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STEWART MOUNTAIN DAM CONCRETE CORE ANALYSIS-1983

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16. ABSTRACT

This report is an evaluation of tests made on 6-inch-diameter concrete cores taken from Stewart Mountain Dam near Phoenix, Arizona. Although the physical properties testing indicates that most of the concrete is above average in strength, disbonding of construction joints and rock mortar interfaces is quite obvious, caused by movement within the dam coupled with alkali-aggregate reaction. Cracking is more obvious in the upper 30 feet of the dam. Much of the testing performed on the specimens was under rapid loading conditions to simulate earthquake movement.

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STEWART MOUNTAIN DAM
CONCRETE CORE ANALYSIS – 1983

by

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May 1984

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Denver, Colorado

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UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

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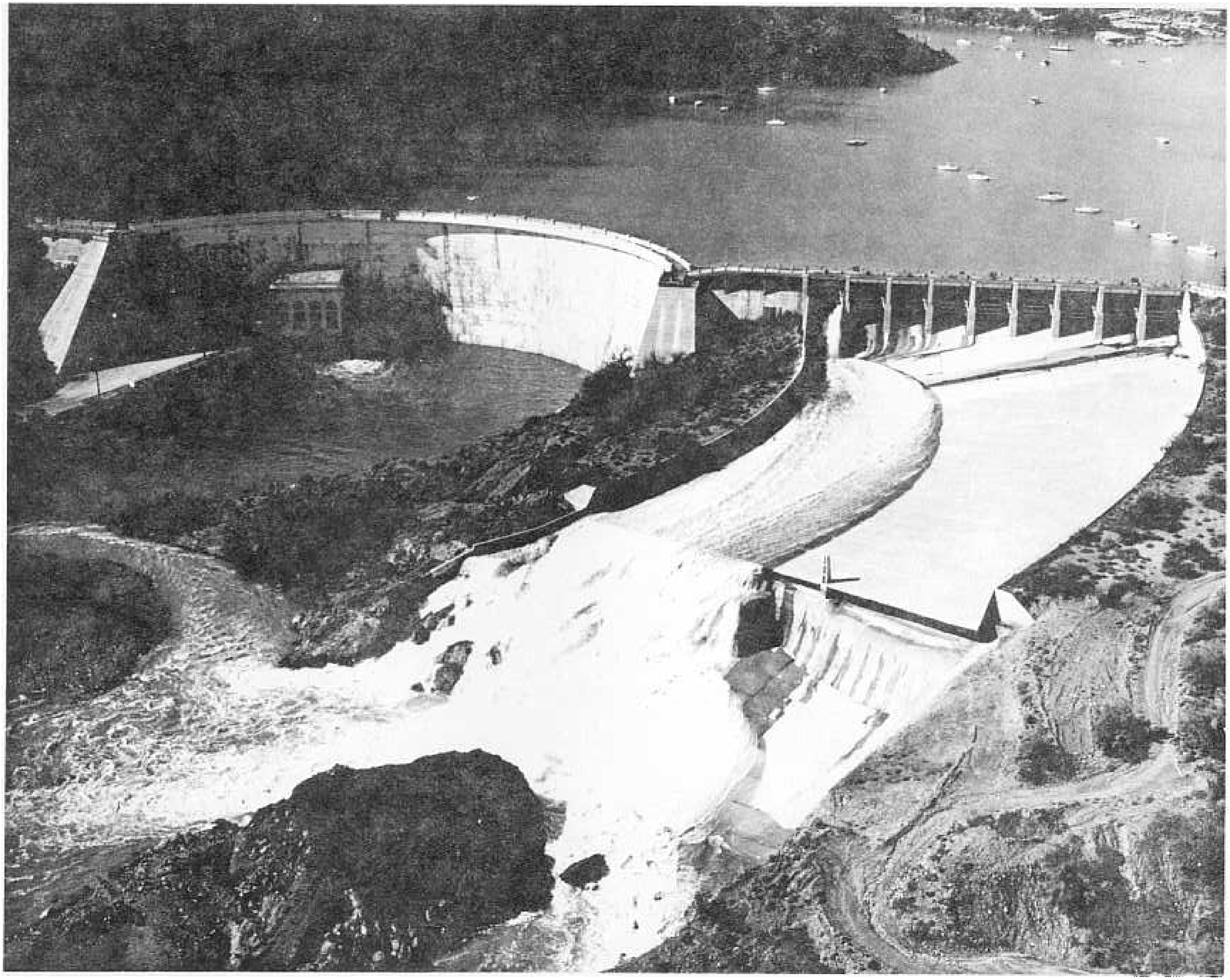
Appreciation is extended to Gloria J. Sheldon of the Applied Sciences Branch for the petrographic analysis (appendix A). Gratitude is also extended to Fred A. Travers for instrumenting the rapid load portion of the test program. A description of the instrumentation system used is included as appendix B.



Frontispiece.–Stewart Mountain Dam. P801-D-80457

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INTRODUCTION

Stewart Mountain Dam is located 41 miles northeast of Phoenix, Arizona. The dam creates a reservoir with a capacity of 70,000 acre-feet. Built by the Salt River Valley Water Users' Association in 1928-1930, the concrete thin-arch structure is 207 feet high with gravity abutments and has an open superelevated channel spillway equipped with radial gates.

This report includes the results of testing approximately 280 feet of 6-inch-diameter concrete core extracted from Stewart Mountain Dam during the spring of 1982 and is part of our continuing effort to monitor the quality of the concrete in this structure. This recent test program included rapid compression and rapid splitting tensile measurements of load capacity, and in the case of the compression tests, strain capacity. These tests were incorporated into the program to simulate earthquake conditions.

Although reference is made to prior test results, previous testing (1977 and before) did not include this rapid loading condition, and direct comparison of the tensile and compressive tests to historical data is not significant. For a detailed time-series comparison of previous cores, refer to Dunstan's report.[1]

CONCLUSIONS

Based on present and previous results from testing and evaluating the cores, the following conclusions are made:

1. "The construction records indicate a concrete mix with a high and wide-ranged, water-to-cement ratio. Good concrete quality control and placement practices by today's standards were not employed throughout construction."[1]

2. The average compressive strength for all specimens tested was 5,250 lb/in², which is above average for mass concrete of this age.

3. Average splitting tensile strengths seem high, at almost 500 lb/in². However, since this test has not been conducted in past core studies, a good basis for comparison is not available.

4. Specimens tested under rapid loading conditions, both compressive and splitting tensile, showed the concrete to be of consistently good quality.

5. Modulus of elasticity and Poisson's ratio are normal for mass concrete, averaging 4.0 x 10^6 lb/in² and 0.23, respectively.

6. "The movement of the dam since 1968 has been less than the potential indicated by laboratory measurements of expansions due to alkali-aggregate reactions . . ."[1]. "The potential for a continued alkali-aggregate reaction in the dam concrete still exists but is of low magnitude. This is substantiated by the fact that only three of the seven cores extracted in 1977 exhibited expansions in a 100-percent relative humidity environment. Additionally, these measured expansions were lower in magnitude than those of previous studies . . ."[1].

7. Based upon the continued reaction potential, monitoring of the dam through instrumentation should be continued.

8. Our standard method of core extraction and physical properties testing may never show us the real problem, if in fact there is one. An alternative may be the use of state-of-the-art in-situ testing.

The major concern with the concrete in Stewart Mountain Dam at this time is the disbonding of construction joints and rock-mortar interfaces caused by movement within the dam, coupled with alkali-aggregate reaction.

PHYSICAL TESTING PROPERTIES AND PROCEDURES

Cores in 1982 were taken from three different drill holes, two from the arch section and one from the left gravity abutment. Drill hole locations are shown on figure 1. All cores were 6 inches in diameter and all were drilled vertically. Specimens were tested in field moist conditions.

The cores were extracted in sections and packed in wooden boxes with damp sawdust. They were then marked with drill hole number and depth, and shipped to the Bureau of Reclamation, Engineering and Research Center, Denver, Colorado (fig. 2). After arrival, the cores were examined, logged, photographed, and marked for cutting into test specimens. The cores received had a large percentage of disbonded joints. All specimens, other than those set aside for direct shear tests, were sawed to 12-inch lengths. Direct shear specimens averaged about 4 inches in length.

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[1] Dunstan, E. R., Jr., "Stewart Mountain Dam Concrete Core Investigations – 1977," Bureau of Reclamation Report No. REC-ERC-79-14, November 1979.

