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SOIL-CEMENT SLOPE PROTECTION ON BUREAU OF RECLAMATION FEATURES

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Engineering and Research Center
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16. ABSTRACT

A summary of Bureau ot Reclamation experience with soil-cement slope protection is presented. Compacted soil-cement has been used as a riprap substitute on 7 major Bureau structures. Preconstruction testing, construction equipment and procedures, construction control testing for soil-cement, and performance of soil-cement facings are discussed. Successful performance of a soil-cement test section at Bonny Reservoir in eastern Colorado was used as the basis for the design of the facings, and durability and compressive strength test results limits established by the test section have been generally followed. Most soils used by the Bureau have been fine, silty sands; a summary of test results is presented. The soil-cement is: (1) mixed in a continuous flow mixing system, (2) placed, and (3) compacted in nearly horizontal lifts with a combination of sheepsfoot and pneumatic rolling. Erosion of uncompacted material at the edge of the lifts results in a stairstep pattern of the slope. Durability tests on record cores taken at most features show low weight losses. Performance of soil-cement facings in service has been generally satisfactory. More than normal breakage has occurred at a few locations on Cheney Dam in Kansas. Has 17 references.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS--/ *soil cement/ *slope protection/ *erosion control/ sands/ gradation/ soil compaction/ freeze-thaw tests/ wetting and drying tests/ mixing/ compaction equipment/ performance tests/ records/ construction/ embankments/ bibliographies/ soil investigations/ construction control/ construction equipment/ tests/ *earth dams/ dam construction/ durability

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by

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May 1971

Soils Engineering Branch
Division of General Researc
Engineering and Research C
Denver, Colorado

UNITED STATES DEPARTMENT OF THE INTERIOR * BUREAU OF RECLAMATION
Rogers C. B. Morton, Secretary
Ellis L. Armstrong, Commissioner

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This summary report was prepared under the supervision of A. A. Wagner, formerly Head, Physical Properties and Construction Control Section (retired); W. Ellis is presently Head of the Section. H. J. Gibbs is the Soils Engineering Branch Chief. Some of the information contained in this report has been extracted from previously published Soils Engineering reports as referenced. Prior to publication, the text of the report was checked by R. R. Ledzian and reviewed by C. A. Lowitz and C. W. Jones, Head, Special Investigations and Research Section.

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INTRODUCTION

The development of projects in areas where rock riprap is scarce has created a need for other means of slope protection. A number of alternate slope protection schemes such as soil-cement, concrete paving, steel sheet, and asphaltic concrete have been investigated. The background of these investigations is given in EM-652.[1] *

The use of soil-cement slope protection was first tried by the Bureau of Reclamation on Bonny Test Section in eastern Colorado. Soil-cement was not used directly on the face of the dam as this was considered to be experimental work. A special embankment was constructed along the south side of the reservoir and faced with soil-cement. Detailed descriptions of the test section and construction procedures are given in EM-652[1] and EM-630[2]. The successful performance of the facing in the test section led to the conclusion that soil-cement could be used as slope protection on major hydraulic structures.

To date (1970), the Bureau of Reclamation has used soil-cement slope protection on seven major water-retaining structures. Most of these structures have been on the Great Plains, an area where suitable rock for riprap occurs only in scattered locations.

However, one of the structures is in Utah and the possible use of soil-cement has also been investigated for structures located in areas normally thought of as being near mountainous areas. However, even in mountainous areas, the haul distance for suitable rock can be many miles. If a suitable source for soil-cement material is available at a short haul distance, the use of soil-cement may be competitive and should be investigated. Soil-cement is usually considered for an alternate method of slope protection if the haul distance to a suitable rock source exceeds about 20 miles. Nevertheless, the nearest rock source, or sources, must be investigated so that alternate bids can be obtained if found by the designer to be desirable.

The following table lists the locations of soil-cement facings constructed to date. The table also gives the year of construction, estimated yardage in the specifications and Soils Engineering Report number which presents the results of the investigation testing program.

The figures in the Appendix also show test results from three features on which soil-cement was investigated but was not used. These three are included since the soil types were somewhat different from the soils used on other structures. These features are Little Panoche Creek Detention Dam, California (EM-712)[10]; Red Bluff Reservoir Bank Stabilization Area, California (EM-692)[11]; and Conconully Dam (EM-746)[12].

