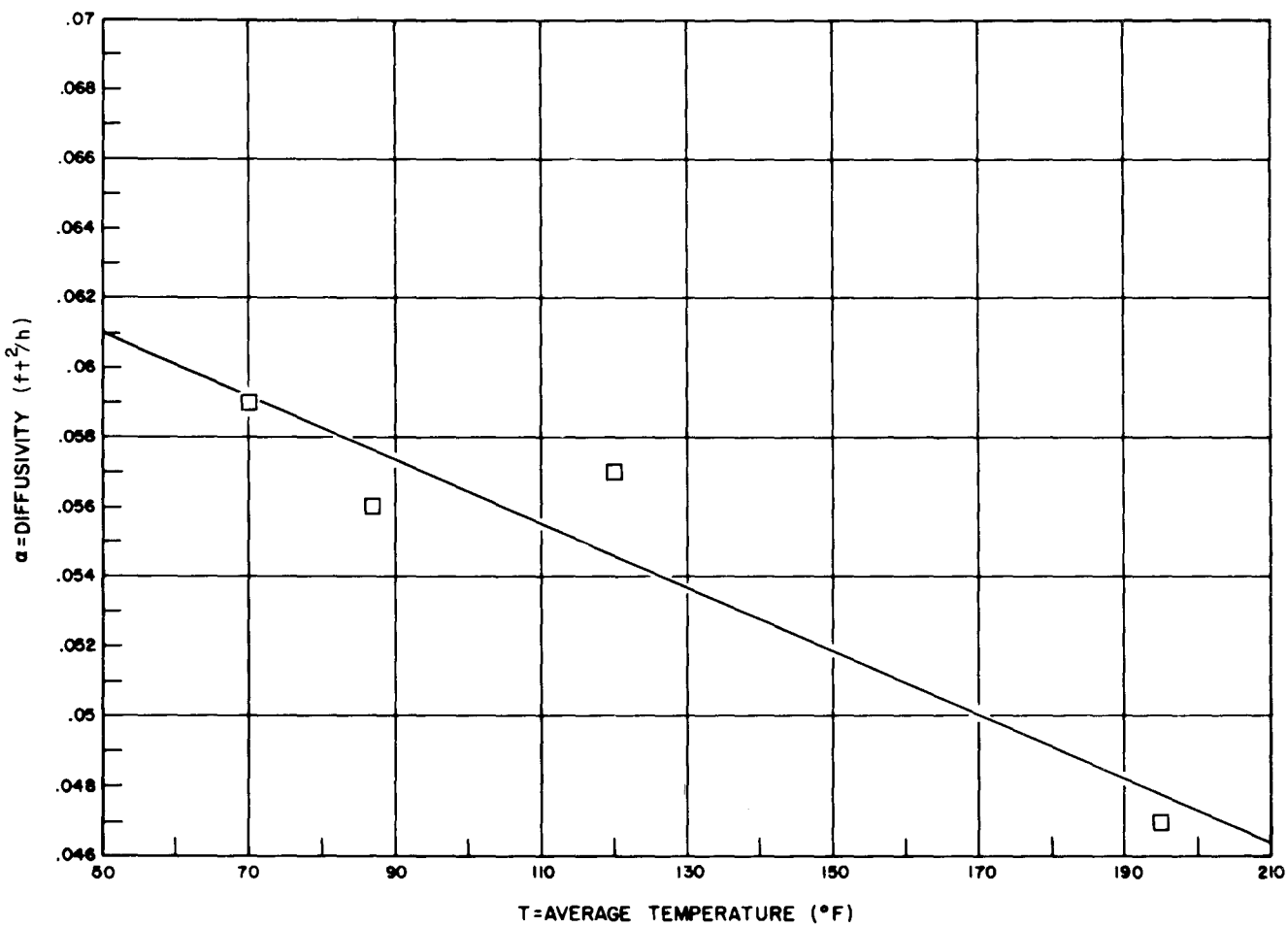


Figure 26. – Laboratory mix program, diffusivity mix L-2, RCC.

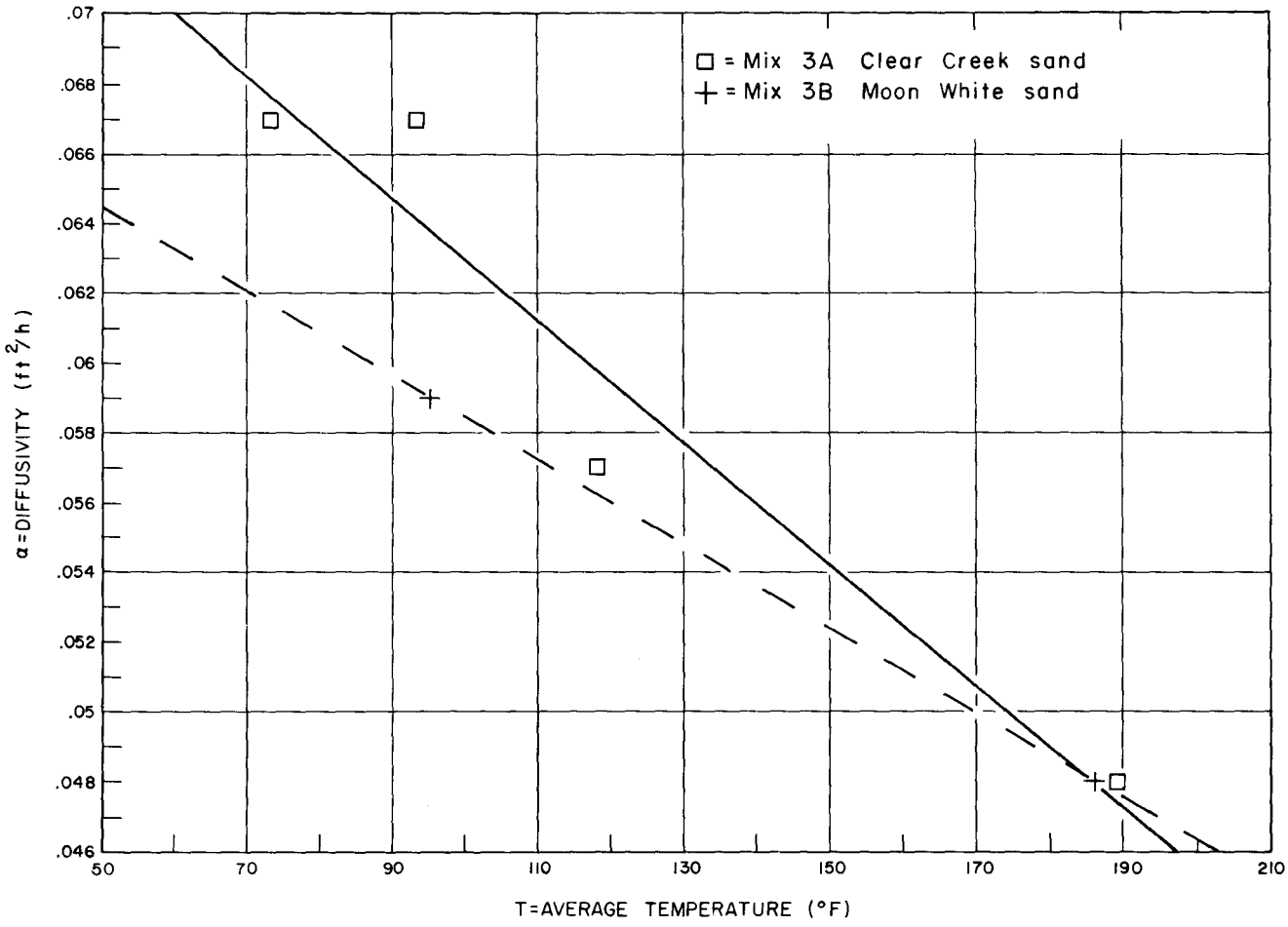


Figure 27. – Laboratory mix program, diffusivity mix L-3, RCC.

oratory results in both modulus of elasticity and Poisson's ratio.

The results of tensile strength testing of construction joints are given in table 26. These results are widely scattered and show no conclusive trends in bond strength development of the three mixes used in the test placement. Almost all of the joints that tested at strengths less than 50 lb/in^2 either were visibly dirty or had poor consolidation at the joint. A few specimens broke outside the joint area through rock pockets or areas of poor consolidation. Comments on tests of construction joints are given in table 26.

Table 27 summarizes the percentage of intact joints at each drilling age. The table includes the estimated time between the placement of each lift. Joints which were exposed only a few hours before the next lift was placed are considered fresh or "plastic" joints. Although the results are not consistent, the bonding of all these joints (1-2, 3-4, 5-6, 7-8, and 9-10) exceed 50 percent at 180 days, and each of these joints had at least one drilling age with a minimum of 70 percent of the joints bonded. The percentage of intact joints where the lift exposure was overnight or greater varies considerably. Two construction joints, 4-5 and 10-11, showed virtually no bond. Construction records indicate that considerable vehicle traffic occurred on lift 4 while it was fresh, thereby disturbing the surface. Cleanup operations did not adequately remove the debris generated, and the poor surface condition could account for the lack of bonding. Rain occurred during the placing of lifts 10 and 11, saturating the surface of lift 10 and bringing an excess layer of paste to the surface. In addition, in almost all cores, voids were visible in the bottom quarter of lift 11, possibly due to poor consolidation and the fact that the specified 12-inch lift thickness was exceeded. These conditions all contributed to the lack of bond development.

The results of tension tests reveal a number of conditions which significantly affect the bond potential of RCC. These include:

1. Cleanliness of the joint

2. Mortar content of the mix design

3. The time expired between lift placements

4. Precipitation during placement and rolling

5. Lift thickness

6. The degree of compaction achieved by the roller

In all cases, cleanliness of the joint is required for joint bonding. The other conditions are interrelated in their contribution to joint bond potential. The mortar content of the RCC mix should be sufficient to fill the voids in the coarse aggregate and to migrate down to the lower construction joint. This requires a mortar content greater than the minimum voids content of the aggregate. Test results indicate that the shortest time interval between the placement of lifts of RCC produces the best joint, provided that the exposed joint is not heavily disturbed by vehicular traffic. If the time interval exceeds the plastic condition, an increase in the mortar or paste content will be required to achieve bonding. If significant precipitation occurs during placement of a lift of RCC, work should be suspended. The vibrating roller causes paste to migrate to the surface which combines with the excess water, producing a weakened plane of laitance. This is probably the only situation where laitance will occur in RCC. Test results indicate that a lift thickness in excess of 12 inches should be avoided. The compactive effort of vibrating rollers is

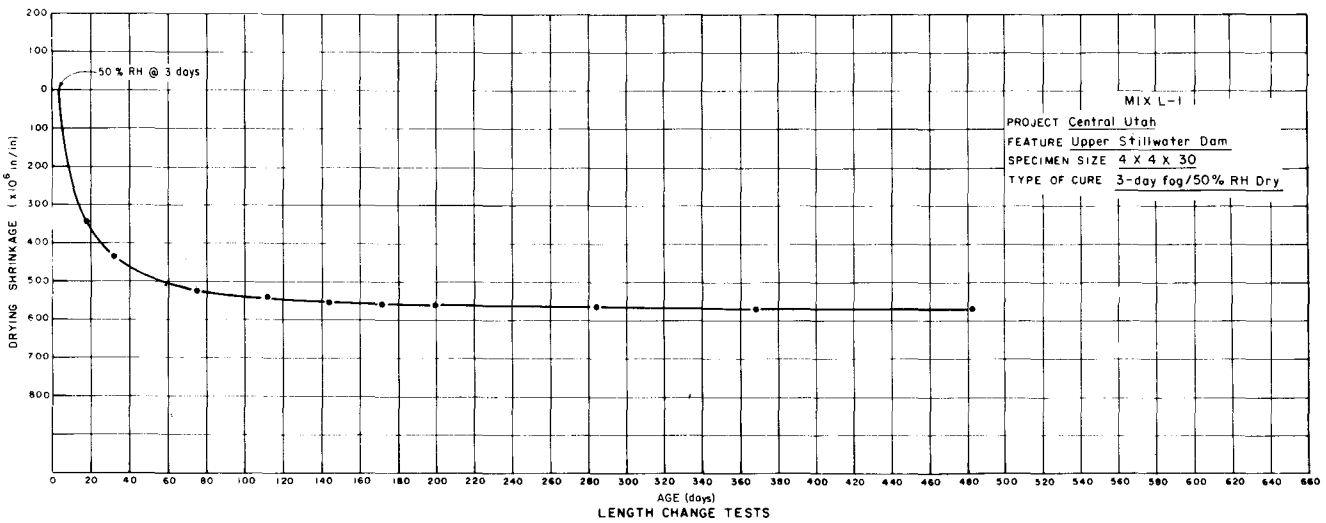


Figure 28. – Laboratory mix program, length change mix L-1, RCC.

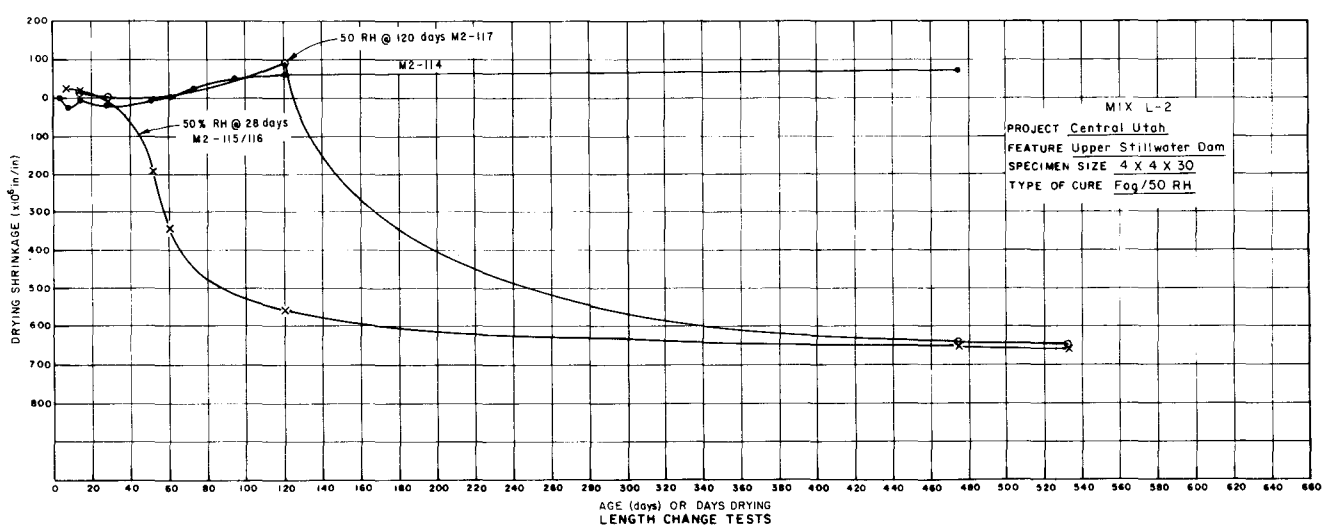


Figure 29. – Laboratory mix program, length change mix L-2, RCC.

Table 19. – Laboratory mix program – length change summary – RCC.