COMPRESSIVE STRENGTH, ELASTICITY, AND POISSON'S RATIO

Cores taken from all three drill holes were tested for compressive strength, modulus of elasticity, and Poisson's ratio. Of the 35 specimens tested, 23 were tested under rapid loading conditions. Load was applied at a constant rate of strain that would fail the average specimens in 50 milli-seconds. The remaining 12 specimens were tested under standard loading procedures (2,000 lb/in² per minute). Under both loading procedures, all specimens were equipped with strain-gages, and strains were read continuously through the use of a data acquisition unit until the specimen failed . Modulus of elasticity and Poisson's ratio were determined over the proportional range.

The overall average compressive strength was 5,250 lb/in², with the rapid load test strengths averaging about 200 lb/in² higher than the standard load. The lowest strength obtained was 2,830 lb/in², under rapid load conditions. The highest strength occurred at 8,690 lb/in², under rapid loading conditions. Compressive strength results are summarized in table 1 and figures 3, 4, and 5.

The average modulus of elasticity for all tests was 4.0×10^6 lb/in², with no significant difference between rapid loading and standard loading. Results are summarized in table 1.

Poisson's ratios averaged 0.23 overall, with the standard loading rate averaging about 0.1 higher than the rapid loading procedure. Results are summarized in table 1.

TENSILE STRENGTH

Direct Tension

Only two specimens (DH-102) were tested in direct tension at the standard load rate of 200 lb/in² per minute. Both of these specimens included construction joints. The two specimens failed at 30 and 55 lb/in², much lower than either the rapid load splitting tensile or standard load splitting tensile strengths. However, this comparison is not valid since both direct tensile failures occurred across construction joints.

Splitting Tension

The overall splitting tensile strength average of the 41 specimens tested was 490 lb/in². Of these 41 specimens, 5 were tested under the standard loading conditions and averaged 340 lb/in². The 36 tested under rapid loading conditions averaged 520 lb/in². Many of the splitting tensile specimens failed around, rather than through aggregate particles, indicating poor bond strength which was probably caused in part by alkali-aggregate reaction. A few adjacent specimens were tested by rotating the specimen 90° relative to each other with no significant difference in results. Results of these tests are shown in table 2 and figures 6 through 9.

BREAK-BOND AND SLIDING FRICTION

Selected intact specimens were subjected to break-bond tests, followed by sliding friction tests. Most specimens were unbonded and these were tested using only the sliding friction test. Several of the specimens contained construction joints. The cores were cut into about 4-inch lengths with the joints as close as possible to midlength. Break-bond data were obtained by subjecting each specimen to a constant standard load while simultaneously increasing the horizontal shearing load until the specimen failed or broke bond (fig. 10). The maximum horizontal shear and constant standard loads were recorded. The failed surfaces were then subjected to a series of tests with increasing standard loads, where the shear loads required to displace the surface at each standard load were determined. The results of the break-bond and sliding friction tests are shown in table 3. Values for cohesion and the tangent of the angle of friction (tan $\emptyset$) are also shown in table 3. These direct shear test results showed no significant difference between drill holes or between upper and lower portions of the dam. Thirty-three specimens were tested, and the results were varied. Many of the samples were of poor quality with considerable amounts of material missing. Also, a large number of fracture and joint surfaces were coated with alkali silica gel.

PETROGRAPHIC ANALYSES

Petrographic analyses were performed on selected specimens from all three drill holes. These included some afterbreak samples of failed specimens. The concrete in Stewart Mountain Dam was found to be of fair petrographic condition. The weakening that has occurred is due primarily to alkali-aggregate reaction. A more detailed description of petrographic results is contained in appendix A of this report.

SUMMARY

The physical properties test results have not changed significantly over the years. As compared to Dunstan's report[1], elastic properties and compressive strengths remain about the same when compared to earlier cores that were also drilled vertically.

The potential for further expansion due to alkali-aggregate reaction exists throughout the dam (fig. 11). In 1977, selected core specimens were placed in a 100-percent relative humidity environment to induce this alkali-aggregate reaction.

> *"If concrete can potentially react, it will do so under this environment and usually expands due to the reaction. However, the 1977 cores produced a gel that exuded out of the concrete, but many of the cores showed little or no expansion. This is induced expansion; the potential expansion may or may not occur in the dam."*[1]

In an attempt to induce even more expansion, some of these same specimens are currently being subjected to wetting and drying cycles. It is too early to report any results, but if increased expansion does occur, this may help explain why there is more disbonding and cracking evident in the upper 30 to 35 feet of the dam. If no further expansion occurs, we must assume that movement within the dam had been more severe in this upper portion.

Table 1.–*Stewart Mountain Dam compressive strength, elasticity, and Poisson's ratio*

Drill hole	Depth from top (ft)	Compressive strength (lb/in²)	Modulus of elasticity (lb/in² x 10⁶)	Poisson's ratio	Remarks
DH-101	5.8	4420	4.39	0.18	rapid load
	9.4	3590	3.38	.25	standard load
	12.7	3880	4.64	.15	rapid load
	26.3	4980	3.91	.21	standard load
	28.8	4530	4.24	.22	rapid load
	36.0	3350	2.93	.34	rapid load
	37.7	3980	5.04	.28	standard load
	39.6	2830	3.68	.11	rapid load
	48.0	4910	2.85	.25	rapid load
	54.7	5510	3.82	.10	rapid load
	58.0	6670	5.05	.22	standard load
	72.0	6450	3.98	.28	rapid load
	73.3	6120	4.30	.32	rapid load
	77.1	6880	4.00	.04	rapid load
	80.8	4700	4.83	.56	standard load
	81.8	6510	3.66	.18	rapid load
	100.0	4650	4.13	.34	rapid load
	102.7	5870	2.95	.17	standard load
	104.5	4980	3.05	.10	rapid load
	107.9	5940	4.29	.17	rapid load
	116.9	4910	3.34	.43	standard load
	117.9	5040	2.94	.18	rapid load
	122.3	5440	4.01	.20	standard load
	124.4	5310	3.08	.16	rapid load
	137.2	6600	4.27	.20	rapid load
	147.7	4970	5.76	.27	standard load
	149.6	7660	4.90	.25	rapid load
DH-102	1.5	4170	4.23	.13	rapid load
	17.3	4180	4.09	.65	standard load
	34.5	3490	2.88	.18	rapid load
DH-103	6.9	5050	2.05	.14	standard load
	17.4	4070	4.65	.38	rapid load
	33.2	7160	5.52	.16	rapid load
	54.3	6260	2.23	.08	standard load
	70.3	8690	5.45	.16	rapid load
Average		5250	4.0	0.23	

Table 2.—*Stewart Mountain Dam tensile strength testing*

Drill hole	Depth from top (ft)	Splitting tensile strength (lb/in^2)	Time to break	Remarks
DH-101	2.5	455	102.4 msec	rapid load[1]
	4.2	385	61.6 msec	rapid load[1]
	7.5	270	1.4 min	standard load
	16.4	280	1.4 min	standard load
	18.2	540	43.3 msec	rapid load[2]
	21.0	505	59.1 msec	rapid load[2]
	24.0	475	52.4 msec	rapid load
	31.0	530	52.8 msec	rapid load
	32.5	400	2.0 min	standard load
	41.0	455	63.4 msec	rapid load[3]
	42.0	455	92.0 msec	rapid load[3]
	49.2	460	89.9 msec	rapid load[4]
	57.8	665	57.8 msec	rapid load[4]
	59.3	665	72.0 msec	rapid load
	63.8	555	50.0 msec	rapid load[5]
	65.2	700	79.1 msec	rapid load[5]
	75.1	650	60.0 msec	rapid load
	78.3	475	34.1 msec	rapid load[6]
	79.8	525	58.5 msec	rapid load[6]
	83.7	850	75.1 msec	rapid load
	87.0	335	1.7 min	standard load
	92.5	575	94.2 msec	rapid load[7]
	97.1	440	70.0 msec	rapid load[7]
	105.9	425	60.0 msec	rapid load
	109.7	320	78.1 msec	rapid load
	120.0	410	2.1 min	standard load
	126.7	425	54.4 msec	rapid load
	144.8	530	62.8 msec	rapid load
DH-102	2.5	510	50.0 msec	rapid load
	7.3	470	96.8 msec	rapid load
	13.9	390	41.5 msec	rapid load
	15.2	30	0.16 min	standard load[8]
	20.3	510	56.8 msec	rapid load
	24.6	285	57.2 msec	rapid load
	30.5	55	0.27 min	standard load[8]
	31.5	430	78.1 msec	rapid load
	36.8	430	60.0 msec	rapid load
DH-103	8.0	470	49.2 msec	rapid load
	24.2	485	*[9]	rapid load
	37.1	605	79.1 msec	rapid load
	55.3	740	65.0 msec	rapid load
	69.3	675	72.9 msec	rapid load
	87.4	410	40.0 msec	rapid load
Average		490		