The purpose of this report is to present a summary of the Bureau's experience with soil-cement facings to date. In addition, the durability results in some of the referenced reports were computed with various assumptions. These test results have been recomputed with the same assumption used on all the tests.

A seepage test section was incorporated into the slope facing on the Lubbock Regulating Reservoir. This section included provisions for measuring the amount of water which permeated the soil-cement. The data collected showed that the soil-cement on that feature had quite low permeability. Report No. REC-ERC-71-13 entitled Soil-Cement Seepage Test

Feature	Location near	Year constructed	Estimated volume (cu yd)	Reference report
Merritt Dam	Valentine, Nebraska	1963	51,000	EM-671[4]
Merritt Dam Modification	Valentine, Nebraska	1968	13,400	EM-611[3]
Cheney Dam	Wichita, Kansas	1964	180,000	EM-668[5]
Lubbock Regulating Reservoir	Lubbock, Texas	1966	53,000	EM-729[6]
Glen Elder Dam	Beloit, Kansas	1967-68	138,000	EM-719[7]
Downs Dike	Beloit, Kansas	1967	63,000	EM-737[8]
Cawker City Dike	Beloit, Kansas	1968	86,000	EM-737[8]
Starvation Dam	Duchesne, Utah	1969	72,000	EM-714[9]

*Numbers refer to references at end of text.

Section, Lubbock Regulating Reservoir, Canadian River Project, Texas,[13] gives a complete summary of the test section details and observations.

TEST PROCEDURES

The first step in any construction project is to find suitable material and soil-cement is no exception to this. Although almost any material can be used to produce soil-cement, certain more desirable material types have been outlined. Figure 1 shows gradation

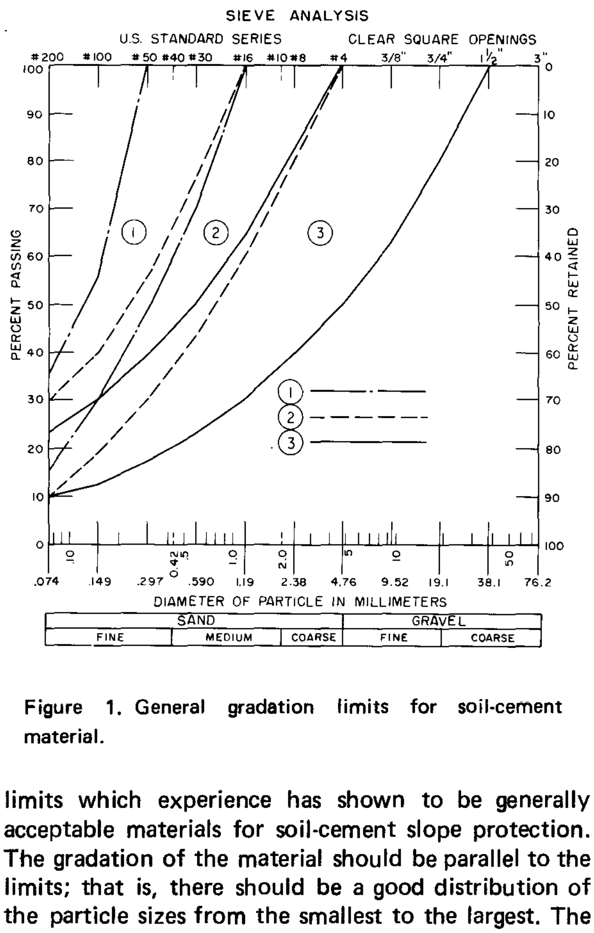

Figure 1. General gradation limits for soil-cement material.

limits which experience has shown to be generally acceptable materials for soil-cement slope protection. The gradation of the material should be parallel to the limits; that is, there should be a good distribution of the particle sizes from the smallest to the largest. The limits defined by the higher numbers on the figure would usually require lower cement contents for an equal quality of soil-cement than the finer materials. This is due to larger surface area of particles per unit volume in the finer materials as well as the higher percentage of voids (that is, lower density) generally obtained in the finer-grained material. As previously stated, these gradation limits do not include all the materials which could be used for soil-cement, but materials outside these ranges would be expected to require higher cement contents. Also materials coarser than Gradation 3 would be quite coarse to mix and place as soil-cement.