Age in days or days drying	Length change, inches x 10^{-6} Mixes[1]						
	[3]L-1-3	[3]L-2-0	L-2-28	L-2-121	[4]L-3-0	L-3-28	L-3-111
7	−230	+10	−45	−85	+50	−70	−80
14	−325	+20	−95	−185	+85	−160	−155
28	−425	+10	−250	−295	+75	−280	−260
60	−515	+5	−500	−445	+105	−395	−390
90	−540	+35	−560	−530	+150	−450	−460
120	−550	+55	−580	−595	+170	−480	−505
180	−560	+55	−600	−665	+185	−530	−555
240	−570	+55	−615	−705	+200	−550	−570
270	−570	+65	−625	−715	+200	−555	−565
365	−575	+65	−655	−735	+200	−555	−565
455	−580	+70	−655	−745	+200	−560	−565

Specimen designation:
[1] Last number in mix designation refers to age that specimen was removed from fog room and placed at 50 percent relative humidity. Zero signifies continuously fog-cured specimen.
[2] L-1 – laboratory mix 1
[3] L-2 – laboratory mix 2
[4] L-3 – laboratory mix 3

Table 20. – Upper Stillwater Dam concrete test placement – length change summary – RCC.

Age in days or days drying	Length change, inches x 10^{-6} Mixes[1]								
	[2]T-1-0	T-1-14	T-1-92	[3]T-2-0	T-2-14	T-2-90	[4]T-3-0	T-3-14	T-3-90
7	0	−340	−95	0	−135	−60	0	−130	−95
14	0	−510	−255	0	−270	−120	−10	−230	−145
28	−15	−815	−525	−5	−400	−190	−40	−320	−190
60	+10	−960	−800	−35	−475	−270	−60	−410	−250
90	+20	−995	−905	−40	−495	−295	−60	−440	−270
120	+50	−1005	−965	−55	−500	−310	−75	−460	−280
180	+65	−1020	−1005	−65	−510	−320	−100	−470	−295
240	+100	−1030	−1005	−50	−515	−330	−50	−480	−300
270	+110	−1030	−1065	−50	−515	–	−60	−480	–
365	–	–	–	–	–	–	–	–	–

Specimen designations:
[1] Last number in mix designation refers to age that specimen was removed from fog room and placed at 50 percent relative humidity. Zero signifies continuously fog-cured specimen.
[2] T-1 mix 1 – lift 3
[3] T-2 mix 2 – lift 6
[4] T-3 mix 3 – lift 8

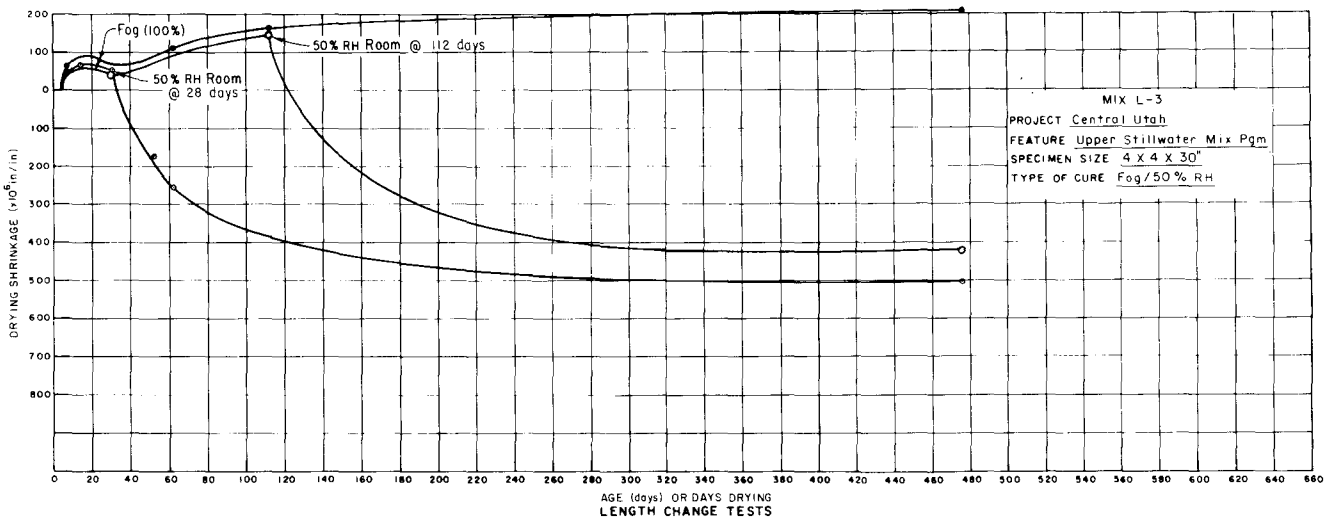


Figure 30. – Laboratory mix program, length change mix L-3, RCC.

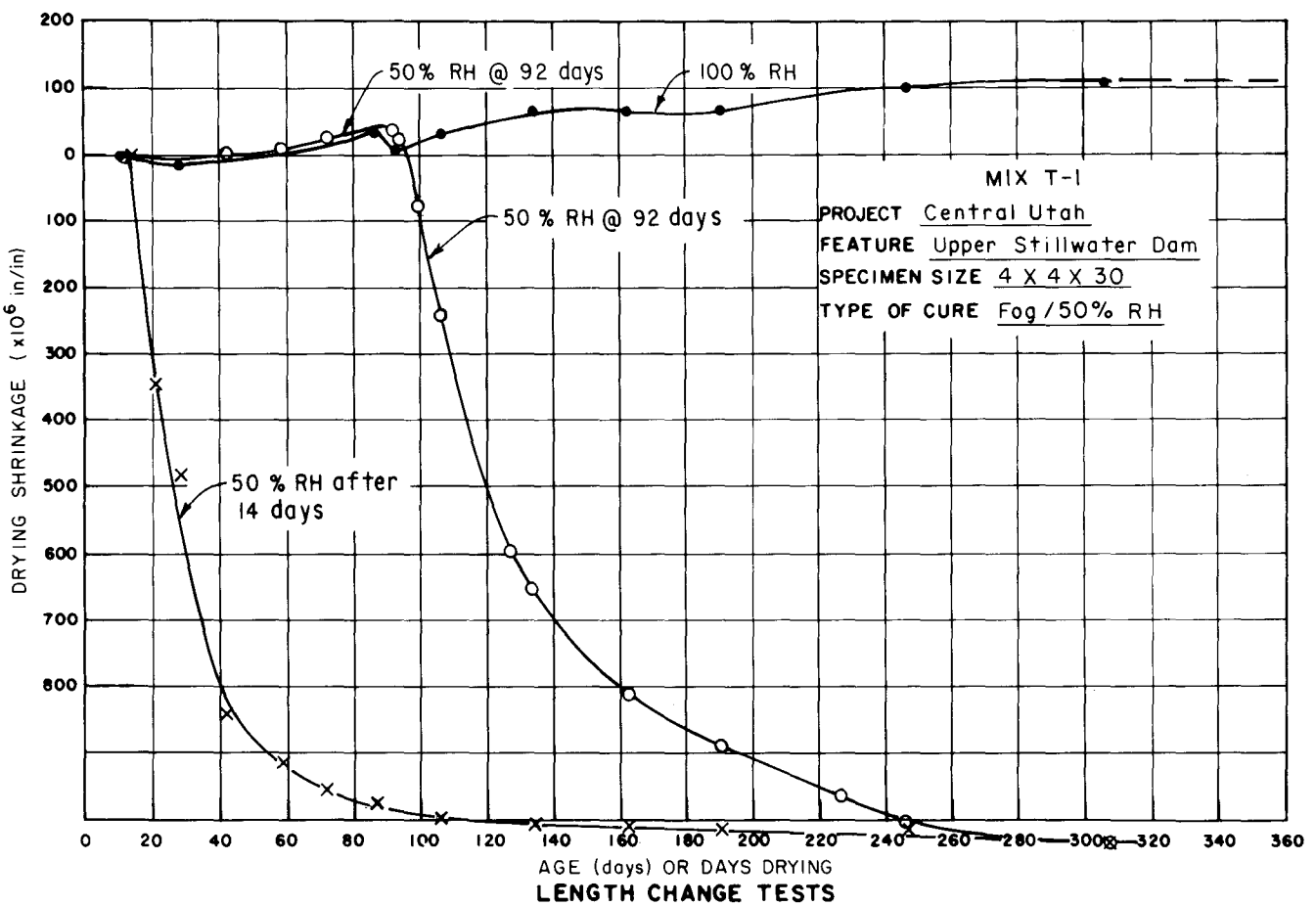


Figure 31. – Test placement length change mix T-1, RCC.

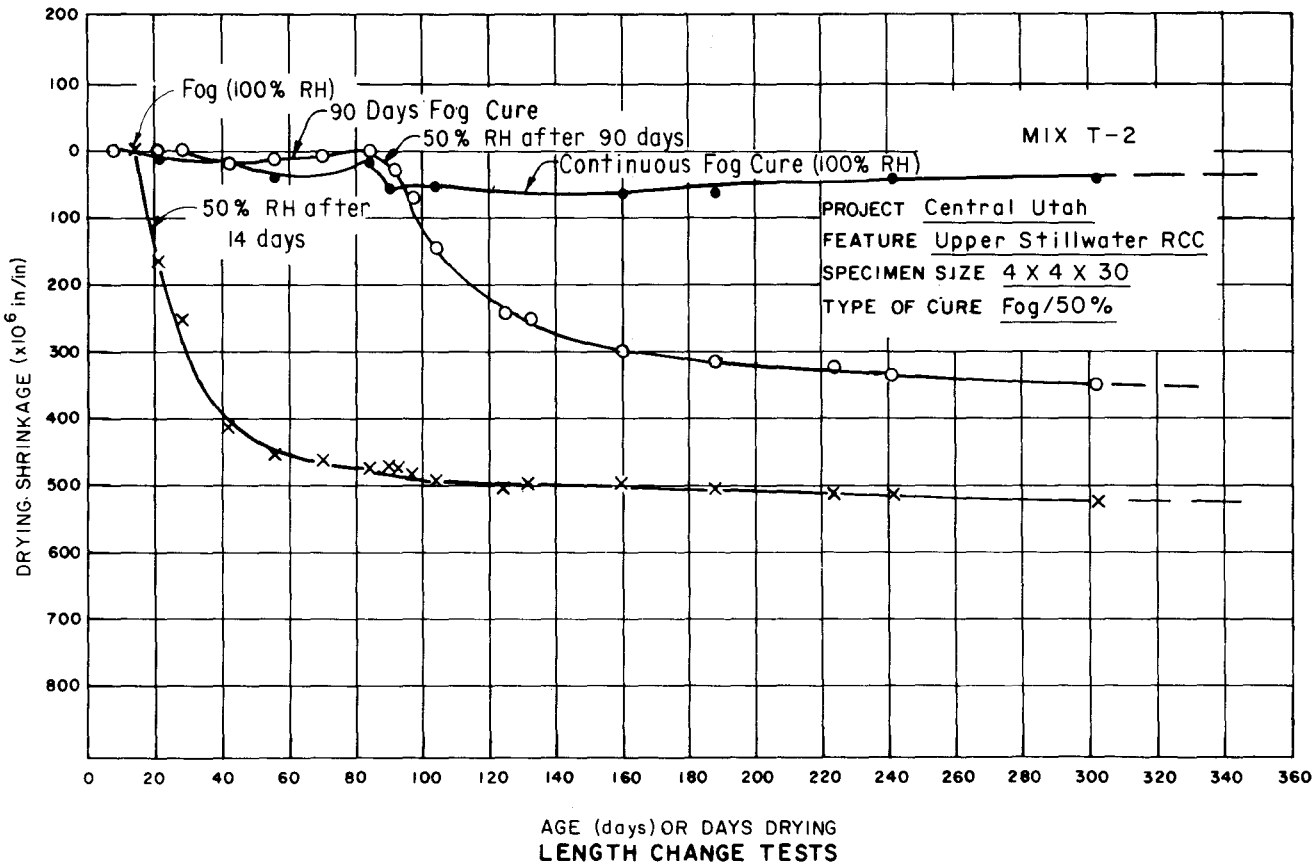


Figure 32. – Test placement length change mix T-2, RCC.

difficult to estimate because it will be different for RCC mixes with different ranges of workability. However, it is essential that full consolidation be achieved to assure bond and prevent seepage between lifts.

A considerable amount of research is still needed to evaluate the parameters which govern the degree of compaction achieved and the ability to develop bond between lifts of RCC. Grouting following construction may fill voids in a dam and decrease the permeability of the structure, but it is doubtful that it would restore bond.

A summary of shear and sliding friction tests for the core program is given in table 28. As with the tensile strength of RCC, the shear strength can also be influenced by the condition and treatment of the joint and degree of compaction achieved. Though surface roughness contributes to shear strength, it is probably not a significant factor for the higher strength levels required. A clean joint, with fully consolidated RCC, is still essential.

Smooth, clean laboratory specimens show almost double the shear strength of core specimens at equal ages under the same normal loads.

Compressive and tensile strength tests also were performed on horizontally drilled cores. The results of these tests are given in tables 29 and 30, respectively.

Facing Concrete

The mix designs for facing concrete used at the test section are given in table 22. The primary mix design utilized a fly ash percentage of 50 percent by volume of cementitious materials. A second mix, utilizing 60 percent fly ash by volume of cementitious materials, was evaluated but found to be unacceptable. The results of compressive and tensile strength testing of the facing concrete from horizontally drilled cores are given in tables 31 and 32.

TEST SECTION–FIELD TEST PROGRAM

Tests were performed on both fresh and hardened concrete samples obtained from the RCC test section. Fresh concrete tests included unit weight, air content and slump for facing concrete, and Vebe time (for workability) and unit weight for RCC. Concrete

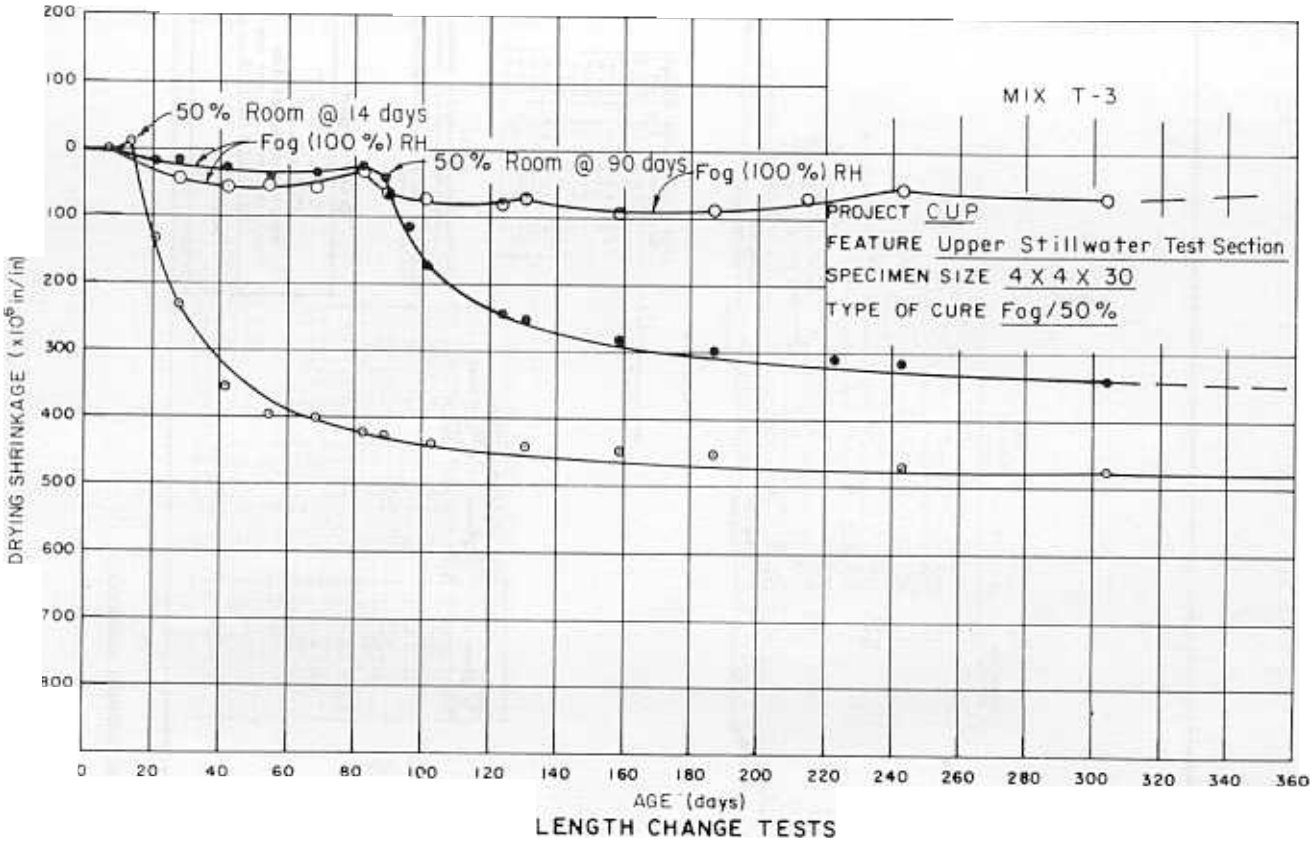


Figure 33 Test placement length change mix T-3, RCC.