[1] to [7] Split adjacent to each other
[8] Tested in direct tension, results not included in average
[9] Monitoring failure

Table 3.—*Stewart Mountain Dam shear and sliding friction tests*

Drill hole	Depth from top (ft)	Normal stress (N) (lb/in²)	Shear stress (S) (lb/in²)	Linear regression: Equation (lb/in²)	Linear regression: Phi (degrees)	Linear regression: Cohesion (lb/in²)	Remarks BB - Break bond SF - Sliding friction
DH-101	10.0	49	206				BB
		93	157				SF
		149	194				SF
		199	244	S = 77.8 + 0.82 (N)	39.3	77.8	SF
	15.0	50	118				SF
		99	176				SF
		150	234				SF
		200	286	S = 63.6 + 1.12 (N)	48.3	63.6	SF
	25.3	25	278				BB
		52	94				SF
		101	158				SF
		198	265	S = 37.6 + 1.16 (N)	49.1	37.6	SF
	35.3	50	160				SF
		101	257				SF
		150	342				SF
		200	338	S = 119.1 + 1.24 (N)	51.2	119.1	SF
	42.7	49	222				SF
		100	283				SF
		151	333				SF
		201	392	S = 168.6 + 1.11 (N)	47.9	168.6	SF
	51.1	50	50				SF
		98	100				SF
		153	149				SF
		201	181	S = 11.1 + 0.87 (N)	41.0	11.1	SF
	56.3	51	93				SF
		100	127				SF
		151	170				SF
		200	173	S = 69.5 + 0.57 (N)	29.6	69.5	SF
	60.3	50	73				SF
		100	122				SF
		152	151				SF
		202	185	S = 41.2 + 0.73 (N)	36	41.0	SF
	67.8	52	116				SF
		99	259				SF
		153	248				SF
		204	274	S = 111 + 0.89 (N)	42	111.0	SF
	90.8	50	124				SF
		100	200				SF
		150	214				SF
		200	255	S = 95.9 + 0.82 (N)	39	96.0	SF
	101.0	51	159				SF
		103	236				SF
		154	321				SF
		205	373	S = 90.7 + 1.41 (N)	55	91.0	SF
	111.3	51	126				SF
		101	170				SF
		151	236				SF
		200	277	S = 70.6 + 1.04 (N)	46	71.0	SF
DH-102	6.4	50	72				SF
		100	129				SF
		151	182				SF
		201	228	S = 22.7 + 1.04 (N)	46	23.0	SF

Table 3.—*Stewart Mountain Dam shear and sliding friction tests*—Continued

Drill hole	Depth from top (ft)	Normal stress (N) (lb/in²)	Shear stress (S) (lb/in²)	Linear regression Equation (lb/in²)	Phi (degrees)	Cohesion (lb/in²)	Remarks BB - Break bond SF - Sliding friction
	19.3	24	438				BB
		50	186				SF
		26	153				SF
		50	195				SF
		100	323				SF
		150	359	S = 109.4 + 1.78 (N)	61	109	SF
	25.8	52	78				SF
		101	130				SF
		150	150				SF
		200	200	S = 43.7 + 0.77 (N)	38	44	SF
	35.5	52	106				SF
		102	190				SF
		152	257				SF
		202	325	S = 36.2 + 1.44 (N)	55	36	SF
DH-103	2.9	52	179				SF
		101	244				SF
		151	282				SF
		202	330	S = 135.1 + 0.98 (N)	44	135	SF
	6.3	10	201				BB
		10	51				SF
		50	148				SF
		103	256				SF
		155	317				SF
		203	329	S = 66.6 + 1.47 (N)	56	67	SF
	16.7	49	34				SF
		100	70				SF
		149	103				SF
		199	136	S = 1.3 + 0.68 (N)	34	1	SF
	27.4	51	106				SF
		101	172				SF
		151	228				SF
		201	266	S = 58.6 + 1.07 (N)	47	59	SF
	31.3	32	28				SF
		50	41				SF
		100	85				SF
		201	151	S = 6.6 + 0.73 (N)	36	7	SF
	41.5	42	55				SF
		100	105				SF
		150	147				SF
		200	177	S = 25.5 + 0.78 (N)	38	25	SF
	43.7	44	87				SF
		100	149				SF
		150	188				SF
		200	232	S = 50.8 + 0.91 (N)	42	51	SF
	46.1	47	48				SF
		102	94				SF
		151	131				SF
		201	173	S = 10.3 + 0.81 (N)	39	10	SF
	59.3	62	74				SF
		102	119				SF
		151	152				SF
		201	202	S = 21.6 + 0.89 (N)	42	22	SF

Table 3.—*Stewart Mountain Dam shear and sliding friction tests*—Continued

Drill hole	Depth from top (ft)	Normal stress (N) (lb/in²)	Shear stress (S) (lb/in²)	Linear regression Equation (lb/in²)	Phi (degrees)	Cohesion (lb/in²)	Remarks BB - Break bond SF - Sliding friction
	62.8	66	150				SF
		101	183				SF
		151	248				SF
		202	278	S = 87.3 + 0.98 (N)	44	87	SF
	64.6	66	274				SF
		100	326				SF
		151	267				SF
		200	295	S = 292.7 – 0.02 (N)	1	293	SF
	67.2	69	326				SF
		100	322				SF
		151	338				SF
		201	333	S = 318.4 + 0.09 (N)	5	318	SF
	71.0	74	171				SF
		100	188				SF
		151	250				SF
		202	301	S = 89.3 + 1.05 (N)	46	89	SF
	73.8	76	204				SF
		100	225				SF
		152	300				SF
		203	299	S = 148.9 + 0.82 (N)	39	149	SF
	76.0	79	210				SF
		100	195				SF
		152	255				SF
		203	321	S = 113.1 + 0.99 (N)	45	113	SF
	85.7	87	234				SF
		101	248				SF
		152	305				SF
		203	331	S = 161.7 + 0.87 (N)	41	162	SF
	88.3	89	207				SF
		100	216				SF
		150	284				SF
		203	312	S = 123.8 + 0.96 (N)	44	124	SF

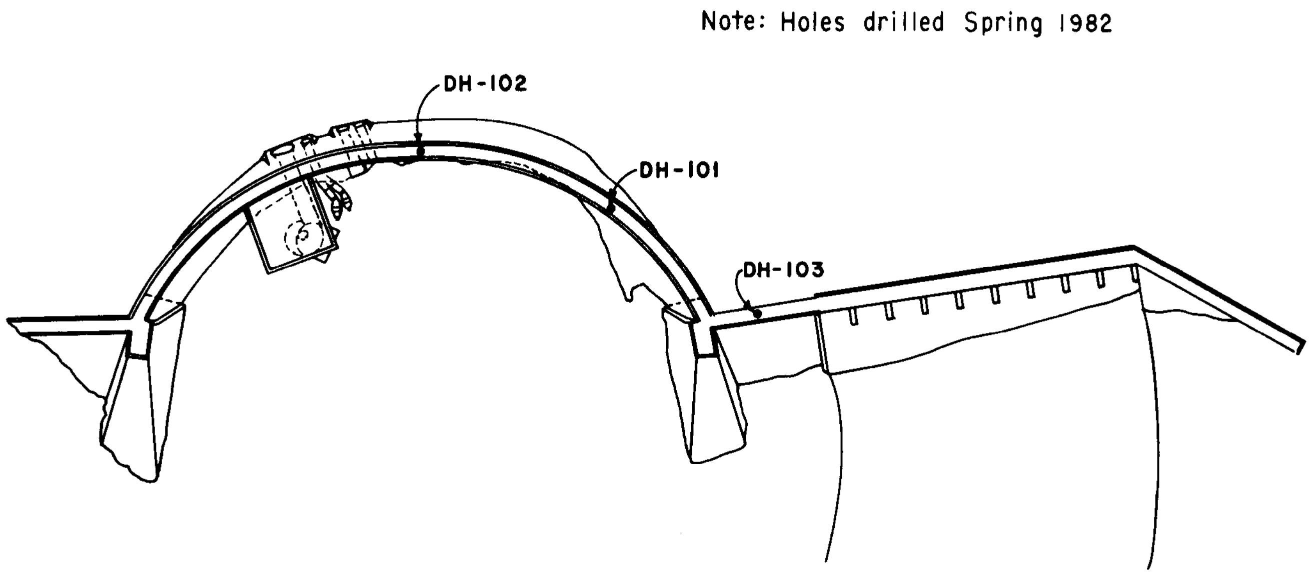


Figure 1.–Stewart Mountain Dam drill hole locations.