Standard Identification Tests

After a potential material source has been located, more detailed testing is performed. Standard test procedures are used for gradation and Atterberg limits tests. In addition to the standard ASTM stirring apparatus for gradation tests, the wrist shaker dispersion method has often been used. The gradation test specimen is set up the same as for the standard gradation test. However, the specimen is dispersed in a 250-milliliter Erlenmeyer flask and agitated for 10 minutes with a device known as a wrist shaker. As implied by the name, this device imparts a motion similar to that of shaking the flask by hand with a wrist action. There are no moving parts in contact with the soil and the dispersant action is less violent than that obtained with the standard ASTM stirring device. A comparison of the gradation curves obtained by the two methods gives a qualitative estimate of the durability of the individual particles. A similar comparison could be obtained using the air dispersion method which is now a standard ASTM test procedure.

If the gradation and Atterberg limits of the material indicates that it is within acceptable limits, Proctor compaction tests are performed. The cement content required for producing soil-cement is estimated based on the gradation characteristics and the Proctor maximum dry density of the material with an average cement content based on soil type. The average and estimated cement contents are shown in Tables 1 and 2 of Portland Cement Association Publication CB-11 14. These cement contents are for use as pavement base on highways and are generally low for use in hydraulic structures. Because of the direct exposure and erosion conditions, the cement contents should be increased by about 2 percent for use as slope protection. Proctor compaction tests are normally performed on the material at the cement content estimated and at cement contents 2 percent above and below that estimated. Test specimens of the soil-cement are placed according to the compaction test results at each cement content. However, if the compaction test results indicate that cement content does not change the compaction characteristics significantly, an average value may be used.

Durability Tests

Durability tests are performed on compacted specimens to determine the resistance of the soil-cement to the effects of wet-dry and freeze-thaw cycles. The wet-dry tests are performed in accordance with ASTM Test Designation D-559 and the freeze-thaw tests are performed in accordance with ASTM Test Designation D-560 with the exceptions discussed below. The ASTM procedures do not require weights at each of the 12 brushing cycles. The weights before and after brushing for each test cycle are obtained in Bureau tests. From these data, the amount of soil-cement brushed off at each cycle can be determined. Since the ASTM procedures do not require weights for each cycle, the loss must be computed from the final weight. Hydration of cement chemically combines water which cannot be driven off by the final drying at 110° C. Because of this chemically combined water, a correction has to be made to the final dry weight. Assumptions for the amount of the water of hydration have been made based on cement content and soil type. Obtaining the weights as done by the Bureau laboratories eliminates the need for the assumption mentioned above. The percent loss is calculated by dividing the accumulated soil-cement loss through the 12 cycles by the initial dry weight. This method of calculating the loss probably overstates the loss by a small amount since the accumulated losses are wet weights rather than dry weights. Nevertheless, the percent losses computed from the accumulated weight losses are usually less than those computed using the assumption for hydration, and the losses continue to decrease at increasing cement contents, a trend which did not always develop using the assumption for hydration.

Compressive Strength Tests

Compressive strength specimens are formed at the same placement conditions as the durability specimens. The procedures used for molding the specimens generally conform to ASTM Designation D-1632 although there has been some variation of the procedure used. Molds currently in use were fabricated from seamless tubing and produce specimens 2.8 inches in diameter by 5.6 inches high (7.1 by 14.2 cm). Most specimens are compacted with the drop hammer apparatus although some specimens have been compacted using a hydraulic loading jack or compression machine. Either method seems to produce suitable specimens. The soil-cement specimens are usually left in the mold for 2 days in 100 percent relative humidity storage prior to extruding them. Extrusion is accomplished by applying a steady load to one end of the specimen with a hydraulic jack or compression machine. After extrusion, the specimens are stored at 100 percent relative humidity and 72° F (fog room curing).

Compressive strength specimens are formed for testing at 3-, 7-, 28-, and 90-day ages. Prior to testing, the specimens are normally soaked in water for a period of 4 to 24 hours. The specimens are then capped with a clay-sulfur compound. The unconfined compressive strength tests are performed in accordance with ASTM Designation D-1633. A hydraulic testing machine is used and the load is applied at about 20 psi (1.4 kg/sq cm) per second.