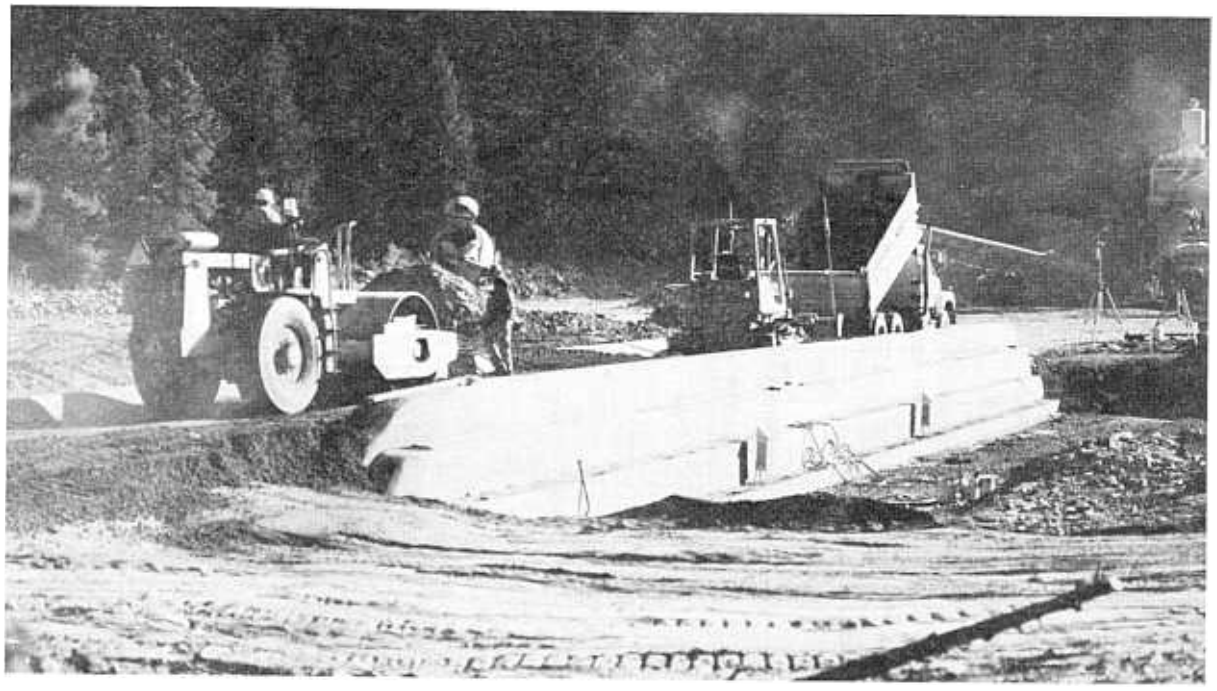
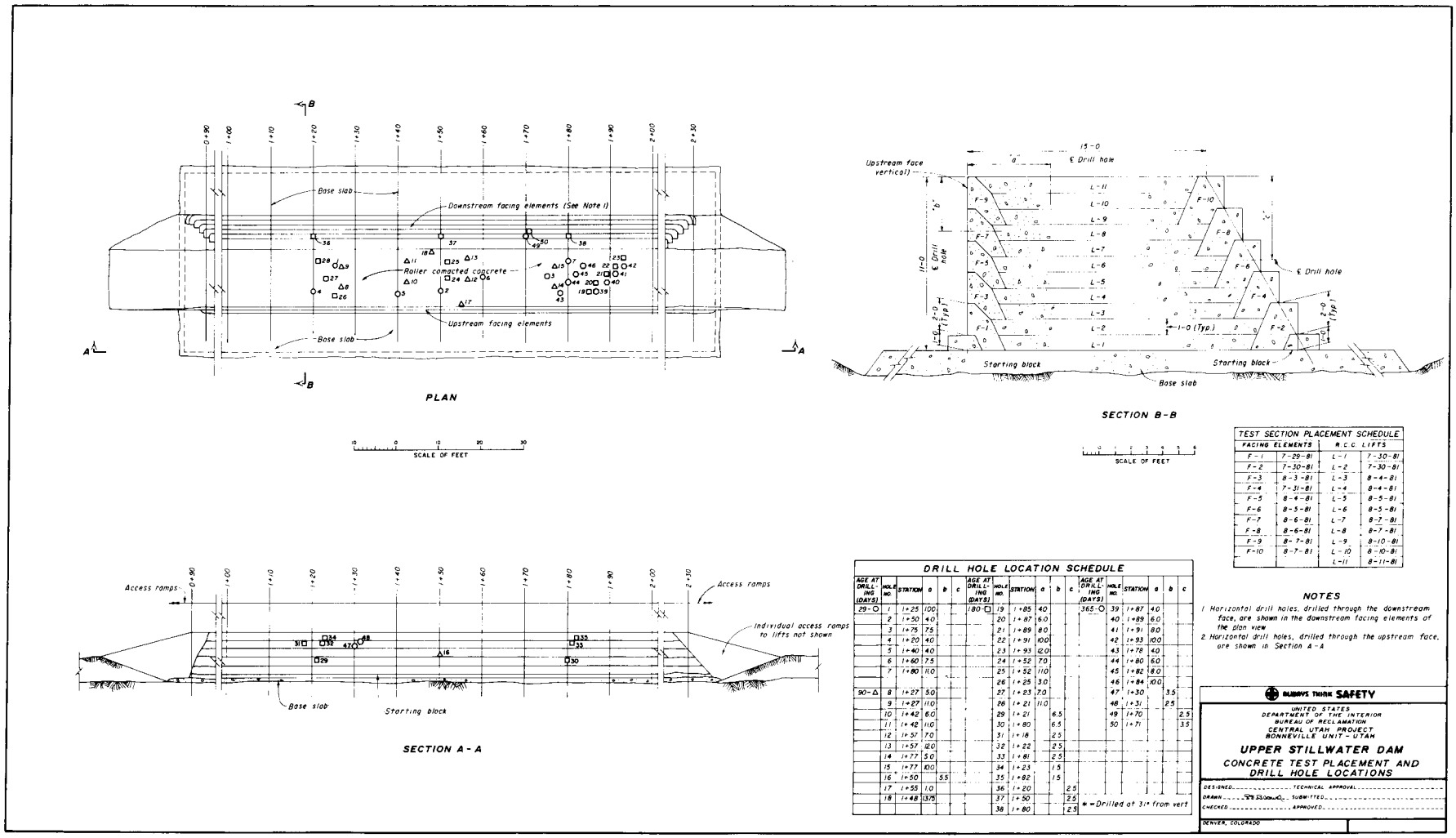


Figure 34. – View of Upper Stillwater Dam RCC test placement. Sequence of RCC construction showing concrete being dumped by an end dump truck, spread by a dozer, and compacted by a vibratory roller. P801-D-80873



TEST SECTION PLACEMENT SCHEDULE			
FACING ELEMENTS		R.C.C. LIFTS	
F-1	7-29-81	L-1	7-30-81
F-2	7-30-81	L-2	7-30-81
F-3	8-3-81	L-3	8-4-81
F-4	7-31-81	L-4	8-4-81
F-5	8-4-81	L-5	8-5-81
F-6	8-5-81	L-6	8-5-81
F-7	8-6-81	L-7	8-7-81
F-8	8-6-81	L-8	8-7-81
F-9	8-7-81	L-9	8-10-81
F-10	8-7-81	L-10	8-10-81
		L-11	8-11-81

DRILL HOLE LOCATION SCHEDULE

AGE AT DRILLING (DAYS)	HOLE NO.	STATION	a	b	c	AGE AT DRILLING (DAYS)	HOLE NO.	STATION	a	b	c	AGE AT DRILLING (DAYS)	HOLE NO.	STATION	a	b	c
29-○	1	1+25	10.0			180-□	19	1+85	4.0			365-○	39	1+87	4.0		
	2	1+50	4.0				20	1+87	6.0				40	1+89	6.0		
	3	1+75	7.5				21	1+89	8.0				41	1+91	8.0		
	4	1+20	4.0				22	1+91	10.0				42	1+93	10.0		
	5	1+40	4.0				23	1+93	12.0				43	1+78	4.0		
	6	1+60	7.5				24	1+52	7.0				44	1+80	6.0		
	7	1+80	11.0				25	1+52	11.0				45	1+82	8.0		
							26	1+25	3.0				46	1+84	10.0		
90-△	8	1+27	5.0				27	1+23	7.0				47	1+30		3.5	
	9	1+27	11.0				28	1+21	11.0				48	1+31		2.5	
	10	1+42	6.0				29	1+21		6.5			49	1+70			2.5
	11	1+42	11.0				30	1+80		6.5			50	1+71			3.5
	12	1+57	7.0				31	1+18		2.5							
	13	1+57	12.0				32	1+22		2.5							
	14	1+77	5.0				33	1+81		2.5							
	15	1+77	10.0				34	1+23		1.5							
	16	1+50		5.5			35	1+82		1.5							
	17	1+55	1.0				36	1+20			2.5						
	18	1+48	13.75				37	1+50			2.5						
							38	1+80			2.5						

k = Drilled at 31° from vert

NOTES

1. Horizontal drill holes, drilled through the downstream face, are shown in the downstream facing elements of the plan view.
2. Horizontal drill holes, drilled through the upstream face, are shown in Section A-A.

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL UTAH PROJECT
BONNEVILLE UNIT - UTAH

UPPER STILLWATER DAM

CONCRETE TEST PLACEMENT AND DRILL HOLE LOCATIONS

DESIGNED ____ TECHNICAL APPROVAL ____
DRAWN ____ SUBMITTED ____
CHECKED ____ APPROVED ____
DENVER, COLORADO

Figure 35. – Concrete test placement and drill hole locations.

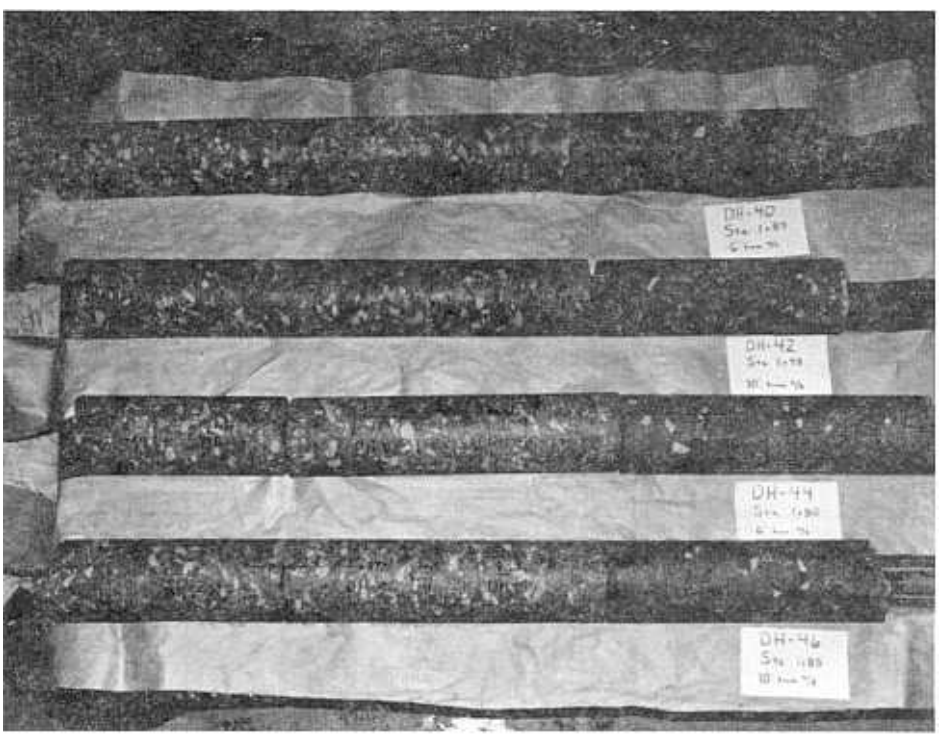
Table 21. – Upper Stillwater Dam concrete test placement – RCC mix design summaries.

Mix No.	T-1		T-2		T-3	
Lift No.	1-3		4-7		8-11	
	Weight, lb	Volume, ft^3	Weight, lb	Volume, ft^3	Weight, lb	Volume, ft^3
Air	1.0%	0.27	1.0%	0.27	1.0%	0.27
Water (W)	182	2.92	159	2.55	157	2.52
Cement (C)	182	.91	111	.56	118	.59
Fly ash (FA)	204	1.36	258	1.73	275	1.84
Fine aggregate:						
Pit run	432	2.84	–	–	–	–
Crushed talus	654	3.99	333	2.03	–	–
ASTM: C 33	–	–	833	5.18	1,132	7.04
Coarse aggregate: No. 4 – 1½ inch	2,282	14.71	2,279	14.69	2,287	14.74
Total	3,936	27.00	3,973	27.00	3,969	27.00
$\frac{W}{C + FA}$ (percent weight)	0.47		0.43		0.40	
FA (percent C+FA volume)	60		75		76	
Theoretical density (lb/ft^3)	145.78		147.14		147.0	

Table 22. – Upper Stillwater Dam concrete test placement – RCC mix designs – facing concrete.