Figure 2.–Typical core sections before being cut into specimens, Stewart Mountain Dam. P801-D-80458

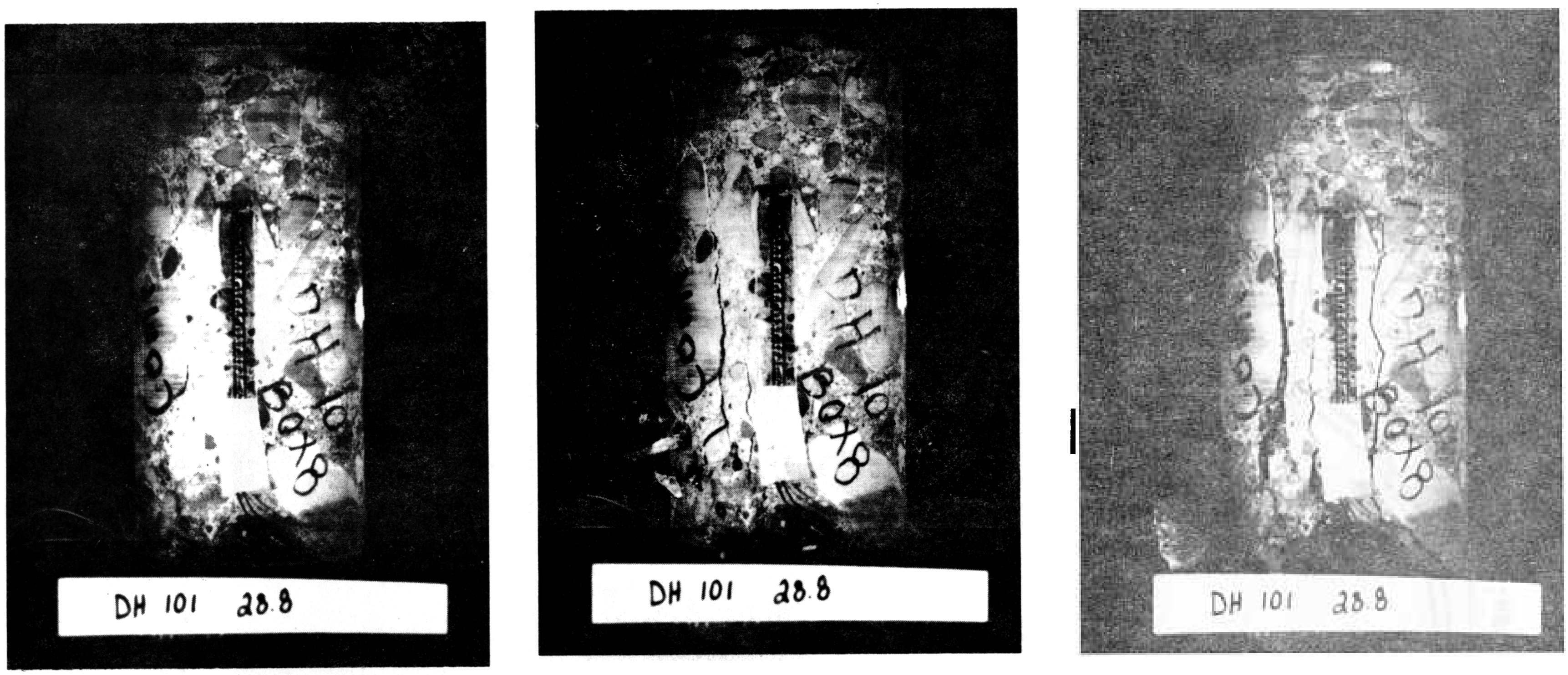


Figure 3.–Typical rapid load compression test specimens, Stewart Mountain Dam. P801-D-80459, P801-D-80460, and P801-D-80461

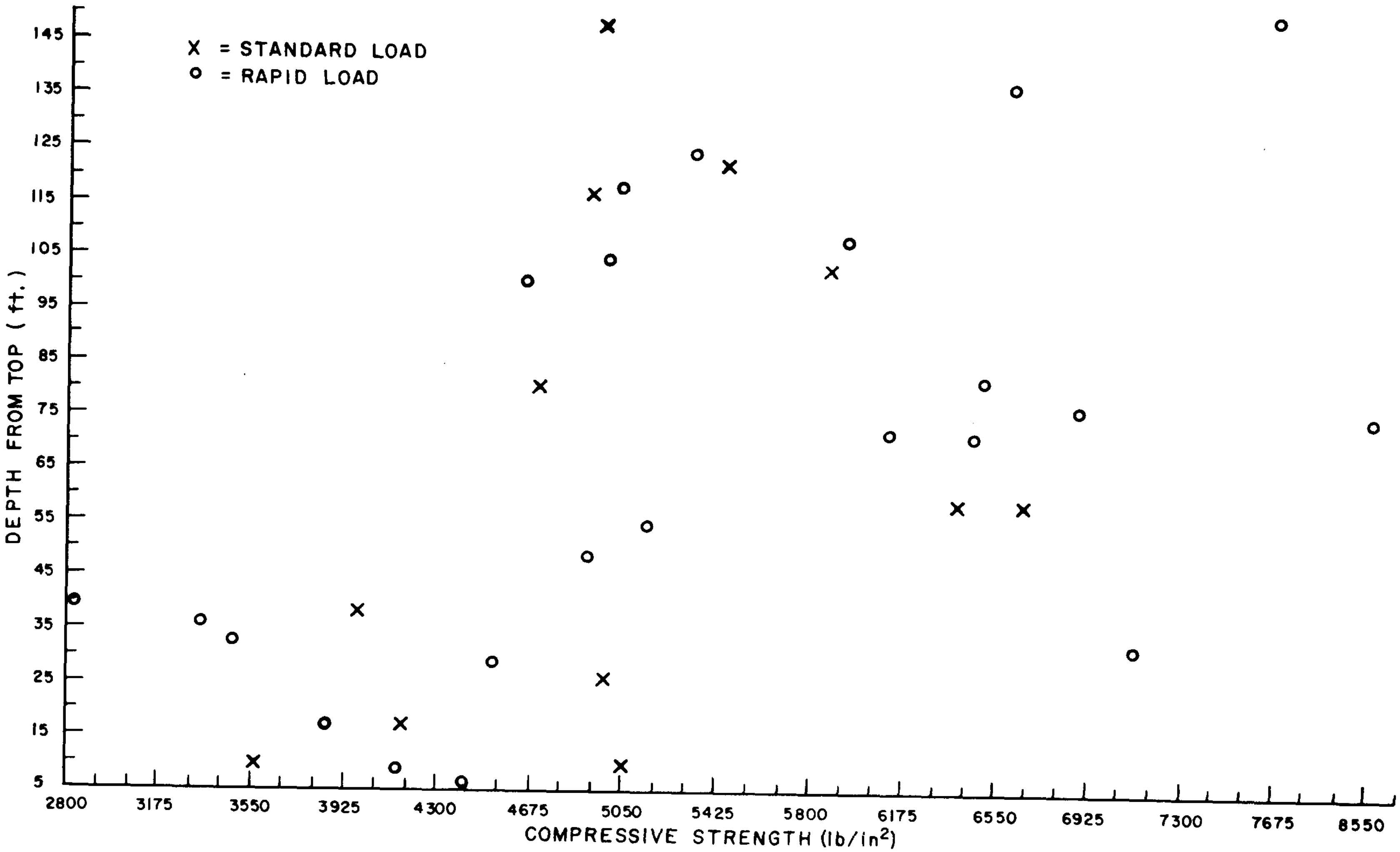


Figure 4.–Stewart Mountain Dam compressive strength versus depth.