CEMENT CONTENT AND MATERIAL SELECTION

Gradation and Density

The gradation of the material has considerable influence on the workability and acceptability as a source for soil-cement. As previously mentioned, materials with large amounts of gravel would be difficult to mix and place as a uniform layer. On the other hand, materials which are very fine would also be difficult to mix adequately. As shown on Figure 2,

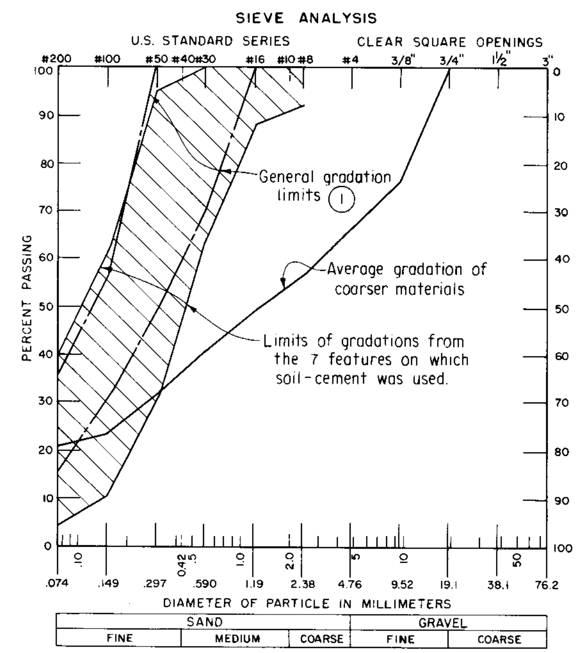

Figure 2. Summary of gradation test results on soil-cement material.

most of the materials tested by the Bureau of Reclamation on features which used soil-cement could be classified as fine, silty sands. These materials are mostly in general gradation Limits 1 and the finer side of Limits 2. Individual gradations for more detailed comparisons are shown in the Appendix on pages 29 through 46. The exceptions to these statements are the materials from projects which did not actually use soil-cement. The materials from Little Panoche Creek Detention Dam and Red Bluff Reservoir were sands and gravels containing up to 50 percent plus No. 4, and the average gradation from these features is identified on Figure 2 as coarser material. The other material was from Conconully Dam and was a "rock-flour" silt which contained only 5 percent sand. Although these materials were not used, test results showed that acceptable soil-cement could have been produced from them.

The State of New Mexico has used a material containing 25 percent gravel on Ute Dam[15]. However, the water surface on this reservoir has never been raised to the level of the soil-cement and no performance data of the soil-cement under wave action are available.

Another thing to take into consideration in the selection of soil-cement material is the presence of "clay balls" (rounded balls of fines and sand which do not break down during ordinary processing). Alluvial sand deposits which might otherwise be acceptable often contain layers of silt and clay. These layers are caused by low flows in the depositing stream and all alluvial deposits have this characteristic to a certain extent. "Clay balls" in the material tend to go through the processing intact and do not disperse through the sand. A small amount of minus 1-inch (2.5-cm) clay balls is not considered sufficient grounds to reject otherwise suitable material. On some projects, the material was screened to remove larger clay balls before the material was introduced into the mixing plant. During the investigation stage, the material should be screened at its natural moisture to obtain an estimate of the amount of "clay balls" present in the deposit. The "clay balls" in an auger hole sample will normally be rather small size due to the action of the auger. However, the presence of over about 10 percent "clay balls" even of the smaller sizes will probably indicate problems during construction. Excavation procedures will not usually break down the clay lenses and the result is large clods of clay within the sand stockpile.