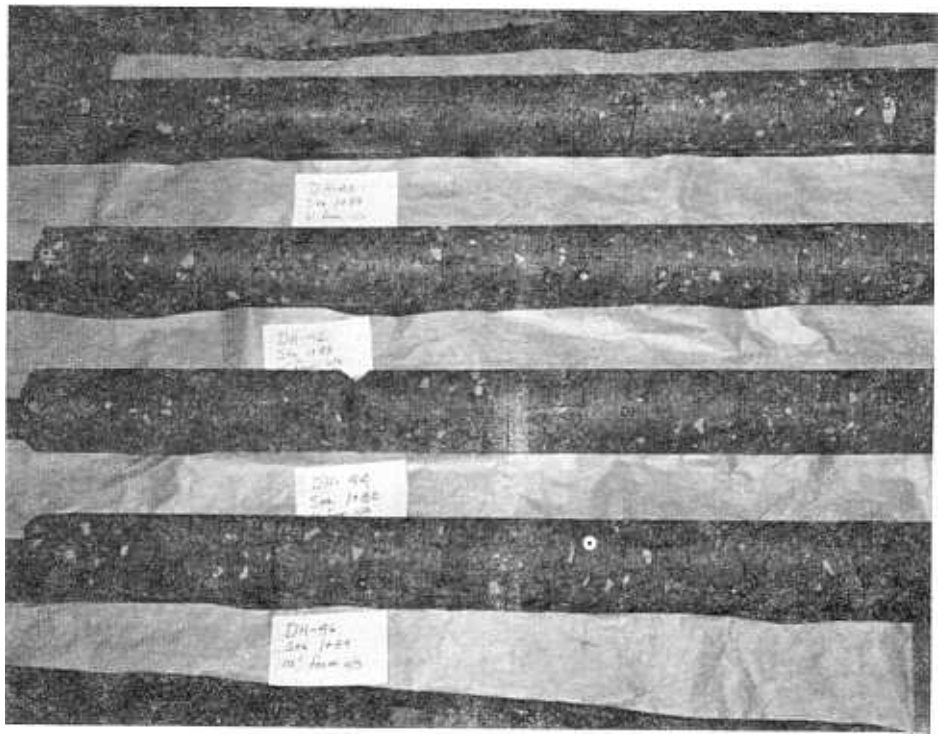
	Mix T-4		Mix T-5	
	Weight, lb	Volume, ft^3	Weight, lb	Volume, ft^3
Air	3.6%	0.97	1.6%	0.44
Water (W)	212	3.40	272	3.57
Cement (C)	369	1.85	317	1.59
Fly ash (FA)	277	1.85	357	2.38
Fine aggregate	1,398	8.70	1,433	8.92
Coarse aggregate	1,585	10.23	1,568	10.10
Total	3,841	27.00	3,947	27.00
$\frac{W}{C + FA}$ (percent weight)	0.33		0.40	
FA (percent C+FA volume)	50		60	
Density (lb/ft^3)	142.25		148.60	
C+FA (lb)	646		674	
Slump (in)	½		⅜	

a. Photo P801-D-80874.



b. Photo P801-D-80875.

Figure 36. – Typical concrete cores extracted from Upper Stillwater Dam, RCC test placement.



c. Photo P801-D-80876.

Figure 36. – Typical concrete cores extracted from Upper Stillwater Dam, RCC test placement – Continued.

Table 23. – Upper Stillwater Dam concrete test placement core program – average strength and elastic properties summary – RCC.

Age at drilling, days	Average age at test, days	Compressive strength, lb/in²	No. of tests	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in	Average age at test, days	Tensile strength, lb/in²	No. of tests	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in
Mix T-1										
28	55	1,660	7	–	–	64	98	6	–	–
90	130	2,400	5	1.63	0.14	155	99	1	–	–
180	246	2,750	4	2.78	.23	274	142	5	1.97	0.11
365	457	2,910	3	1.27	.16	457	148	4	2.29	.16
Mix T-2										
28	57	1,960	7	–		55	89	5		
90	111	2,690	8	1.36	.13	151	178	3		–
180	263	2,920	5	1.44	.09	265	173	2	1.74	.10
365	448	3,080	3	1.45	.15	450	138	5	1.79	.17
Mix T-3										
28	49	2,690	9	1.60	14	78	213	2	–	
90	102	3,080	11	2.10	12	151	199	3	–	–
180	238	3,120	7	2.35	11	250	224	5	3.21	.19
365	435	3,640	6	2.13	18	438	234	9	2.94	.12

Table 24. – Upper Stillwater Dam concrete test placement core program – compressive strength summary – RCC.

Age at drilling, days	Mix No. T-	Lift No.	Drill hole No.	Age at test days	Unit weight, lb/ft³	Compressive strength, lb/in²	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in
28	3	11	2	29	145.9	1,880	–	–
	3	11	5	51	–	2,720	–	–
	3	11	–	91	–	3,570	–	–
	3	10	5	53	–	3,060	–	–
	3	9	–	30	148.8	2,080	–	–
	3	9	2	30	148.3	1,990	–	–
	3	9	5	53	–	3,860	–	–
	3	8	3	55	147.5	2,440	1.6	0.14
	2	7	3	55	–	2,500	–	–
	2	6	3	56	–	1,720	–	–
	2	5	1B	35	147.4	1,910	–	–
	2	5	5	57	146.5	2,260	–	–
	2	5	–	99	–	2,710	–	–
	2	4	2	36	–	1,120	–	–
	2	4	4	58	–	1,520	–	–
	1	3	3	58	–	1,160	–	–
	1	3	4	58	–	2,240	–	–
	1	2	1B	40	148.6	1,180	–	–
	1	2	5	63	–	1,830	–	–
	1	1	2	40	146.3	1,630	–	–
	1	1	3	63	–	1,720	–	–
	1	1	4	63	–	1,860	–	–
90	3	11	8	120	144.8	2,640	1.94	.10
	3	11	11	120	146.3	3,510	2.54	.12
	3	11	–	109	145.4	2,000	1.82	.12
	3	10	11	120	146.1	3,540	–	–
	3	10	13	120	146.8	1,210	–	–
	3	9	9	120	147.1	4,830	–	–
	3	9	10	120	144.9	3,700	–	–
	3	9	11	120	145.5	3,450	–	–
	3	8	8	123	146.3	3,880	–	–
	3	8	11	123	147.8	2,500	–	–
	3	8	14	123	147.3	2,620	–	–
	2	7	10	123	146.8	2,350	–	–
	2	6	–	114	146.8	2,780	1.49	.16
	2	6	15	124	146.5	2,670	–	–
	2	5	8	125	144.6	2,590	–	–
	2	5	9	125	146.6	2,970	–	–
	2	5	15	125	147.0	4,080	–	–
	2	4	–	116	145.8	1,710	1.13	0.14
	2	4	14	127	144.8	2,370	1.46	.10
	1	3	14	126	–	1,540	–	–
	1	2	13	130	145.6	2,280	–	–
	1	1	8	131	144.7	2,410	1.52	.15
	1	1	10	131	145.4	2,620	1.74	.14
	1	1	13	130	147.0	3,170	–	–

Table 24. – Upper Stillwater Dam concrete test placement core program – compressive strength summary – RCC – Continued.

Age at drilling, days	Mix No. T-	Lift No.	Drill hole No.	Age at test days	Unit weight, lb/ft³	Compressive strength, lb/in²	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in
180	3	11	23	232	–	2,490	1.98	–
	3	11	19	232	–	3,690	–	–
	3	11	27	239	147.7	4,070	2.72	.11
	3	10	23	234	–	3,650	–	–
	3	10	24	240	144.0	1,900	–	–
	3	8	25	246	149.2	2,870	–	–
	3	8	22	243	144.9	3,200	–	–
	2	6	28	269	148.4	3,130	–	–
	2	5	19	245	144.8	3,110	–	–
	2	4	26	271	147.0	2,220	1.28	.12
	2	4	28	271	145.4	2,570	1.60	.06
	2	4	20	261	150.1	3,580	–	–
	1	2	19	272	145.3	3,050	3.68	.33
	1	1	20	262	147.0	2,460	1.39	.18
	1	1	19	262	146.1	2,610	4.08	.25
	1	1	26	215	146.0	2,880	1.95	.17
365	3	11	46	438	146.7	4,670	2.45	.15
	3	11	45	435	146.7	4,240	2.43	.14
	3	11	42	438	146.22	4,270	2.17	.15
	3	11	39	435	145.8	2,870	2.04	.19
	3	10	43	436	140.6	3,040	–	–
	3	8	44	442	149.5	2,740	1.54	.27
	2	5	39	441	144.8	3,100	1.44	.18
	2	4	44	451	147.4	2,810	1.42	.13
	2	4	40	451	147.0	3,340	1.48	.15
	1	3	41	457	146.6	2,630	1.27	.16
	1	1	43	460	147.7	3,250	–	–
	1	1	40	455	146.7	2,850	1.27	.16

test cylinders were cast for strength and elastic properties. Drying-shrinkage bars were also cast with RCC, and the results are given in the drying shrinkage section.

The test results of specimens can only be considered an estimate of actual RCC properties because of the variations in the batching, mixing, and handling of the RCC and facing concrete. The variations primarily resulted from the use of equipment which did not meet specified tolerances and accuracy. The results were further complicated because test batches for each mix were taken from different lifts for testing at different ages, which was necessary because of the difficulty and time required to cast a large number of the cylinders from one batch. Batch-to-batch consistency was not sufficient to establish acceptable strength development relationships for each mix.

Tests were performed on the cylinders from the test section to determine the compressive strength, tensile strength, and elastic properties of RCC. The test cylinders were cast in the same manner as in the laboratory program.

Results of field cylinders tests of RCC are given in table 33. Values of modulus of elasticity and Poisson's ratios are computed from stress-strain curves and are based on an average of two or three tests. All of the 180-day field test cylinders experienced some drying during transportation to the Denver laboratories. These tests are between 5 and 20 percent higher than saturated tests would yield, particularly mix T-3, which experienced the most drying.

The results of strength tests from the test placement field cylinder program agree with the core test results, but again differ from the laboratory program.

Table 25. – Upper Stillwater Dam concrete test placement core program – tensile strength summary – RCC.

Age at drilling, days	Mix No. T-	Lift No.	Drill hole No.	Age at test days	Unit weight, lb/ft³	Compressive strength, lb/in²	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in
28	3	11	47	56	–	165	–	
	3	11	7	99	–	260	–	
	3	8	4	60	–	88	–	
	2	7	5	60	–	48	–	
	2	6	2	34	145.0	42	–	
	2	5	4	59	–	113	–	
	2	5	5	60	–	108	–	
	2	4	5	60	–	134	–	
	1	3	5	63	–	136	–	
	1	2	2	40	144.5	109	–	
	1	2	3	67	–	53	–	
	1	1	1B	40	146.2	97	–	
	1	1	5	64	–	117	–	
	1	1	7	110	–	79	–	
90	3		9	148	–	240	–	
	3	9	12	166	–	209	–	
	3	8	–	140		149		
	2	6	12	153	–	184	–	
	2	5	13	171	–	194	–	
	2	4	9	129	–	155	–	
	1	3	11	155	–	99	–	
	1	1	9	133	–	129	–	
	1	1	14	150	–	48	–	
	1	1	15	159	–	77	–	
	3	11	28	288	146.8	216	4.34	0.19
	3	11	24	235	142.0	274	2.59	–
	3	11	21	239	–	244	2.71	
	3	9	26	243	148.4	292	–	
	3	9	27	–	147.3	–	–	
	3	8	26	246	146.4	94	–	–
	2	5	24	245	147.1	–	–	–
	2	4	27	271	146.12	171	1.70	.08
	2	4	25	258	146.8	175	1.77	.11
	1	3	25	261	146.32	188	–	–
	1	3	24	295	146.1	*88	–	–
	1	2	25	265	144.7	185	–	–
	1	1	27	275	145.6	155	2.02	.14
	1	1	28	275	145.6	94	1.91	.08
	3	11	41	435	146.6	267	3.21	0.13
	3	11	43	435	145.7	204	–	
	3	10	40	438	–	260	3.40	.09
	3	10	42	439	146.2	274	–	
	3	9	41	436	148.6	264	3.64	18
	3	9	42	439	148.1	322	–	
	3	8	40	442	148.7	144	2.38	.06
	3	8	41	439	147.6	167	2.68	.17
	3	8	42	442	148.2	200	2.30	.11

* Failure occurred between testing plate and specimen.

Table 25. – Upper Stillwater Dam concrete test placement core program – tensile strength summary – RCC – Continued

Age at drilling, days	Mix No. T-	Lift No.	Drill hole No.	Age at test days	Unit weight lb/ft³	Compressive strength, lb/in²	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in
	2	6	42	450	154.6	55	1.70	.10
	2	5	41	441	146.4	185	1.88	.20
	2	5	42	450	146.5	182		
	2	4	41	456	150.4	105	–	
	2	4	46	451	147.0	163	1.80	.20
	1	3	46	451		120	2.10	.14
	1	2	39	460	148.5	165	–	
	1	1	39	460	148.4	178	2.56	.15
	1	1	44	455	146.2	127	2.20	.20

Table 26. – Upper Stillwater Dam concrete test placement core program – construction joint tensile strength summary RCC.

Age at drilling, days	Mix No. T-	Construction joint No.	Drill hole No.	Age at test, days	Unit weight lb/ft³	Tensile strength, lb/in²	Comments on break
28	3	10-9	1B	30	147.4	116	
	3	10-9	2	30	146.2	100	
	3	10-9	3	54	–	207	
	3-2	8-7	5	60	–	48	Partly on joint
	3-2	8-7	2	35	147.9	19	Poor consolidation at break
	2	7-6	7	103	–	17	
	2	7-6	4	58	–	85	On joint
	2	6-5	6	104	–	43	
	2-1	4-3	1B	38	146.2	59	Broken in lift 3
	1	3-2	7	106	–	10	
	1	2-1	6	110	–	42	Poor consolidation – no bond to aggregate
90	3	10-9	8	123	–	191	Five percent voids – failed on joint
	3	10-9	9	123	–	207	
	3	10-9	15	140	–	206	
	3-2	8-7	10	182	–	163	
	3-2	8-7	9	161	–	169	On joint
	3-2	8-7	12	152	–	81	
	2	7-6	9	169	–	75	On joint
	2	7-6	14	169	–	132	
	2	6-5	–	128		84	
	2	6-5	11	154		48	
	2-1	4-3	15	155		95	
	1	3-2	9	176		79	
	1	3-2	13	159		56	Dirty joint
	1	3-2	10	133		23	One-half broken aggregate – dirty joint
	1	2-1	12	150		*46	Dirty joint
	1	2-1	15	171		60	On joint

Table 26. – Upper Stillwater Dam concrete test placement core program – construction joint tensile strength summary – RCC – Continued.

Age at drilling, days	Mix No. T-	Construction joint No.	Drill hole No.	Age at test, days	Unit weight, lb/ft³	Tensile strength, lb/in²	Comments on break
180	3	10-9	25	243	147.5	165	
	3	10-9	22	–	145.8	–	
	3	10-9	20	–	146.9	–	
	3	9-8	24	236	147.8	51	Dirty joint
	3	9-8	23	236	148.3	22	Poorly consolidated joint
	3-2	8-7	23	237	–	133	
	3-2	8-7	19	232	–	109	Fifty percent broken aggregate voids at break
	3-2	8-7	21	232	–	126	Some broken aggregate
	3-2	8-7	24	–	147.2	–	
	3-2	8-7	27	–	147.6	–	
	2	7-6	22	255	148.2	18	
	2	7-6	24	239	147.1	79	Clumps of dirt – wood on joint
	2	7-6	19	232	–	121	Poor consolidation – dirty joint
	2	7-6	25	258	147.6	85	
	2	6-5	21	238	–	115	
	2	6-5	22	256	–	75	
	2	6-5	23	238	147.6	30	Voids at joint – dirty joint
	2	6-5	27	240	149.6	39	Loose sand on joint
	2	6-5	20	259	–	75	
	2-1	4-3	19	268	146.1	83	
	2-1	4-3	22	258	146.1	127	
	2-1	4-3	21	268	144.8	102	
	1	3-2	26	271	145.9	127	
	1	3-2	28	271	146.8	94	
	1	2-1	21	272	–	119	
	1	2-1	22	262	145.9	66	
365	3	10-9	46	439	147.4	253	
	3-2	8-7	39	439	149.0	124	On joint
	3-2	8-7	45	439	148.2	117	Dirty joint
	2	7-6	41	439	147.6	12	Very poor consolidation
	2	6-5	40	450	145.6	83	Ten to 15 percent broken aggregate – poor consolidation
	2	6-5	44	450	146.8	43	Twenty percent broken aggregate – poor consolidation
	2	6-5	45	441	147.1	70	Dirty joint
	2-1	4-3	42	451	146.2	65	Broken in lift 3 – poor consolidation
	2-1	4-3	45	456	146.7	97	Twenty-five percent through joint
	1	3-2	40	451	146.1	98	Poor consolidation
	1	3-2	44	451	145.1	110	Seventy-five percent broken aggregate
	1	3-2	45	456	147.2	132	In lift 3
	1	2-1	41	460	147.4	102	Through joint
	1	2-1	46	455	145.6	115	Ten percent broken aggregate

* Shock-loaded.