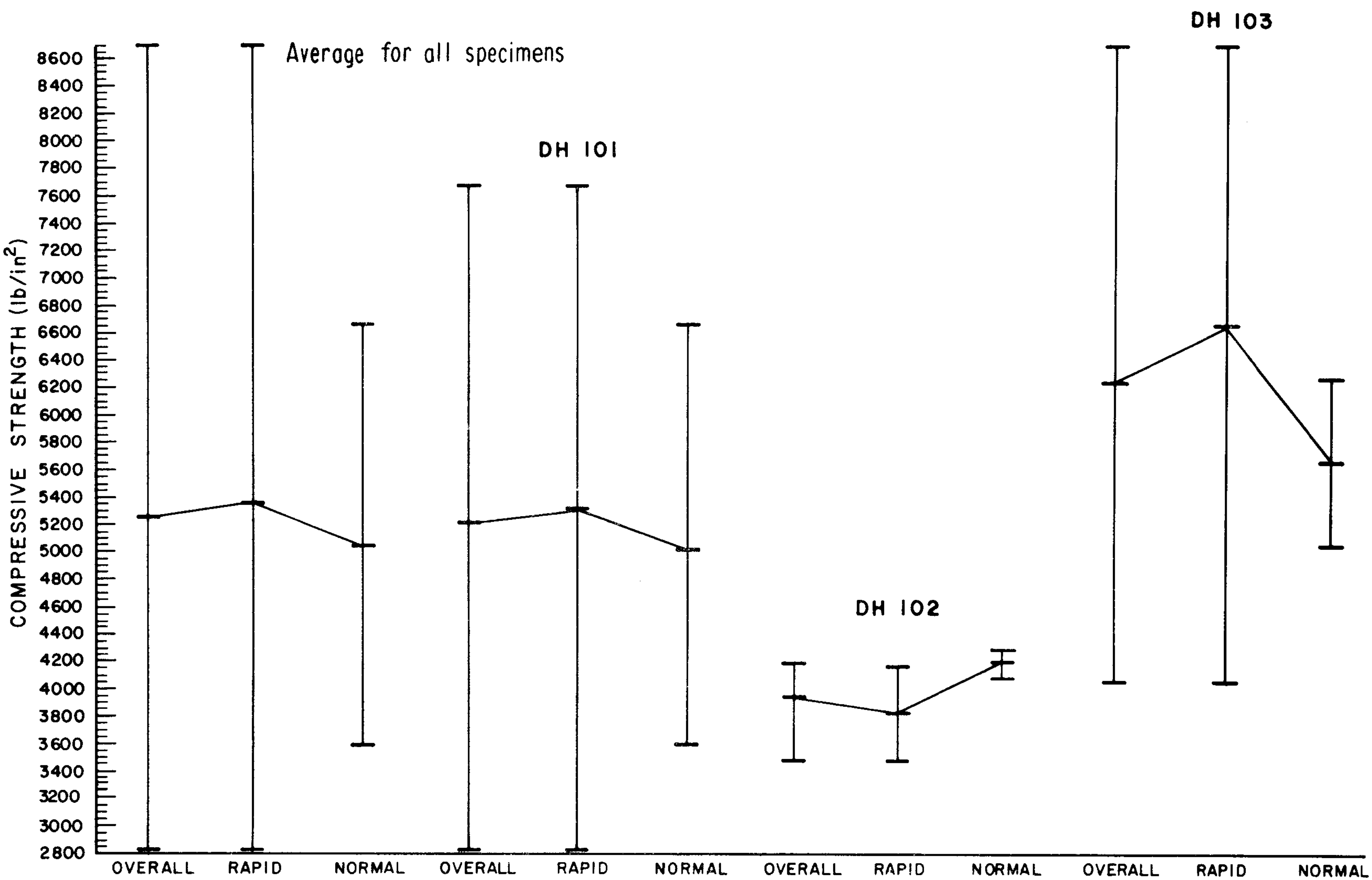


Figure 5.–Stewart Mountain Dam compressive strength summary showing high, low, and average strengths.

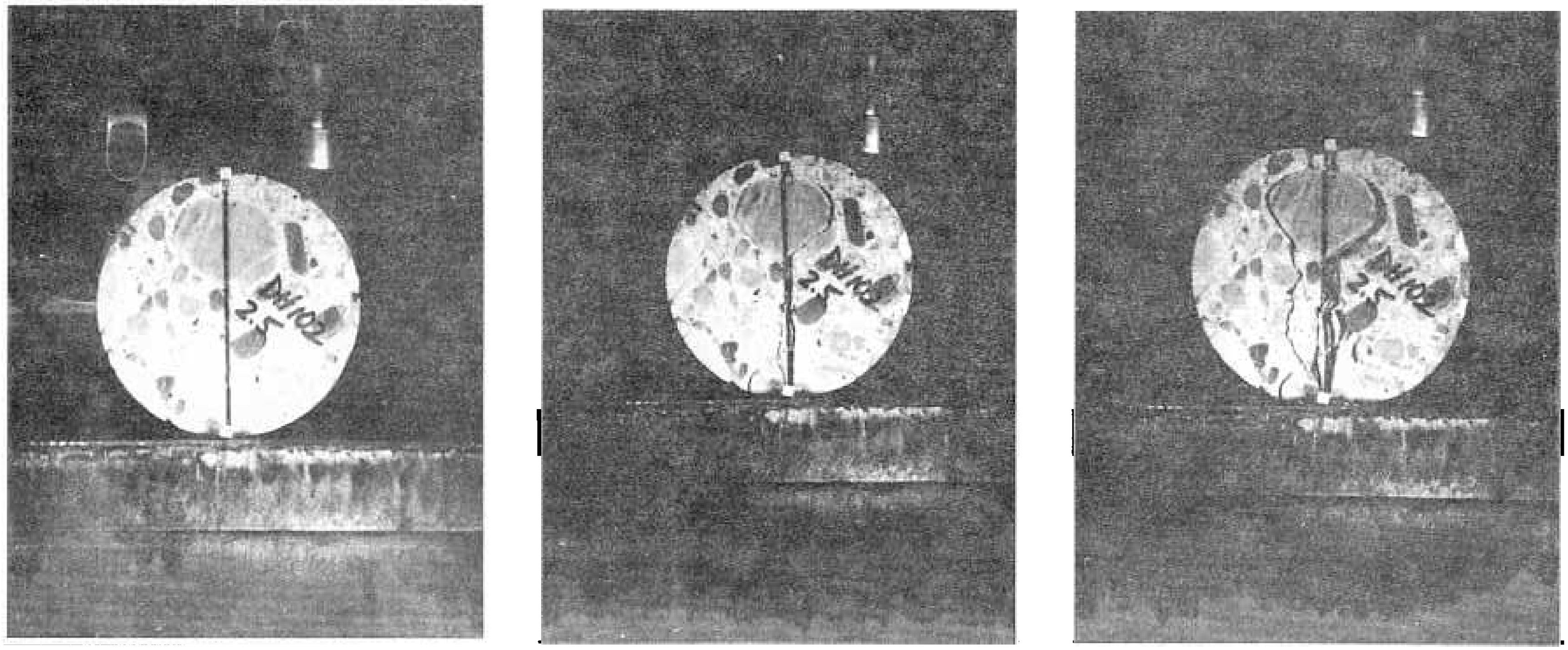


Figure 6.–Typical rapid load splitting tensile test specimens, Stewart Mountain Dam. P801-D-80462, P801-D-80463, and P801-D-80464

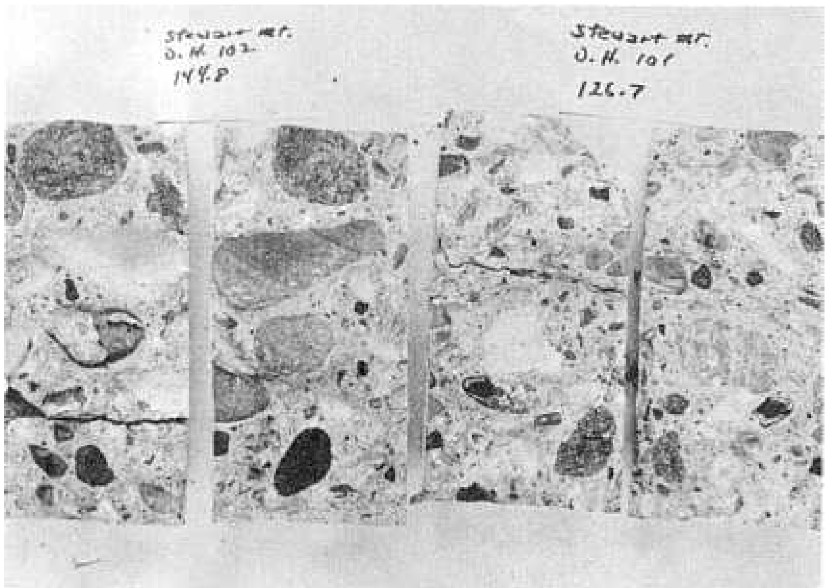


Figure 7.–Typical rapid load splitting tensile afterbreaks, Stewart Mountain Dam.
P801-D-80465