For the type of fine, silty sands that have been used by the Bureau of Reclamation, it has normally been thought that 10 to 30 percent fines is the most desirable range. The limits shown on Figure 2 show that most of the soils are near this range. Soils with less fines are thought to require more cement to produce soil-cement of equal quality. This would be due to the use of cement to fill voids which would normally be filled with fines rather than coating the soil particles to cement them together. Some indication of this can be seen in the increase of the Proctor maximum dry density at increasing cement contents on materials with very low percentages of fines for the following samples: Merritt Dam 15R-49, -111; Cheney Dam 25J-X108; Glen Elder Dam 18C-X224; and Downs Dike 40Y-42. A review of the durability test and compressive strength test results in the Appendix does not show any clear-cut trends for these samples. When compared to other samples from the same features which had a supposedly more favorable gradation, some of the samples deficient in fines show poorer characteristics while others show about the same characteristics. An additional comparison of samples containing up to 40 percent fines also failed to show any definite trends. However, from the limited data available for comparison, excess fines seem to be more detrimental than a deficiency of fines. These comparisons indicate that general guidelines can be used in the search for materials; however, each sample should be tested and judged on its own merits before potential sources of material are rejected.

Very little soil-cement work has been done by the Bureau on soils which have plastic fines. Fines with a slight amount of plasticity would probably mix and handle quite well; however, plastic fines would make mixing the soil, water, and cement adequately more difficult.

Durability Tests

The ability of compacted soil-cement to resist the destructive effects of wet-dry and freeze-thaw cycles determine to a large extent whether it will form an acceptable slope protection. Standard durability tests are performed to determine the relative quality of the materials being proposed for use. At the time the Bonny Test Secton was constructed, the Portland Cement Association did not have basic criteria for the design of soil-cement to be used in hydraulic structures. The criteria for use on pavement base course construction was that the specimen losses should not exceed 14 percent during the 12 cycles of the durability tests for AASHO Soil Classifications A-1, A-2-4, A-2-5, and A-3.[14] These soil classifications include nearly all of the soils used by the Bureau although a few soils might fall into the A-4 if the percent of fines are over 36 percent even though they are nonplastic or of very low plasticity. PCA recommends a cement content which will give 10 percent or less loss for A-4 soils. It was recognized that exposure on the face of a dam would be more severe than that on a pavement base. Therefore, the cement content was increased 2 percent above that required for pavement base on the Bonny Test Section.[1] This resulted in a specified cement content of 10.4 percent by dry weight for Type A soil and 8.1 percent by dry weight for Type B soil.

The specified cement contents on the Bonny Test Section produced soil-cement with maximum losses of about 8 percent on the freeze-thaw tests and about 6 percent on the wet-dry tests. The durability test results on the two types of soil used for Bonny Test Section are shown on Figure 3. The performance on Bonny

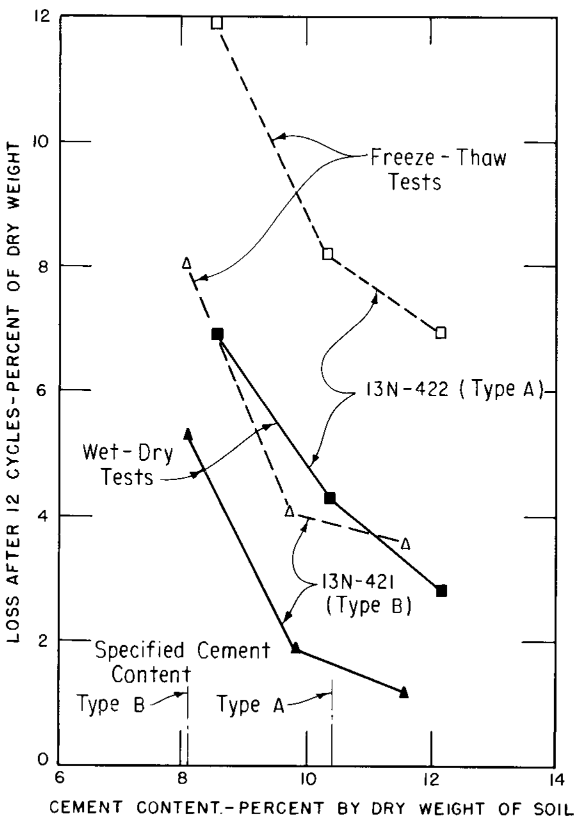

Figure 3. Soil-cement durability test results—Bonny Test Section.