Table 27. – Upper Stillwater Dam concrete test placement core program – construction joint bonding summary – RCC.

Joint No.	Mix No. T-	Time between lift placements, hours	Percent[1] of joints bonded at: 28 days	90 days	180 days	365 days	Comments
1-2	1	3.5	14	38	56	75	
2-3	1	86.5	14	75	30	71	
3-4	1-2	3.0	43	43	70	50	
4-5	2	40.0	0	0	17	0	Lift 4 was disturbed by curbing equipment
5-6	2	0.0	14	38	80	38	
6-7	2	20.0	43	50	80	71	
7-8	2-3	.5	71	67	100	63	
8-9	3	71.0	14	0	70	13	
9-10	3	1.0	86	100	88	67	
10-11	3	20.0	0	0	20	0	Rain occurred between the placing of lifts 10 and 11

[1] Percentages are based on a sample size of 4 to 10 cores.

Mix T-3 possesses the highest strength, followed by mixes T-2 and T-1, respectively. The difference between these results and the laboratory program was explained previously as being attributed to changes in materials properties. The strength of field cylinders is considerably higher than core strengths. This is attributed to the curing which was provided (73 °F water cure) and the lack of voids which appeared in many core specimens. The late strength development of the test placement mixes agrees with the laboratory program having 28-day strengths at one-third of the 1-year strengths. Based on field test cylinders, all three mix designs meet the required 1-year strength criteria both compression and tension. However, this does not address the problems of achieving consolidation and joint bonding which were evident with the core results.

Mix T-1, which had a higher silt content in the sand than mix T-2, had lower strengths and considerably higher drying shrinkage (see laboratory program results, "Drying shrinkage" section). This apparently was caused by an increase in water demand, though it cannot be substantiated by recorded batch weights. Mix T-3 had considerably higher strengths than either of the other two mixes.

It has been reported previously by others [7] that increasing the silt content can increase compressive strength. Test results from this program do not support this conclusion. It appears this may be true for lean concrete mixes with low strength requirements, where the silt serves to increase the paste content above the minimum voids required for achieving compaction. Thus, the silt would help achieve better compaction rather than contribute toward any cementitious chemical reaction. In the case of meeting the high-strength and bonding criteria for Upper Stillwater Dam, the use of high silt contents is considered detrimental, provided compaction can be achieved without it.

The results of facing concrete field cylinders are given in table 33. These tests indicate that considerable strength potential exists if the cure on the facing concrete is maintained. The facing elements were cured with curing compound on the upstream face and a 7-day water cure on the downstream face. The limited number of tests performed reveal that the strength development of cores compares favorably with cylinders. This indicates that the moisture loss from facing elements is relatively low, probably due to low permeability.

Table 28. – Upper Stillwater Dam concrete test placement core program – shear and sliding friction summary by construction joint – RCC.

Joint No.	Mix No. T-	Age at drilling, days	No. of specimens	Average age at test, days	Break bond equation[1,2]	Sliding friction equation[3]
9-10	3	28	3	81	[4]	S = 49.0 + 0.91 (V)
	3	90	3	199	S = 318.8 + 1.60 (V)	S = 33.7 + 0.97 (V)
	3	180	2	209	–	S = 28.3 + 0.92 (V)
	3	365	1	466	–	S = 41.3 + 1.93 (V)
9-8	3	180	2	269	–	S = 32.2 + 0.87 (V)
9-7	3-2	28	1	145	–	S = 25.4 + 1.00 (V)
	3-2	90	2	202	–	S = 17.4 + 1.40 (V)
	3-2	180	4	272	S = 190.3 + 1.34 (V)	S = 39.3 + 1.03 (V)
	3-2	365	1	472	–	S = 28.9 + 1.06 (V)
7-6	2	28	1	33	–	S = 41.1 + 0.65 (V)
	2	90	2	202	–	S = 44.1 + 0.89 (V)
	2	180	3	291	[5]	S = 59.3 + 0.99 (V)
	2	365	3	474	S = 252.9 + 0.70 (V)	S = 27.0 + 1.05 (V)
6-5	2	90	1	204	–	S = 25.7 + 0.79 (V)
	2	180	3	287	S = 154.3 + 0.53 (V)	S = 36.4 + 0.89 (V)
5-4	2	180	1	293	–	S = 47.0 + 0.81 (V)
4-3	2-1	28	1	148	–	S = 28.9 + 0.90 (V)
	2-1	90	2	205	–	S = 68.8 + 0.80 (V)
	2-1	180	3	295	[4]	S = 47.4 + 0.85 (V)
	2-1	365	2	479	–	S = 23.7 + 0.86 (V)
3-2	1	90	3	205	S = −29.2 + 2.79 (V)	S = 35.1 + 1.00 (V)
	1	180	1	294	–	S = 21.1 + 0.87 (V)
	1	365	2	472	–	S = 34.4 + 0.92 (V)
2-1	1	90	2	210	–	S = 27.1 + 0.81 (V)
	1	180	3	300	[4]	S = 45.9 + 1.09 (V)
	1	365	3	481	S = 73.5 + 2.14 (V)	S = 22.9 + 0.95 (V)

[1] A minimum of three break bond tests are required to derive an equation.
[2] General equation for shear: $S = C + (\tan \Phi) V$

where: S = shear strength or shear resistance lb/in^2
C = cohesion, lb/in^2
tan Φ = coefficient of internal friction
V = normal stress, lb/in^2

[3] Sliding friction equations based on four tests of each specimen.
[4] Break bond results are erratic; correlation not possible.
[5] Only two break bond results obtained.

Table 29. – Upper Stillwater Dam concrete test placement core program (horizontally drilled cores) – compressive strength summary – RCC.

Age at drilling, days	Mix No. T-	Lift No.	Drill hole No.	Age at test, days	Unit weight, lb/ft^3	Compressive strength, lb/in^2	Modulus of elasticity, lb/in^2 x 10^6	Poisson's ratio, in/in
180	3	9	32	270	145.95	1,320	–	–
	3	9	33	270	147.45	4,740	3.50	0.14
	2	6	30	293	145.60	3,800	–	–
	1	2	29	299	146.20	2,460	–	–
365	3	9	49	457	148.32	4,370	3.14	.16

Table 30. – Upper Stillwater Day concrete test placement core program (horizontally drilled cores) – tensile strength summary – RCC.

Age at drilling, days	Mix No. T-	Lift No.	Drill hole No.	Age at test, days	Unit weight, lb/ft³	Tensile strength, lb/in²	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in
180	3	11-10	35	269	146.11	233	–	–
	3	10-9	34	270	144.36	236	–	–
	3	9	32	270	145.99	128	–	–
	3	9	33	270	147.98	371	–	–
	3	9	37	270	146.08	177	–	–
	2	6	30	293	144.80	225	–	–
365	3	9	49	457	147.99	352	3.72	0.14
	3	9	48	457	146.58	333	3.71	.14

Table 31. – Upper Stillwater Dam concrete test placement core program – compressive strength summary – facing concrete.

Age at drilling, days	Element No.[1]	Drill hole No.	Age at test, days	Unit weight, lb/ft³	Compressive strength, lb/in²	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in
90	[2]1-u.s.	17	132	144.50	6,430	–	–
180	5-u.s.	30	–	147.00	–	–	–
	10-d.s.	37	280	141.95	6,710	3.24	–
	[4]8-d.s.	36	281	142.01	5,330	3.17	–
	8-d.s.	38	281	145.25	6,700	–	–
365	9-u.s.	48	461	146.92	8,810	–	–
	7-u.s.	48	461	142.27	[5]7,290	3.60	0.18
	10-d.s.	49	462	143.28	6,920	–	–

[1] Elements No. 5 and 6 contain 40:60 cement to fly-ash ratio; all others contain a 50:50 ratio.
[2] u.s. = upstream.
[3] Sample is taken from a diagonal drill hole; all others are horizontal drill holes.
[4] d.s. = downstream.
[5] Sample was vacuum saturated prior to testing.

Table 32. – Upper Stillwater Dam concrete test placement core program – tensile strength summary – facing concrete.

Age at drilling, days	Element No.[1]	Drill hole No.	Age at test, days	Unit weight, lb/ft^3	Tensile strength, lb/in^2	Modulus of elasticity, lb/in^2 x 10^6	Poisson's ratio, in/in
180	[2]9 -u.s.	34	273	148.02	501	4.86	0.17
	9 -u.s.	35	273	147.72	504	4.57	.15
	7 -u.s.	32	274	142.26	433	4.17	.16
	7 -u.s.	31	274	147.24	436	4.81	.17
	5 -u.s.	30	–	144.90	–	–	–
	3 -u.s.	29	296	145.20	504	4.64	–
	[3]10 -d.s.	38	280	145.50	416	4.69	–
	10-8 -d.s.	33	273	145.05	395	4.59	.17
	8 -d.s.	37	281	140.92	361	3.98	–
	8 -d.s.	38	281	144.16	372	5.12	–
365	7 -u.s.	47	461	143.02	[4]467	4.13	.17
	10 -d.s.	49	462	143.93	510	4.21	.13
	8 -d.s.	49	462	140.17	488	–	–
	8 -d.s.	49	462	141.62	[4]450	4.30	.17

[1] Elements No. 5 and 6 contain a 40:60 cement to fly ash ratio; all others contain a 50:50 ratio.
[2] u.s. = upstream.
[3] d.s. = downstream.
[4] Sample was vacuum saturated prior to testing.

Table 33. – Upper Stillwater Dam concrete test placement – average strength and elastic properties summary – field cylinder test program.

Mix No. T-	Average age at test, days	Compressive strength, lb/in²	No. of tests	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in	Tensile strength, lb/in²	No. of tests	Modulus of elasticity, lb/in² x 10⁶	Poisson's ratio, in/in
				Roller compacted concrete					
1	28	1,590	8	–	–	155	3	–	–
1	90	2,240	6	–	–	–	–	–	–
1	180	[1]3,940	3	1.72	0.13	210	3	1.88	–
1	365	4,350	2	1.59	.16	215	3	2.05	0.11
2	28	1,370	9	–	–	105	2	–	–
2	90	2,170	6	–	–	–	–	–	–
2	188	[1]4,120	3	1.50	.17	220	3	1.45	–
2	365	4,490	3	1.62	–	215	2	–	–
3	28	2,210	8	–	–	130	3	–	–
3	90	3,780	6	2.10	.15	270	2	2.87	.15
3	210	[1]7,430	4	2.45	–	350	3	4.14	–
3	365	7,005	2	3.23	.16	400	2	3.80	.13
				Facing concrete					
4	1	520	6						
4	2	1,170	3						
4	7	3,010	7						
4	28	4,430	16						
4	90	6,500	2						
4	180	7,700	1						
4	365	6,200	1						
5	1	350	2						
5	2	905	2						
5	7	–	2						
5	28	3,860	5						

[1] Test specimens experienced drying during transportation.

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APPENDIX A

UPPER STILLWATER DAM

RCC TEST PLACEMENT – CONSTRUCTION

APPENDIX A

UPPER STILLWATER DAM

RCC TEST PLACEMENT - CONSTRUCTION

The Upper Stillwater Dam RCC test placement was constructed in August of 1981. The purpose of the test placement was to evaluate construction methods and equipment proposed for the dam and to gather additional information on mix designs for facing concrete and RCC. The 100-foot long, 11-foot high test placement was constructed with a cross section equivalent to the upper 11 feet of the dam in the spillway section. This 15-foot wide cross section will be widened to 30 feet in the dam to facilitate access for construction equipment. The test placement, shown on figure 35*, consists of five vertical upstream facing elements, five "stairstepped" downstream facing elements placed on a 0.6 to 1.0 slope, and 11 lifts of RCC. The test placement was constructed on an RCC leveling slab or "base slab" which was placed on bedrock and varied in thickness from 1 to 5 feet. One of the major objectives of the test placement was the evaluation of the laser-controlled, slipform paving machine used to construct the upstream and downstream facing elements of the dam. Of particular concern were (1) how well the laser could control alinement, (2) could the facing elements be placed on top of each other, and (3) could bond between elements be achieved? Cracking of the elements was not a concern at this time, as this problem could be handled by controlling where the cracking could occur, or by allowing the cracking to take place and grouting where required.

Construction of the test placement consisted of slipforming the upstream and downstream facing elements followed by two 1-foot-thick lifts of RCC. This sequence of placing was followed to completion. The facing elements were placed with a laser guided slipform paver supplied with concrete from a transit mixer. The RCC was mixed by a small pugmill at the batch plant. Placement of the RCC was accomplished by transporting the concrete in end dump trucks, dumping and spreading initially by a dozer and then a motor grader, and compaction by a smooth drum vibratory roller. Due to the size of the test placement, use of the motor grader was eliminated, and leveling of RCC was accomplished with a small dozer equipped with lasers to control lift thickness. The test placement was instrumented with thermocouples for the measurement of temperature rise of RCC for 1 year. The construction of the test placement is described on figures 1A through 17A.

Quality control procedures at the test placement consisted of monitoring batch weights of materials, observation of the number of roller passes made by the roller, control of lift thickness and alinement of facing elements, performing unit-weight and consistency tests of fresh concrete, and casting the test specimens for hardened concrete testing.

Equipment and Procedures Used in Test Placement

The equipment used for the construction of the test placement was typical of that which might be found on a highway construction project.

Concrete aggregate was manufactured from sandstone talus material removed from the left abutment of the dam. Selective borrow from the talus deposit was accomplished by using a Caterpillar 988C rubber-tired front-end loader to move the oversize boulders aside and haul the usable material to the crusher. This loader worked very well for the short haul distance between the deposit and the crusher. However, the solid bucket created a problem in that it picked up and transported a large amount of fine material, silt, clay, and organic substance, which contaminated the crusher fines which were to be used for sand. This problem could have been alleviated by the use of a slotted rock bucket which would have allowed the deleterious material to pass through the bucket and be left in place. A 42- by 32-inch Cedar Rapids jaw crusher was used as the primary crusher. The secondary crusher was a Symons cone which reduced all of the rock to a size smaller than 1⅞-inches. This crushing operation generated approximately 0.2 ton of sand per ton of talus rock handled. A vibrating screen was used to separate the coarse aggregate from the sand. Conveyors transported the materials from the screen to the stockpiles.

* Figure 35 is located with the main text of this report.