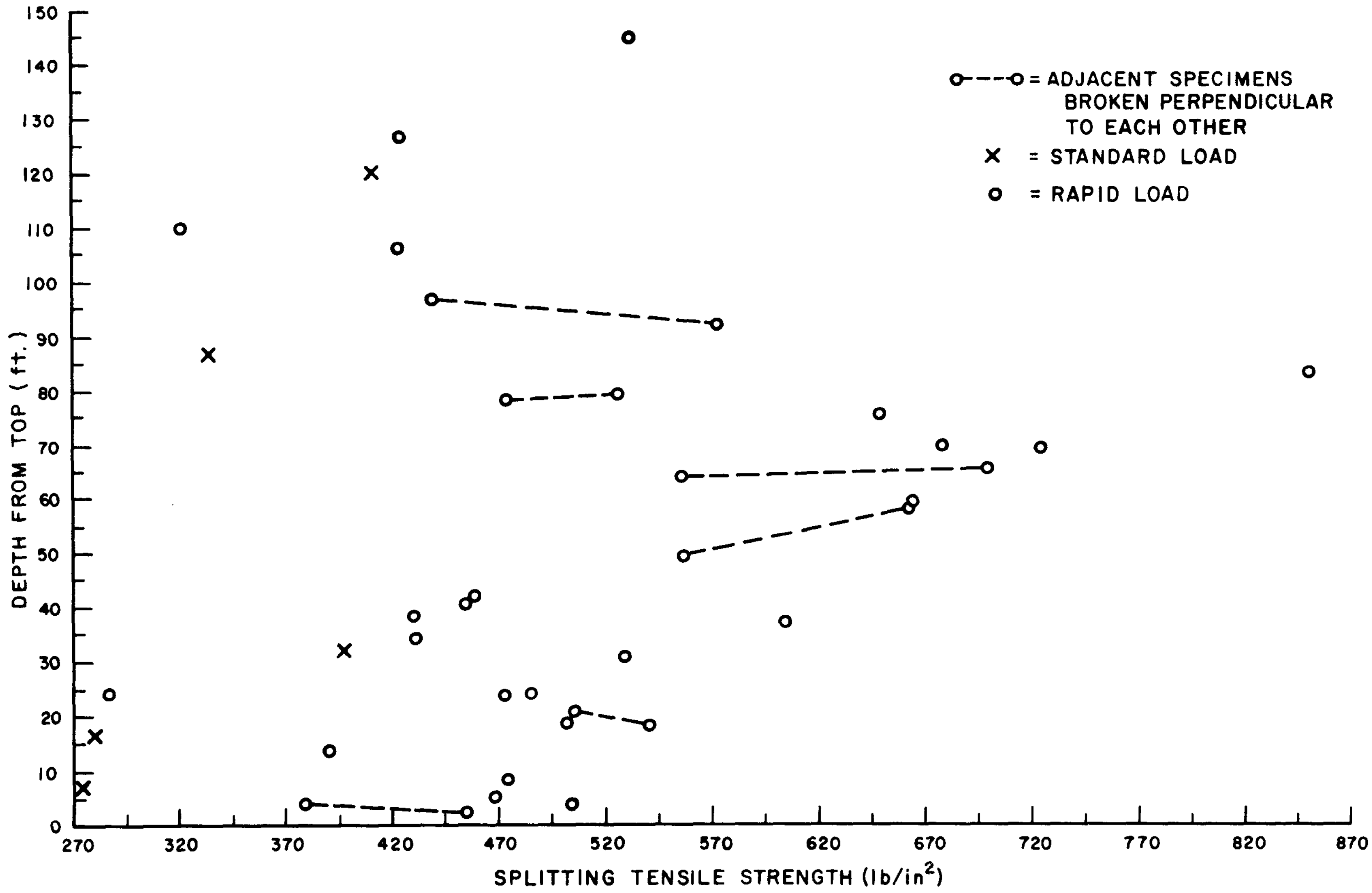


Figure 8.-Stewart Mountain Dam splitting tensile strength versus depth.

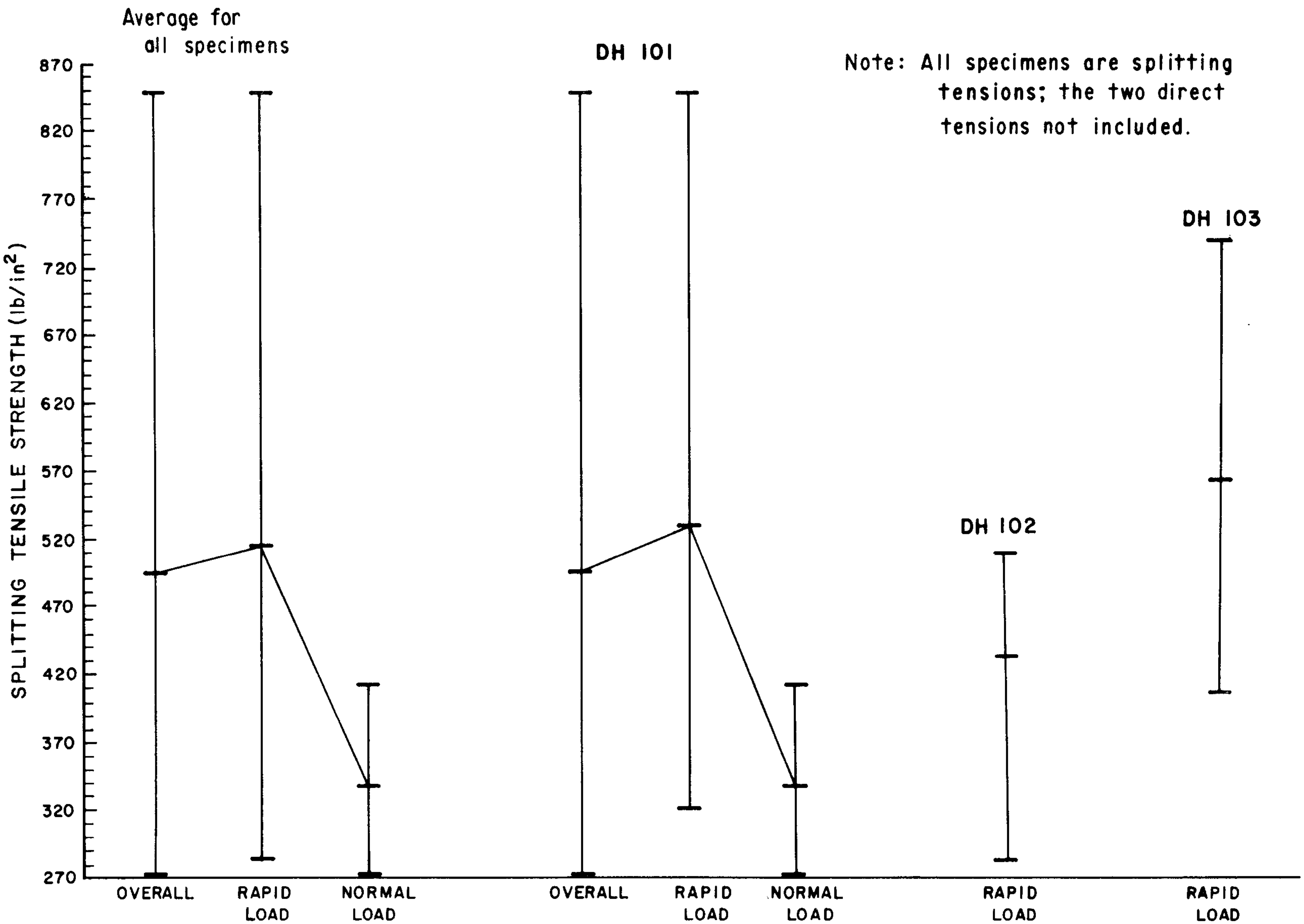


Figure 9.–Stewart Mountain Dam splitting tensile strength summary showing high, low, and average strengths.

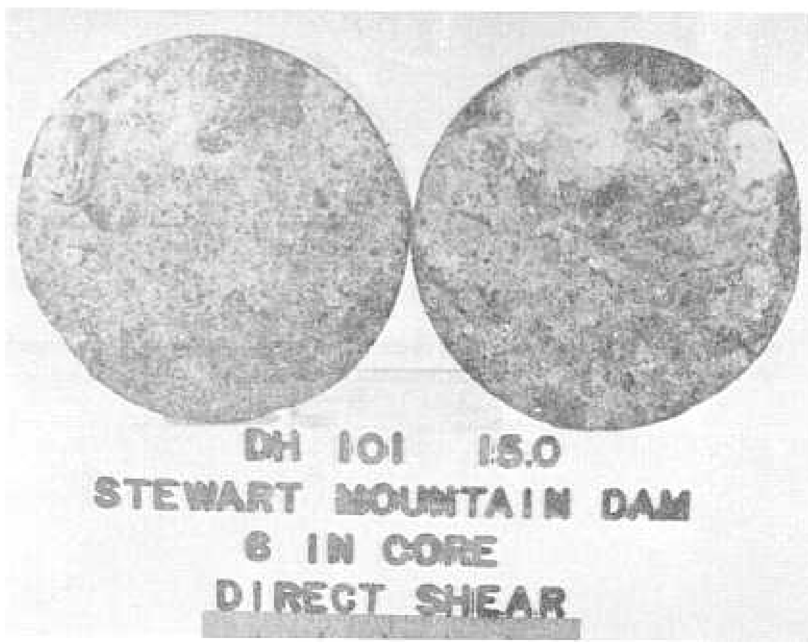


Figure 10.–Typical direct shear specimen, Stewart Mountain Dam.
P801-D-80466 and P801-D-80467