Test Section was satisfactory and those limits have been used as design criteria for soil-cement slope protection. As noted on pages 47 and 57 in the Appendix, the losses on the Bonny Test Section materials are shown directly as reported in EM-250.[16] The original data for these tests were not available so the losses could not be computed from the accumulated losses as was done for the other samples. Also, a comparison with durability test results shown on Figures 3 and 5 indicate that the testing on the samples from Bonny Test Section and initial program on Merritt Dam may have been performed by somewhat different procedure. These test results show losses which are higher and increase much more rapidly at decreasing cement contents than similiar soils tested from other projects. For example, if the brushing cycles had been performed with more than the 3 pounds (1.36 kg) of force specified, the losses would be larger. If that supposition is correct, soil-cement tested according to the specified procedures which shows lower losses would be about the same quality.

The durability test results from all the projects are summarized on Figures 4 and 5 and are shown on pages

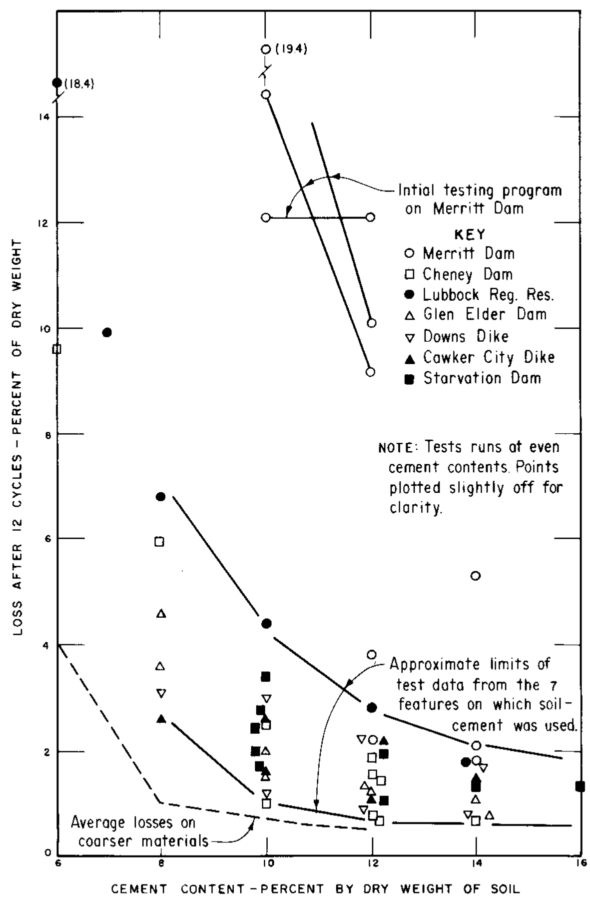

Figure 4. Summary of freeze-thaw durability test results on soil-cement.

47 through 66 of the Appendix for each feature. The specified cement contents have been 12 percent by dry weight of soil on 6 of the features on which soil-cement was used and 14 percent on Merritt Dam. These cement contents are above that required to produce soil-cement with the 6 and 8 percent durability losses. The recommended cement content on Little Panoche Creek and Red Bluff Reservoir were 6.3 and 9 percent, respectively, for these coarser materials.

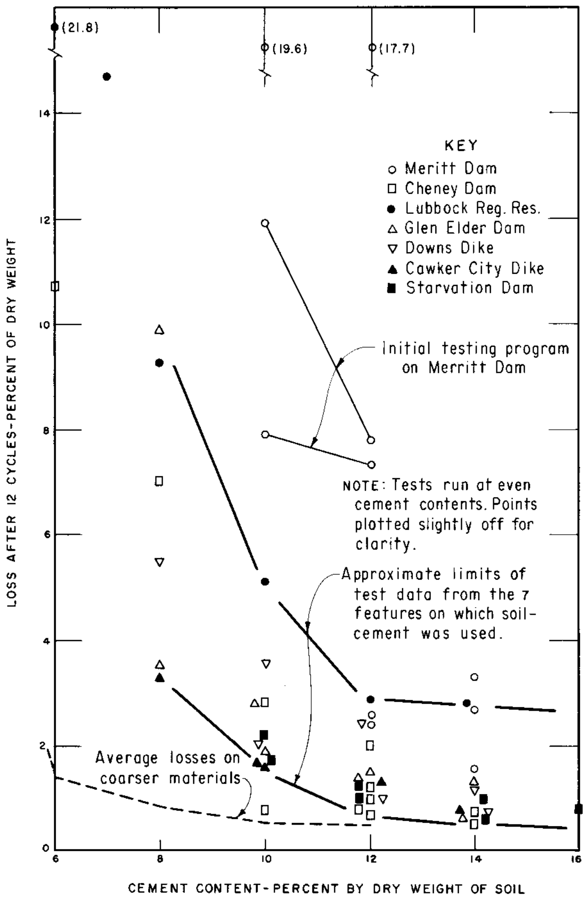

Figure 5. Summary of wet-dry durability test results on soil-cement.