A rubber-tired front-end loader was used to transport sand and coarse aggregate from the stockpiles to the RCC batcher bins mounted above the batch plant. The RCC batchplant was a 10 yd^3 capacity Ross "Rustler" portable batcher. A swivel chute at the end of the discharge conveyor allowed dry concrete ingredients to be directed to the transit mixers for conventional concrete or to the pugmill for RCC.

Conventional concrete for the upstream and downstream facing concrete was mixed in two 8 yd^3 Challenge mixers. Even though the mixers were new and the mixing and discharging bins were clean, these mixers were not capable of discharging the low slump concrete (¼ to 1-inch slump) required by the slipform facing element machine. The very slow delivery of concrete from the transit mixer to the slipform machine caused considerable repair and undesirable hand finishing of the facing elements.

The upstream and downstream facing elements were placed using a Gomaco Commander III Slipform Curbing Machine. The forms for the upstream and downstream facing were designed by the Bureau and built by the Gomaco Corporation. These forms were quite large containing approximately 6 ft^3 per linear foot of travel in the downstream face and 4 ft^3 per linear foot in the upstream face. It appears that these forms should be moving continuously to reduce tearing of the concrete surface. Each form had four 3-inch diameter immersion vibrators fastened inside the forms. Each vibrator was separately controlled for vibration frequency. Location of these vibrators is critical to the density of the concrete and surface finish.

When RCC was mixed, the swivel chute on the batch plant was directed to a conveyor belt which fed the RCC mixer. This mixer was a Model S-60 Heatherington-Burner 180 ton per hour, 8,000-pound capacity pugmill. The pugmill was a horizontal tub, twin paddle, with a sliding bottom dump gate. The mechanical condition of the pugmill would allow only 1.75 yd^3 of RCC to be mixed. This batch size was effectively mixed in the pugmill mixer. This batch weight was approximately 7,000 lb, 1,000 lb less than the mixer capacity. The pugmill was evidently designed to mix asphaltic concrete.

The RCC was transported from the mixer to the test placement in two 10 yd^3 capacity end dump trucks. These trucks handled the RCC with only minor segregation when dumped. However, the trucks were not capable of spreading the RCC as it was deposited. The truck driver was inexperienced, making it difficult to ascertain whether the dumping difficulties were caused by the driver, the dump box, or the consistency of the mix.

The RCC was deposited in one large pile and spread to the desired thickness. The first spreader used was a Caterpillar 14E, rubber-tired motor grader. In addition to being too large for the small section, the rubber tires compacted the RCC causing the roller to bridge, restricting the ability to consolidate the RCC between the tire tracks.

The second spreader used was a Caterpillar D6 track-mounted dozer. This piece of equipment worked quite well but was much too large. Finally, the contractor furnished a Caterpillar D3 track-mounted dozer to spread the RCC. This piece of equipment worked quite well.

Line and grade of facing elements were controlled with a Blount Instrument's class IIIB laser-guidance system. Two of these instruments were used, one for line and one for grade.

Two laser receivers were mounted on each end of the dozer blade to control the lift thickness of the RCC. Over and under indicators were mounted in view of the operator, and he controlled the blade height manually. This system was quite effective after the dozer operator became confident in the use of equipment.

Four laser receivers were mounted on the slipform machine, two for line and two for grade. The electric-hydraulic hookup offered complete laser control of the machine. The operator was only required to ensure that a steady supply of concrete was available in the form. Again, after the operator gained confidence in the use of the lasers, the equipment worked quite well.

A 10-ton Raygo 404B smooth drum vibrating roller was used to compact the RCC. This was a single drum roller with rubber tires for guidance. This type of roller was probably capable of adequately compacting the RCC mixture used in the test section. However, due to operator difficulties. it was not possible to completely evaluate the roller, because we were unable to determine the number of roller passes required to compact the RCC. No definite pattern was ever established for rolling the RCC. Indication of inadequate compaction and thick lifts were noted in the core samples taken from the test section.

No concerted effort was made to clean construction joints, and only an occasional attempt was made at high-pressure water blasting some joints. A model 002815729 Giant/Bosch water blaster was used in these attempts.

Figure 1A. – Laser transmitter used for line and grade control of slipform paving machine for upstream and downstream facing elements. P801-D-80877

Figure 2A. – Laser receivers mounted on slipform paver for line and grade control of the facing elements. P801-D-80878

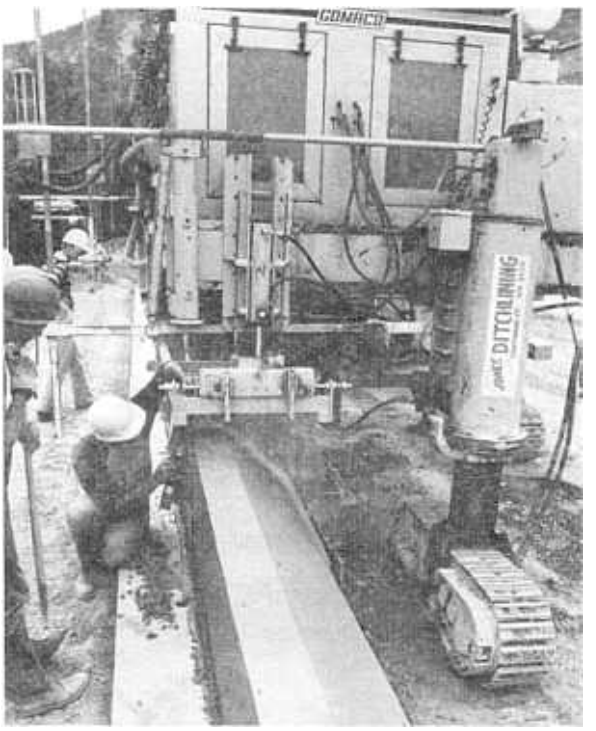

Figure 3A. – Vertical upstream facing element being placed by laser controlled slipform paving machine. Note laser receivers for line and grade control mounted on slipform paver. P801-D-80879

Figure 4A. – Completed vertical upstream facing element placed by laser controlled slipform paving machine. P801-D-80880

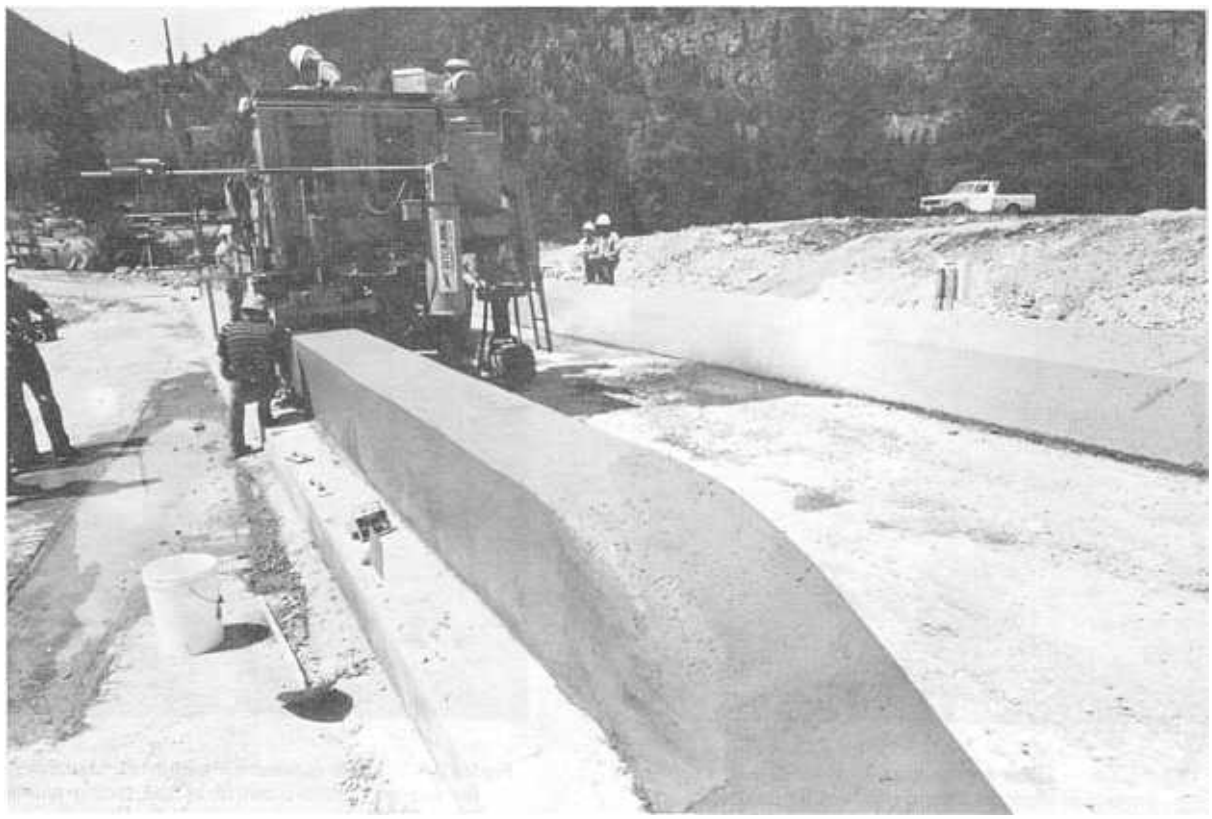

Figure 5A. – Downstream facing element being placed by laser controlled slipform paving machine. P801-D-80881

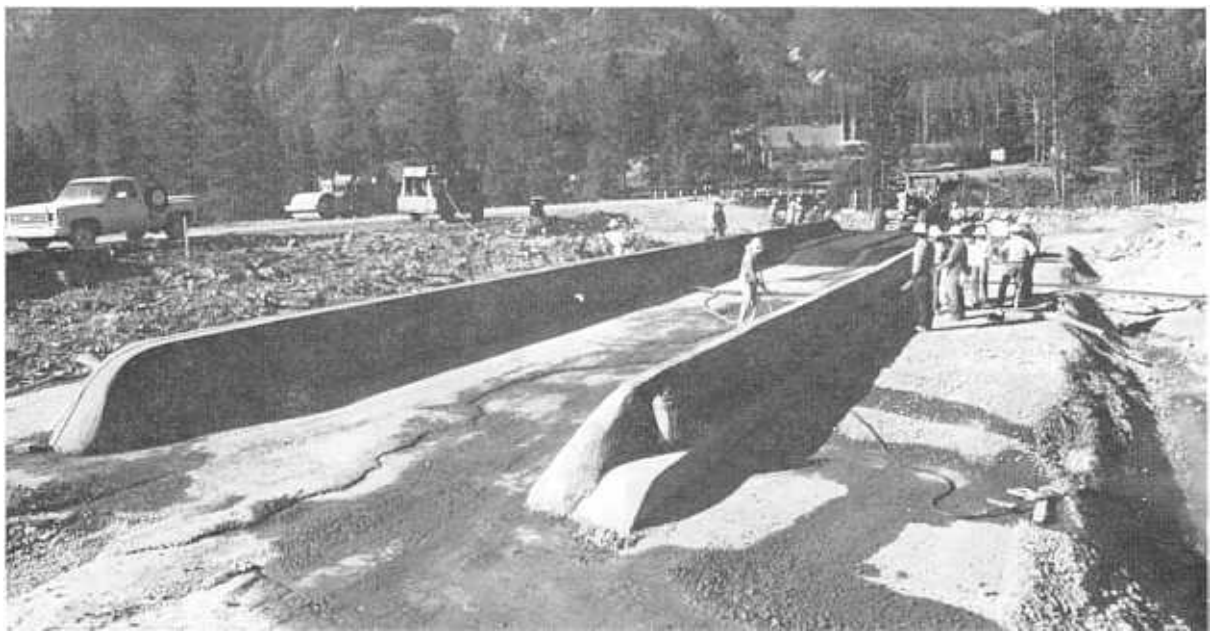

Figure 6A. – Three-foot-high upstream and downstream facing elements in place prior to placing two 1-foot-thick lifts of RCC. Facing elements serve as forms for RCC. P801-D-80882

Figure 7A. – Pugmill mixing and depositing RCC into end dump truck. P801-D-80883

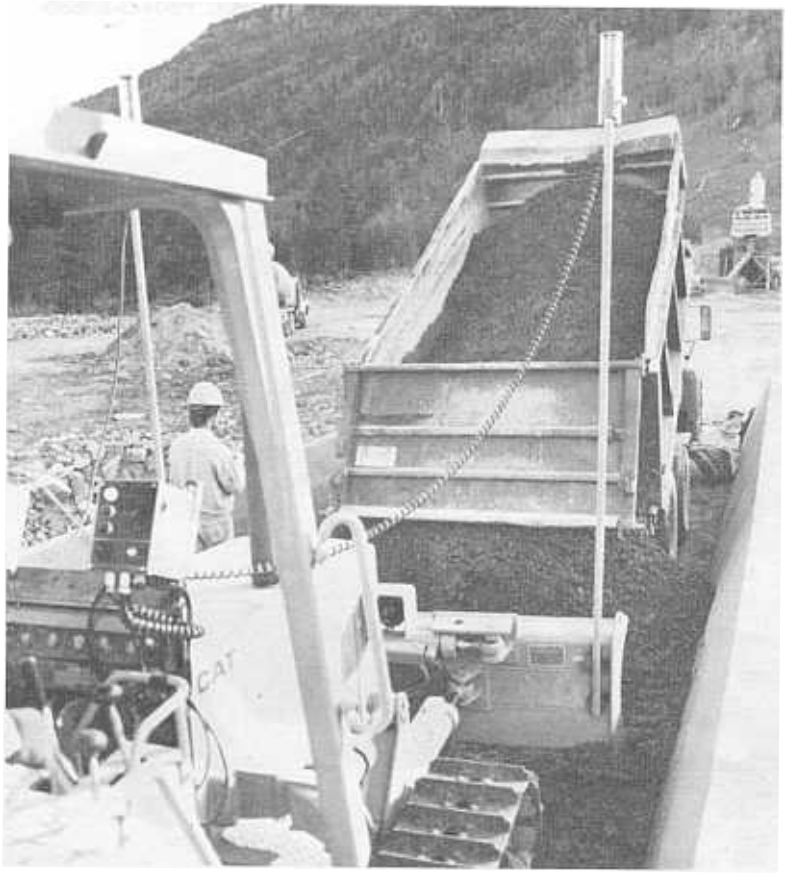

Figure 8A. – RCC being deposited by end dump truck. Dozer is spreading RCC; note laser control on dozer. P801-D-80884