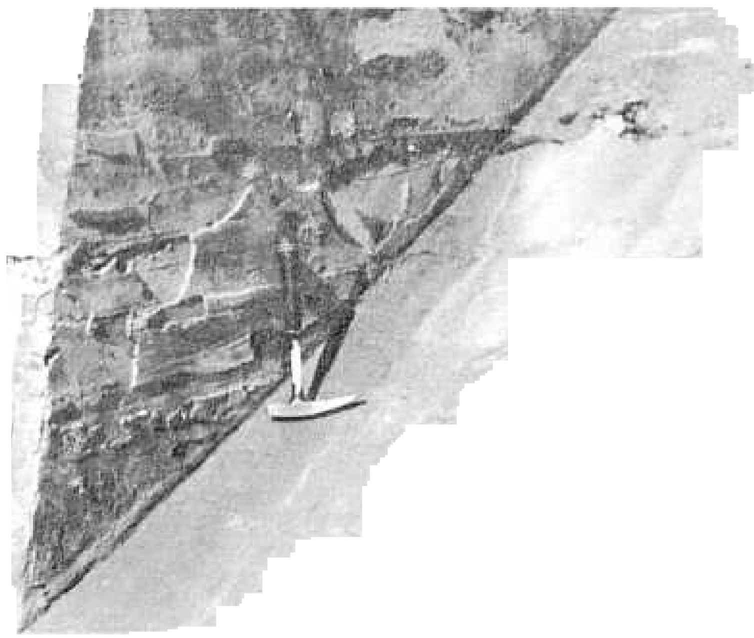


Figure 11.–Typical pattern cracking caused by alkali-aggregate reaction, Stewart Mountain Dam. P801-D-80468

APPENDIX A

OPTIONAL FORM NO. 10
JULY 1973 EDITION
GSA FPMR (41 CFR) 101-11.6

UNITED STATES GOVERNMENT

Memorandum

Memorandum
Denver, Colorado

TO : Chief, Concrete and Structural Branch

DATE: December 21, 1982

FROM : Acting Chief, Applied Sciences Branch

SUBJECT: Petrographic Examination of Concrete Core - Stewart Mountain Dam - Salt River Project, Arizona

Examined by: G. J. Sheldon

Petrographic referral code: 82-89

INTRODUCTION

A cursory examination of Stewart Mountain Dam concrete core from drill holes 101, 102, and 103 was performed in the Concrete Laboratory to select representative fragments for further examination and testing in the Petrographic Laboratory. The purpose of the examination was to perform a complete petrographic analysis on the concrete. In addition, afterbreak analyses were performed on compressive strength and direct shear test specimens.

PETROGRAPHIC EXAMINATION

The selected concrete cores were examined megascopically, microscopically in thin sections and polished surfaces, by X-ray powder diffraction and differential thermal analyses, and by some qualitative physical and chemical tests.

A detailed "Petrographic Examination of Concrete" sheet is attached which includes cursory observations and petrographic descriptions of aggregate, paste, air voids, secondary and hydration products, fractures, and reaction rims. All of the examined cores were similar and described together.

AFTERBREAK ANALYSES

The petrographic afterbreak analyses consisted of visual and stereomicroscope examinations of representative end pieces of six compressive strength specimens tested to failure by the Concrete Laboratory and 30 sliding friction and 3 break bond specimens of apparent construction joints tested by the Rock Mechanics Section. The afterbreak analyses were performed in order to determine those physical and/or structural features influencing the strengths.

Compressive Strengths

The strengths of the specimens varied from about 3,590 lb/in^2 to 6,670 lb/in^2 and were apparently related to the degree of alkali-aggregate reaction. DH-101, 9.4 feet, the lowest strength specimen, contained more reaction rims, highly absorptive and soft glassy volcanic particles, secondary deposits, and silica gel-filled macrofractures than the other examined cores.

Sliding Frictions

Although the sliding friction tests were to be performed on construction joints, 18 of the 30 specimens were tested on apparent mechanical breaks as evidenced by freshly broken rock along the open surfaces. In addition, specimens DH-101, 10.0 feet; DH-101, 67.8 feet; and DH-102, 35.5 feet were tested along open, silica gel-filled fractures. Only specimens DH-101, 15.0 feet, 51.1 feet, and 56.3 feet and DH-103, 16.7 feet, 31.3 feet, 41.5 feet, 46.1 feet, 71.0 feet, and 76.0 feet were tested along apparent construction joints as evidenced by grout layers along the open surfaces. The strengths of all the specimens were influenced chiefly by the roughness of the tested surface.

Break Bonds

Although the break bond tests were to be performed on construction joints, specimens DH-102, 19.2 feet and DH-103, 6.3 feet failed along apparent mechanical breaks and specimen DH-101, 25.3 feet failed along a silica gel-filled fracture.

CONCLUSIONS

The examined concrete from Stewart Mountain Dam is petrographically of fair quality, primarily due to weakening of the concrete by alkali-aggregate reaction. The aggregate is petrographically of fair physical quality and deleteriously alkali-reactive. The paste and aggregate are generally well distributed and the paste-aggregate bond is moderately strong, although locally weak in areas associated with alkali-aggregate reaction. Deterioration due to alkali-aggregate reaction is evidenced by silica gel lining or filling air voids, rock sockets, and fractures; rimming, partially filling, and/or replacing some glassy volcanic particles; and soaking a few areas of paste. Only a few fractures were observed in the examined core.

No evidence of deterioration other than that caused by alkali-aggregate reaction could be detected in the examined concrete. The concrete is well hydrated and only slightly carbonated. Minor amounts of ettringite and soluble chloride and sulfate ions are present but do not appear to be causing adverse damage at this time. No evidence of freeze-thaw deterioration could be detected.

Attachment

Copy to: D-220
D-915
D-1511 (Gaeto)
D-1523

PETROGRAPHIC EXAMINATION OF CONCRETE

Subject:	Stewart Mountain Dam - Salt River Project, Arizona	Field No:	Drill holes 101, 102, and 103

Cursory observations

Six-inch-diameter core; light pinkish-gray paste on surface; hard; dense; well compacted; good paste-aggregate distribution; contains many gel-filled voids and open or filled fractures throughout the core and a few reaction rims around aggregate particles.

Petrographic examination

Aggregate - Gravel: subrounded in shape; consists primarily of granite, gneiss, quartzose sandstone, quartzite, siltstone, chert, and glassy volcanics including a few grayish-pink or yellowish-white, highly absorptive, rhyolite or dacite particles; generally a few to many glassy volcanic particles rimmed, partially filled, and/or partially replaced with generally clear to white silica gel

Sand: generally angular to subrounded in shape; includes the same rock types found in the gravel as well as monomineralic grains of quartz, feldspar, mica, amphibole, and a few miscellaneous detrital minerals

Gravel and sand: petrographically of fair physical quality and deleteriously reactive with high-alkali cement

Paste - Light brownish-gray; slightly absorptive; slightly effervescent with dilute hydrochloric acid; breaks with moderately hard hammer blows around large gravel-size aggregate particles and through small gravel- and sand-size particles; moderately strong paste-aggregate bond, although weaker in areas associated with alkali-aggregate reaction; generally well distributed with aggregate; a few areas soaked with silica gel

Air voids - A few small, rounded, entrapped air voids generally lined to filled with white secondary products

Secondary products - Silica gel: minor to moderate amounts of white to clear, soft to hard, silica gel occurs lining or filling a few air voids, rock sockets, and fractures; rimming, partially filling, and/or replacing some glassy volcanic particles; and soaking a few areas of paste. No silica gel developed in concrete soaked several days in Denver tap water

Other secondary products: minor amounts of small, white ettringite crystals and calcium carbonate lining or filling a few air voids and rock sockets, and minor amounts of soluble chloride and sulfate ions chemically detected

Hydration products	- Minor amounts of calcium silicates; minor to moderate amounts of calcium hydroxide; very few unhydrated cement particles; adequate water of hydration
Fractures	- A few macrofractures and microfractures present; macrofractures in paste and generally filled with silica gel; microfractures generally in paste and around a few aggregate particles and generally unfilled
Reaction rims	- A few to many present; composed of generally thin to thick, hard to soft, and generally clear to white rims; primarily around glassy volcanic particles
NOTE	- The lower strength concrete in drill hole 101 exhibited more reaction rims, highly absorptive and soft glassy volcanic particles, secondary deposits, and silica gel-filled macrofractures than the other examined concrete cores

APPENDIX B

STEWART MOUNTAIN DAM

RAPID COMPRESSION AND
RAPID SPLITTING TENSILE
CORE TEST PROGRAM

INSTRUMENTATION SYSTEM

By Fred A. Travers
Concrete and Structural Branch
Division of Research
January 1983

Stewart Mountain Dam
Rapid Compression and
Rapid Splitting Tensile
Core Test Program

Instrumentation System

General

The Stewart Mountain Dam Rapid Compression and Rapid Splitting Tensile Core Tests were performed in the Vibration Laboratory in November and December of 1982. Load was applied in both cases at a constant rate of strain at a rate that would break the average specimen in 50 milliseconds. Load was measured in both tests and axial and lateral strain were measured for the compression tests. The data were recorded and printouts and plots of the data were generated. The system configuration is shown in figure 1.