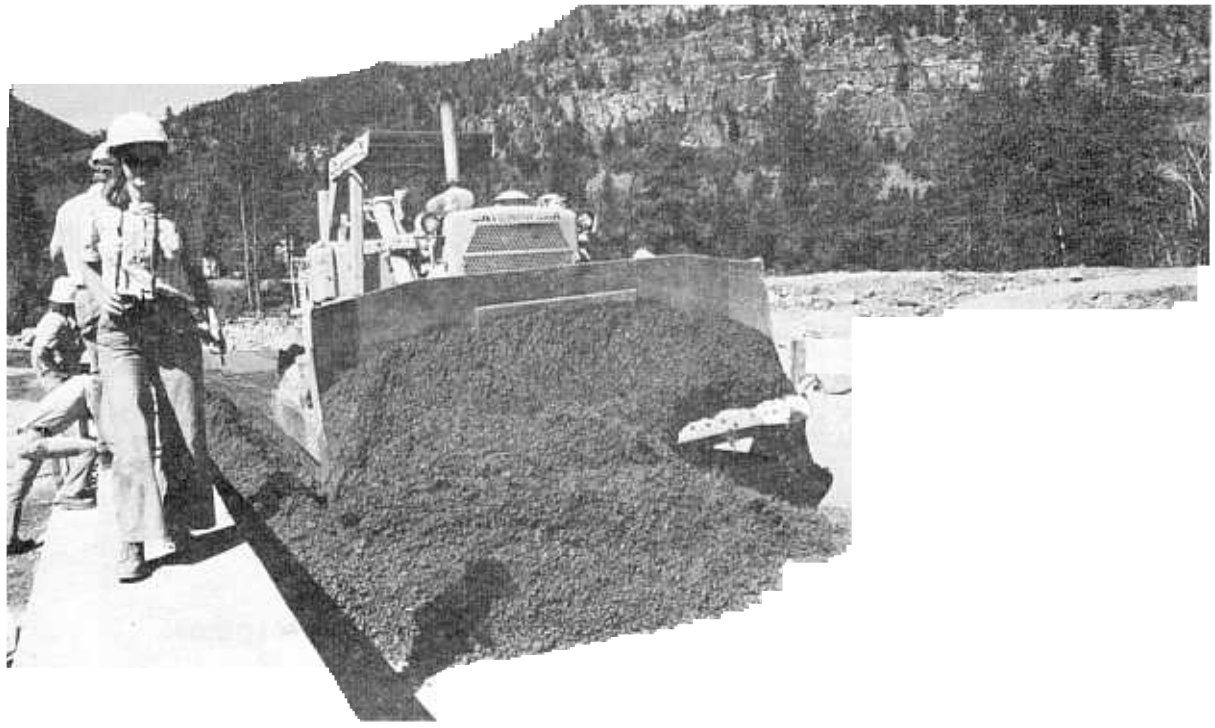

Figure 9A Dozer initially used for spreading RCC. P801-D-80885

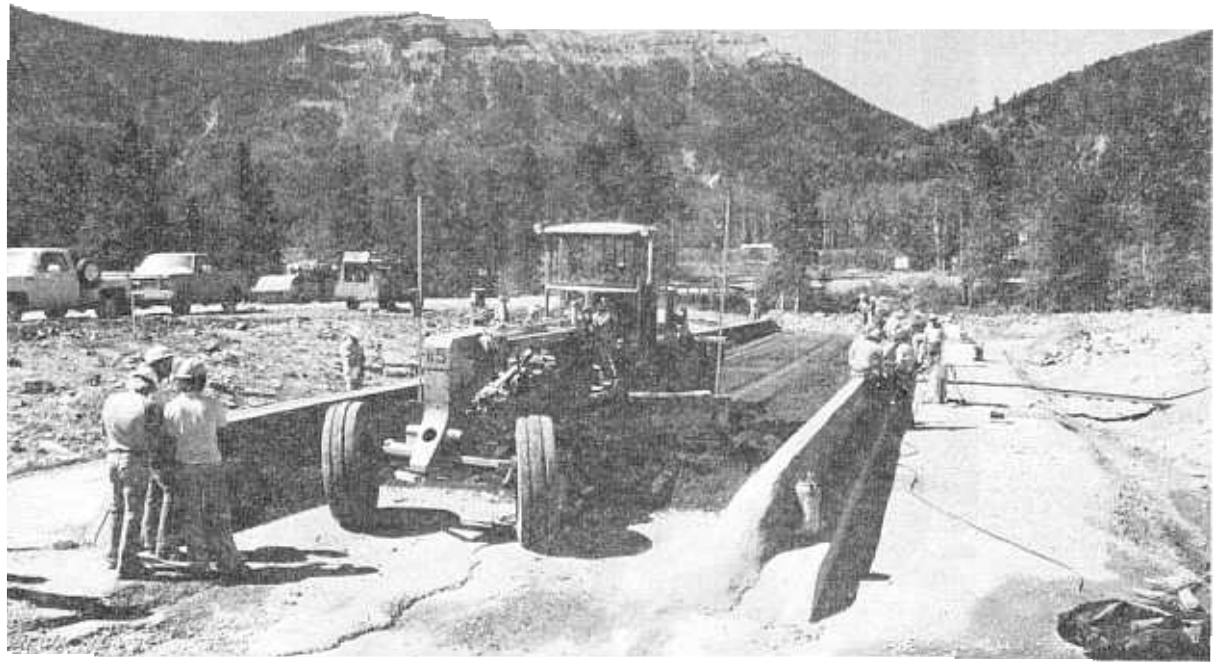

Figure 10A. – Motor grader with laser controls initially used for spreading RCC to final lift thickness. This was discontinued due to the size of the work area and precompaction of loose RCC by tires of grader. P801-D-80886

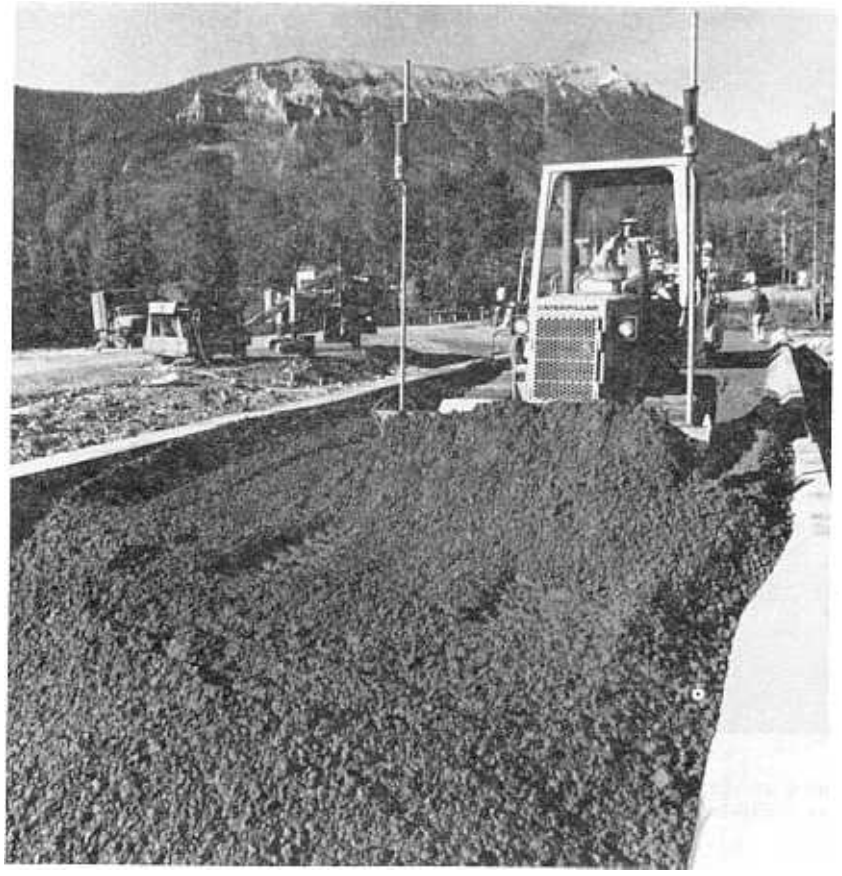

Figure 11A. – Dozer used for spreading RCC to final grade prior to compaction. Note laser receivers mounted on blade to control lift thickness. P801-D-80887

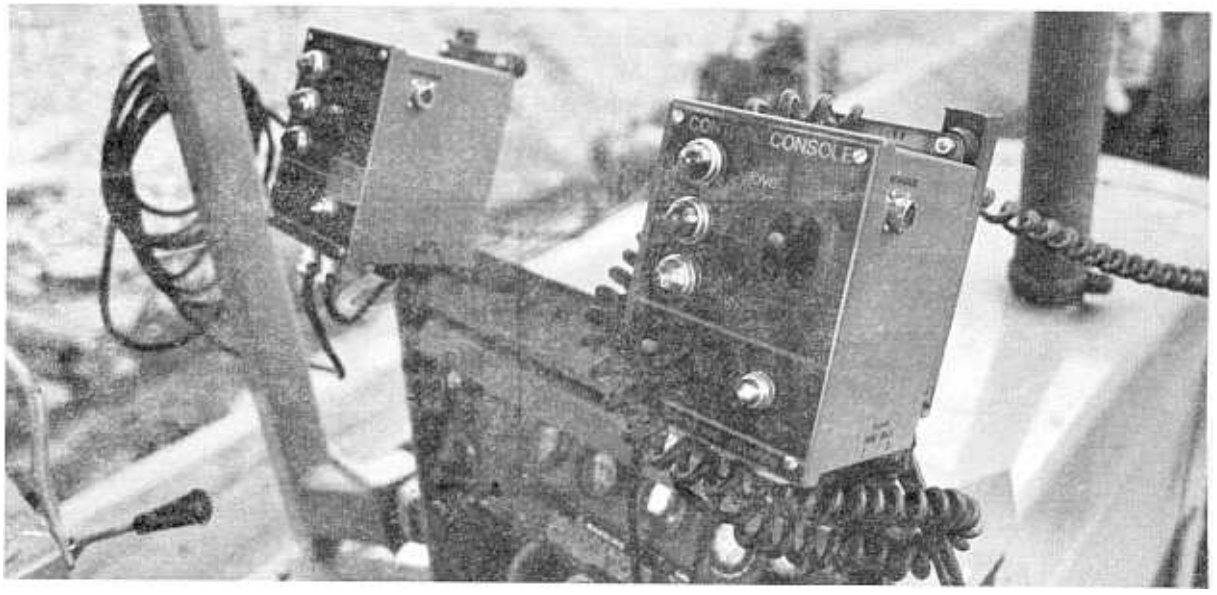

Figure 12A. – Laser controls installed on dozer to control lift thickness of RCC. P801-D-80888

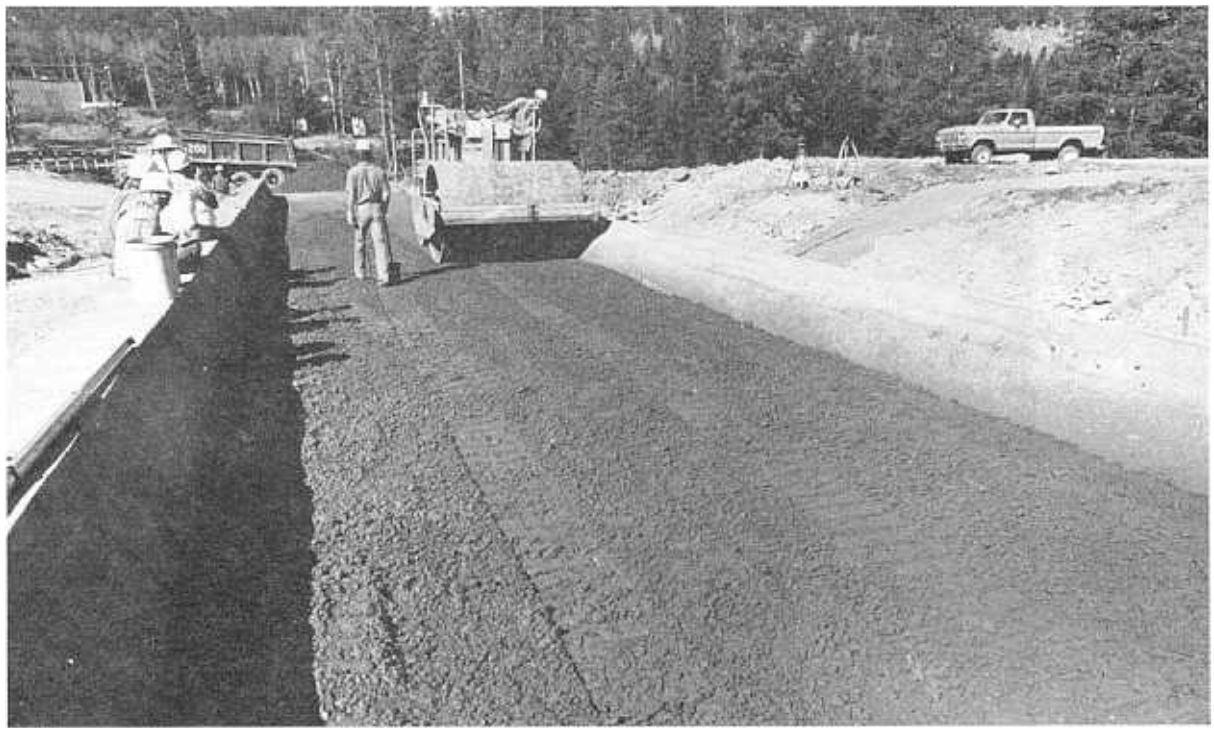

Figure 13A. – Smooth drum vibrating roller used for compaction of RCC. Note that roller is capable of operating adjacent to facing elements. P801-D-80889

Figure 14A. – Smooth drum vibrating roller making second pass over RCC. P801-D-80890

Figure 15A. – First lift of RCC after compaction by vibrating roller. P801-D-80891

Figure 16A. – Vertical upstream face of test placement. Slipform paver is placing the final upstream facing element. P801-D-80892

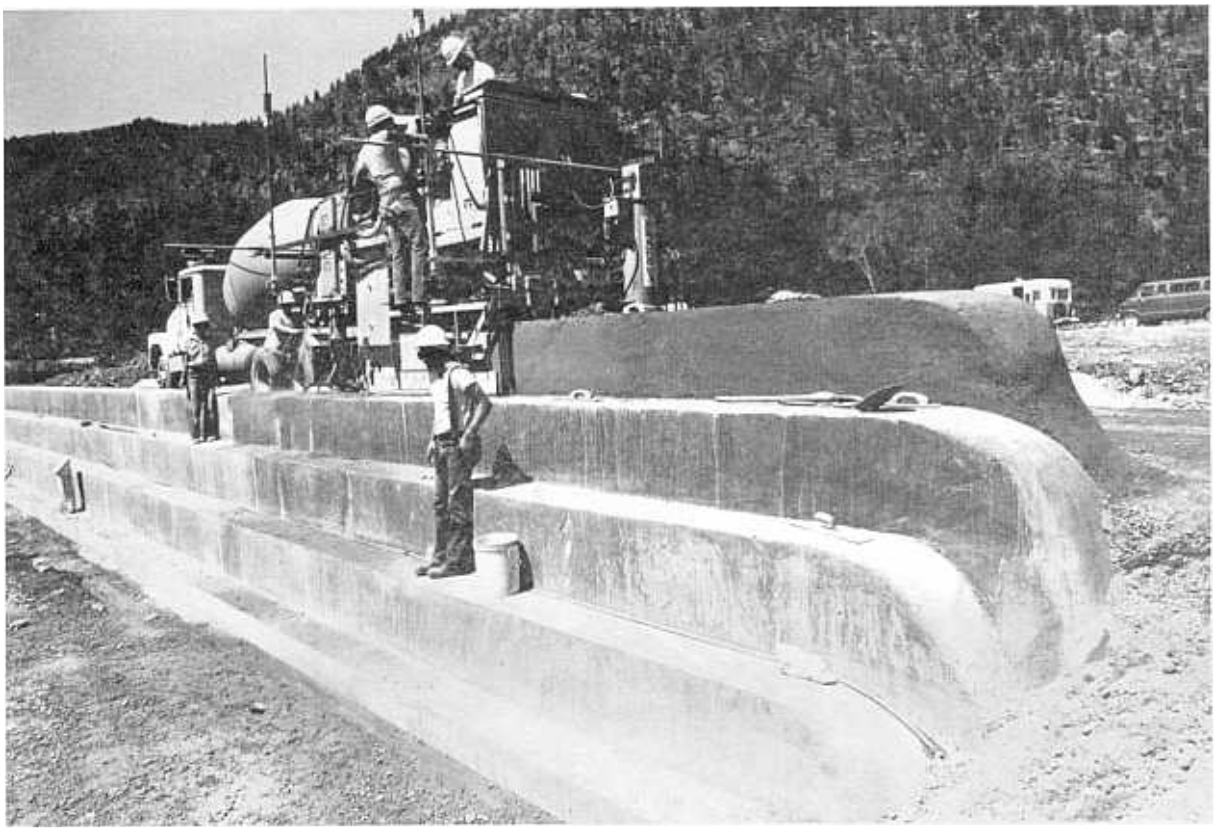

Figure 17A. – Stairstepped downstream face of test placement. The stairstep design will be used in the dam spillway section for energy dissipation. P801-D-80893

APPENDIX B

UPPER STILLWATER DAM

LABORATORY MIX PROGRAM

ADDITIONAL STUDIES

APPENDIX B

UPPER STILLWATER DAM

LABORATORY MIX PROGRAM

ADDITIONAL STUDIES

Early Strength Studies

As discussed in the laboratory mix design program, the adiabatic temperature rise tests indicate that very little heat is generated during the initial 24 to 36 hours after mixing. In an effort to evaluate the early strength gaining potential of RCC under different ambient conditions, a study was conducted to determine the compressive strength under 50 °F, 73 °F, and adiabatic curing temperatures. Test specimens from mix L-3, cast in plastic molds, were sealed and placed in temperature-controlled calorimeter rooms. Two rooms were maintained at 50 °F and 73 °F, respectively, while a third room began at 50 °F (the specified placing temperature for the dam) and was adjusted up to 87 °F in accordance with adiabatic curve established for mix L-3 with an ASTM type A WRA (fig. 21)*. Tests were performed at 2, 3, 7, 14, and 28 days' age. The results of this test program are shown in table 1B and on figure 1B. No tests could be performed at 1 day due to the low strength of the RCC.

The 73 °F cure produces the highest early strengths, followed by the adiabatic and 50 °F cure. The adiabatic cure produces the higher strengths, beginning at an age of 5 to 6 days. The 50 °F cure produces a very slow strength gain through 28 days, amounting to only 30 percent of adiabatic 28-day strength. It is very difficult to establish what temperature cycle the RCC will be subjected to, depending on the work schedule proposed by the contractor for the dam. It is expected that during spring and fall, the RCC may be subjected to ambient temperatures less than 50 °F until the lift is insulated by lifts placed above it. Following this period, the adiabatic condition will be maintained. If an exposed lift of fresh RCC is cooled overnight, there exists the potential to actually begin the adiabatic condition at a lower starting temperature which would further lower the early strength. This may not be considered detrimental to the RCC due to the possible reduction in thermal cracking in the dam.

Preliminary Studies – Accelerated Strength Testing of RCC with High Replacement of Fly Ash

One major concern of quality control of RCC is the larger volume of concrete placed during the time it takes to perform standard 28-day strength tests of concrete. It is estimated that as much as 300,000 cubic yards of RCC could be in place before the first 28-day cylinder is tested. In order to estimate the strength gaining

Table 1B. – Laboratory mix L-3 – effect of curing temperature on compressive strength development – Upper Stillwater Dam – RCC.

Temperature condition	Compressive strength, lb/in²				
	Age (days)				
	2	3	7	14	28
Adiabatic	280	445	960	1,705	3,095
73 °F	–	–	825	1,160	[1]1,785
50 °F	100	250	525	820	1,080
73 °F	350	–	1,110	–	[2]1,620

[1] Actual test mix
[2] Laboratory mix program

* Figure 21 is located with the main text of this report.

potential of RCC and make quicker batching adjustment, accelerated strength curing of concrete is required. Three such methods have been proposed: (1) warm water curing (95 °F) for 24 hours, (2) boiling water curing for 3½ hours, and (3) autogenous curing for 48 hours. These three methods do not readily apply for the mix designs developed for Upper Stillwater Dam because of the high percentages of fly ash. Preliminary tests with mortar cubes indicated that sufficient strength gain did not develop under a 95 °F warm water cure. This would also apply for an autogenous curing, especially with the 50 °F placing temperatures. There has been concern expressed over the use of boiled water curing with fly ash, due to high temperatures causing additional chemical reactions to occur which would not normally occur in the expected curing conditions at the dam.

In an effort to determine the minimum curing temperatures required for accelerated curing, additional testing was performed at 130 °F temperatures. Concrete specimens were cast in plastic molds according to standard procedures developed in the mix program and sealed prior to curing. Specimens were tested at 1, 2, and 3 days under the 130 °F cure and at 7, 28, and 90 days under a standard fog-cure. Tests were performed on mix L-3 with laboratory standard fly ash M-6498 and a sample of fly ash from the source used at concrete test placement. The results are summarized in table 2B.

Based on these test results, the compressive strength of accelerated-cured specimens equals the compressive strength of 28-day fog-cured specimens at an average age of 54 hours. Though this would be sufficient for standard concrete placements with 28-day design strengths, it would probably not be sufficient for mass concrete placements with 1-year strengths that are approximately three times greater than the 28-day strength. If accelerated curing is to be used as a quality control test method, either the curing

Table 2B. – Additional studies – accelerated compressive strength testing – Upper Stillwater Dam – RCC.

Batch	Age, days	Compressive strength, lb/in²	1-day accelerated strength, percent of			2-day accelerated strength, percent of			3-day accelerated strength, percent of		
			[1]7-day strength	28-day strength	90-day strength	7-day strength	28-day strength	90 day strength	7-day strength	28-day strength	90-day strength
[2]1	1	620	66.0	36.7	18.9						
	2	1420				151.1	84.0	43.3			
	3	1975							210.1	116.9	60.2
	7	940									
	28	1690									
	90	3280									
[3]2	1	560	56.6	29.0	12.8						
	2	1760				177.8	91.2	40.3			
	3	2820							284.8	146.1	64.5
	7	990									
	28	1930									
	90	4370									
[3]3	1	460	57.5	27.4	12.5						
	2	1510				188.8	89.9	41.1			
	3	2325							290.6	138.4	63.4
	7	800									
	28	1680									
	90	3670									
Average			60.0	31.0	14.7	172.6	88.4	41.6	261.8	133.8	62.7

[1] Strength based on standard fog cure
[2] Mix L-3 – Laboratory standard fly ash – M-6498
[3] Mix L-3 – Western Wyoming fly ash

temperature will have to be increased or the age of testing lengthened. A 72-hour cure at 130 °F would be considered the minimal length of cure. Testing should be performed at temperatures greater than 130 °F to determine if higher strengths can be achieved without affecting the chemical composition of concrete. Additional testing should also be performed to determine the effect of initial placing temperature on the accelerated strength.

Density of Fresh Concrete – Nuclear Densometer Versus Sand Cone Density

Two possible methods have been proposed to measure the in-place density of RCC: the conventional sand cone density and the nuclear density gage. A small scale test placement was constructed to evaluate the in-place density of RCC using these two test methods. A 4- by 5- by 1-ft RCC slab was constructed in two lifts using a plate vibrator to consolidate the fresh concrete. The fresh concrete density of 144.4 lb/ft^3 was computed using the Vebe test. The in-place density was measured at four locations with the nuclear density gage and in one location with the sand cone (figs. 3B to 6B). Density measurements of drilled cores were performed after 28 days' age. The results of nuclear gage tests of this test placement are given in table 3B. These results were not adjusted for chemical composition effect, which, due to the silicious composition of concrete materials, would increase the readings by approximately 2 lb/ft^3.

The sand cone density was found to be in excess of the theoretical maximum density of the RCC. This was caused by the RCC squeezing into the hole after it was dug causing the volume occupied by the removed concrete to be reduced. This same problem occurred in sand cone density tests attempted at the Upper Stillwater test placement. The average density by nuclear gage tests of the 8-inch depth of 142.8 lb/ft^3 was 98.9 percent of Vebe density, 97.3 percent of the theoretical density, and 97.6 percent of the density of saturated, drilled cores. These results should also be adjusted to account for the differences in density arising from chemical composition effects of the materials. In the case of the silicious materials used for the RCC, the nuclear density should be increased 1 to 2 lb/ft^3, thus bringing the density closer to the theoretical maximum density. The density of the cores is probably slightly higher than the actual density because small voids, which occupy concrete volume, do not show up as volume when the density is measured by weight and displaced volume in water. The in-place density of the RCC would probably be between 145 and 145.5 lb/ft^3 which would put the density by nuclear gage within 2 percent of the actual density. It does not appear that the sand core density is a viable alternative for in-place density, with the exception of a lift which is more than 12 to 18 hours old, when the squeezing will not occur. This, however, would not fit into the construction schedule, nor would it allow the RCC to be further compacted if it did not meet specification limits. The nuclear gage appears to be the only alternative at this point.

Table 3B. – Additional laboratory studies – nuclear density gage, block No. 1 Upper Stillwater Dam – RCC.

Test	No. of readings	Average wet density, lb/ft^3	Standard deviation, lb/ft^3
All tests	26	142.5	2.0
Backscatter	9	140.9	2.25
Probe:			
2-inch depth	1	144.7	–
4-inch depth	4	143.6	2.3
6-inch depth	2	144.3	–
8-inch depth	10	142.8	0.8

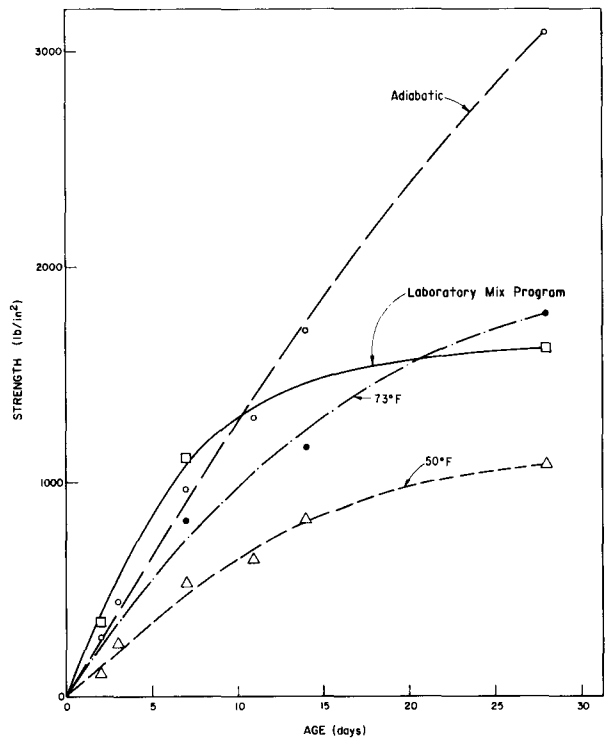

Figure 1B. – Effect of curing temperature on compressive strength gain – laboratory mix L-3 – RCC.

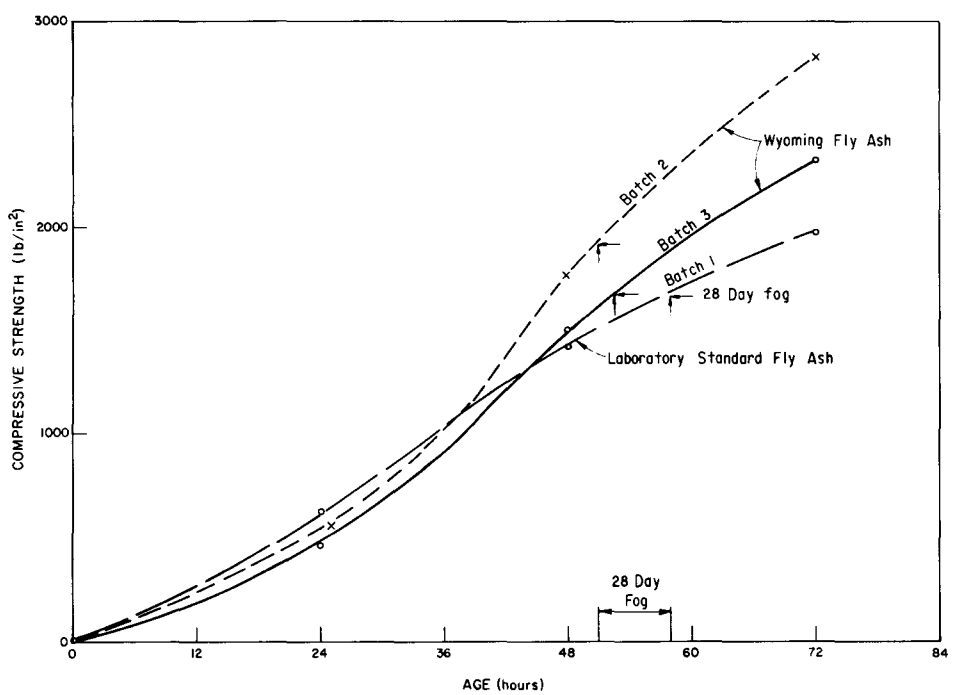

Figure 2B. – Laboratory mix program – accelerated curing at 130 °F, RCC.

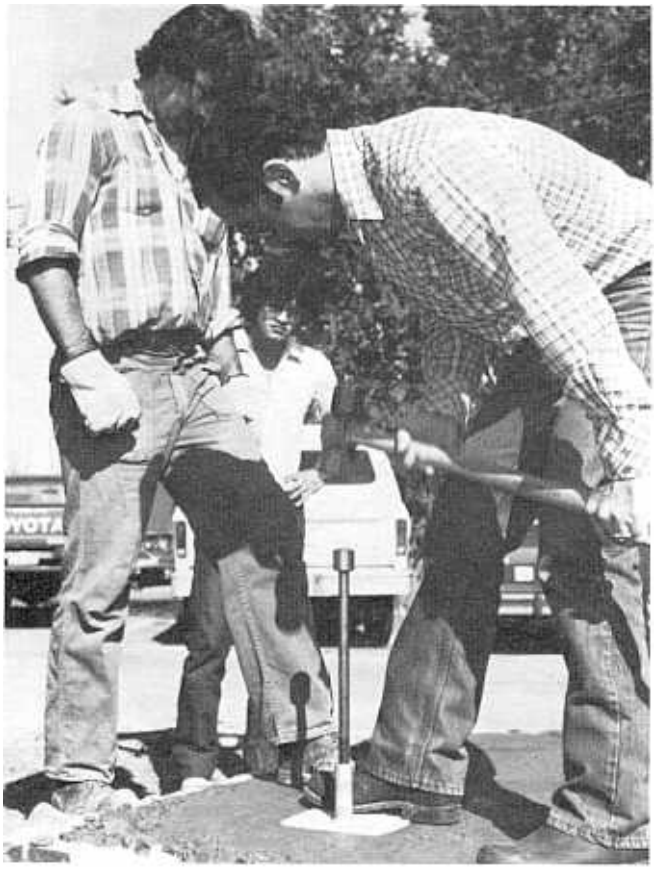

Figure 3B. – In-place density measurement of RCC. Guide plate and hole forming device used to drive hole for nuclear gage probe. P801-D-80894

Figure 4B. – In-place density measurement of RCC. Density reading being taken with the nuclear density gage. P801-D-80895

Figure 5B. – In-place density measurement of RCC. Hole being dug for sand cone density. P801-D-80896

Figure 6B. – In-place density measurement of RCC. Density of RCC being measured by sand cone methods. P801-D-80897

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Mission of the Bureau of Reclamation

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

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

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

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