Detailed System Description

1. Load System - The 1 million pound load frame was used with its integral 4-inch stroke servo valve controlled hydraulic ram. The system hydraulic pressure was set to 3000 lb/in^2 to give an adequate flow rate through the servo valve to produce the required displacement. The servo valve was controlled by the MTS servo controller and the LVDT internal to the ram was connected to provide strain feedback to the servo controller.

After the specimen was positioned in the load frame, a moderate preload was manually applied by adjusting the set point control on the servo controller. An EXACT Model 126 function generator was set to provide a single shot triggerable voltage ramp of appropriate slope and duration to program the strain rate for the servo controller to break the average specimen in 50 milliseconds. The function generator also provided a trigger signal to the Biomation Waveform Recorder at the end of the ramp.

To test a specimen, the ramp trigger button on the function generator was pushed. This started the voltage ramp which caused the servo controller to open the servo valve which moved the ram up and failed the specimen.

2. Specimen Instrumentation - For both the compression and splitting tensile tests, load on the specimen was measured using a 750,000-pound capacity load cell. The signal from this load cell was conditioned by a Bell and Howell strain gage amplifier and this signal was then fed to the Biomation Waveform Recorder. Calibration of the load cell was done using the 4,000,000 pound testing machine. The calibration data are included in appendix A. No other instrumentation was used on the splitting tensile tests.

Each specimen to be compression tested was instrumented with four strain gages epoxied to its sides, two 4-inch gages axially and two 2-inch gages laterally. The axial pair of gages were connected in a bridge configuration such that their strain would be averaged, and their signal was conditioned by a Bell and Howell strain gage amplifier and fed to the Biomation Waveform Recorder. The lateral pair of gages were similarly connected and conditioned and the signal fed to the Biomation Waveform Recorder.

3. Data Acquisition System - Recording of the load and strain signals was done using the Biomation Model 1015 Waveform Recorder. A trigger signal from the function generator at the end of the ramp combined with an appropriate amount of pretrigger delay provided a recording window that was properly positioned to record the test parameters from the beginning of the load ramp to failure. The recording rate of 0.1 milliseconds per sample provided a window length of 0.1024 seconds.

The waveform recorder samples all channels simultaneously and consequently the compression test data in which three parameters were recorded on individual channels show no time skew from one channel to the next.

The waveform recorder provided a display of the test data on a Tektronix oscilloscope connected to it for this purpose.

Calibration of the waveform recorder was done using an external power supply and DVM. These provided reference levels for use by the computer in calculating actual voltage levels recorded. Documented listings of the calibration programs are included in appendix B.

4. Computer System - An HP-9825 Desktop Computer with 23K RAM, General and Extended I/O ROMs, Plotter ROM, String ROM, Advanced Programming ROM, Systems ROM, and a Disc Drive ROM was used to analyze the data gathered by the Biomation Waveform Recorder and prepare printouts and plots and store the data on flexible discs.

After a test had been run, the computer would transfer the test data from the waveform recorder into its memory and would then scale it to the correct engineering units. A plot of the load vs. time was produced on the graphics CRT for both compression and splitting tensile tests. Axial and lateral strain vs. time were then plotted for the compression tests only. Next, upon command from the operator, all data were stored on two separate flexible discs.

A printout of the data followed. For the compression tests time, load, stress, axial strain, lateral strain, modulus of elasticity, and Poisson's Ratio were calculated and printed. Every tenth point was printed to conserve paper, except that all points were printed around the maximum load in the range of ten points before to ten points after. For the splitting tensile tests, time and load were printed followed by the maximum load and tensile strength.

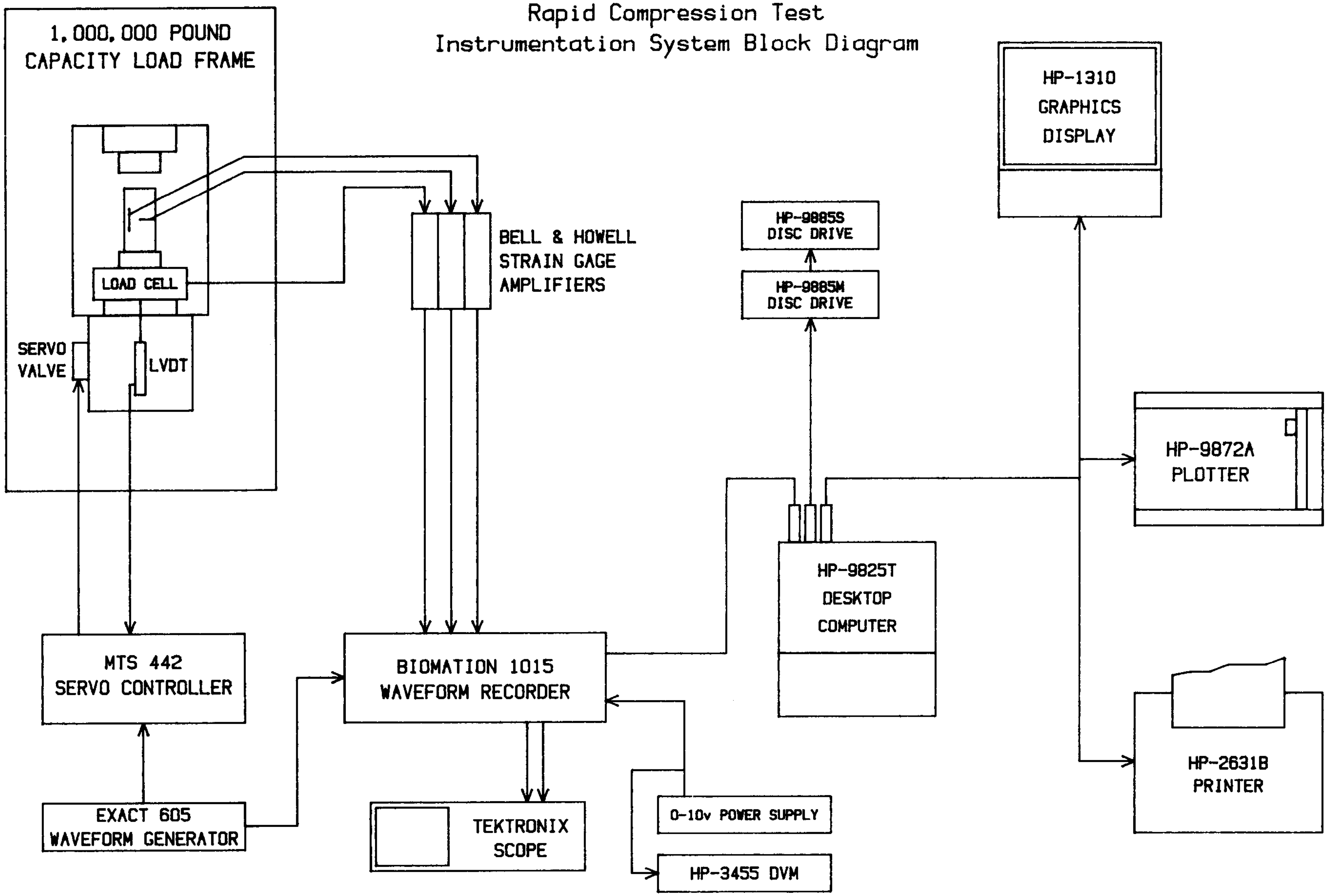
Plots of load vs. time were made for the compression and splitting tensile tests. Additionally, for the compression tests, a plot of stress vs.

axial strain and stress vs. lateral strain was made in two separate colors on the same set of axes.

Documented listings of the programs used are contained in appendix B followed in appéndix C. by sample printouts and plots from the tests.

Figure 1

Stewart Mountain Dam Core Tests
Rapid Compression Test
Instrumentation System Block Diagram



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.