The coarser materials tend to show that a lower cement content could be used with a material with better distribution of grain sizes. On most facings constructed by the Bureau, the specified cement content produced soil-cement with losses of 3 percent or less on the laboratory test specimens. However, if the more recent soil-cement tests have been performed with less brush pressure, the quality may be about the same. In addition, on some of these features, the test results shown in the reports referenced in the Introduction showed higher losses than those shown in the Appendix of this report. This difference was caused by the assumption used to calculate the results rather than using the actual accmumulated losses as was done in this report. Due to the difficulties of proportioning cement and soil accurately during construction, cement content is usually not specified near the content at which the losses begin to rise rapidly. For example, on Figures 4 and 5, the upper limits of the test data begin to rise quite rapidly at 10 to 12 percent cement. The cement content would normally be specified somewhat higher than that point. This point is also illustrated in more detail on pages 51, 52, 61, and 62 of the Appendix for Glen Elder Dam, Downs Dike, and Cawker City Dike. A cement content about 2 percent less than specified would have produced soil-cement with about the same durability test losses. However, at the lower cement contents, a further reduction in cement content would have resulted in much higher durability losses.

Compressive Strength Tests

Compressive strength test results are used mainly as a qualitative measure of quality rather than a quantitative measure. Compressive strength is a faster test to perform and it is used in construction control testing rather than using durability tests. The results of the construction control specimens can be then compared to the investigation program tests to ascertain if soil-cement about equal in quality to that tested is being produced.

Although used mainly as an indicator, compressive strength does have some bearing on the design of soil-cement. Soil-cement does need some strength to resist breaking under the stresses caused by wave action. Soil-cement slope protection can be envisioned as a series of beams placed horizontally or nearly so up the slope. This results in an offset of each successive layer equal to the product of the slope and layer thickness; therefore, each layer has an exposed portion which is not restrained on top. Waves breaking on the surface cause pressures between the lifts if they are not properly bonded. These pressures and the uplifts caused by breaking waves cause the exposed portion of the lift to act as a cantilevered beam. Wave action probably would not cause large stresses in this "beam" but part of the stress would be in tension. Tensile strengths are normally thought of as being 10 to 15 percent of the compressive strength. Therefore, low compressive strength test results might indicate the facing would deteriorate by chunks breaking off during wave action rather than individual grains loosening by wet-dry or freeze-thaw action.

The Portland Cement Association criteria on compressive strength is only that the strength should increase with age and greater cement content.[14] Figure 6 shows the compressive strength test results from the investigation work for Bonny Test Section. The specified cement content on these materials gave a minimum 7-day compressive strength of about 600 psi (42 kg/sq cm) and a minimum 28-day compressive strength of about 875 psi (62 kg/sq cm). These strengths have been used as minimum requirements on Bureau features since that time.

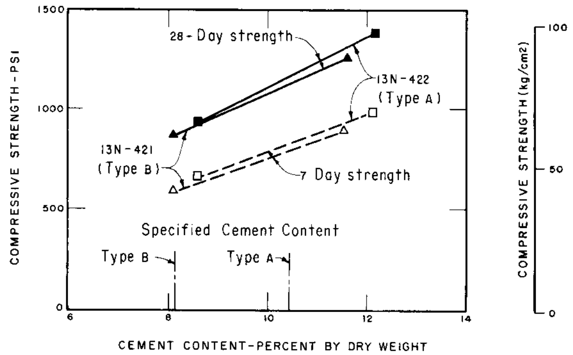

Figure 6. Soil-cement compressive strength test results—Bonny Test Section.

The compressive strength test results are summarized on Figures 7 and 8 and are shown individually in the Appendix (on pages 67 through 84) for each feature.

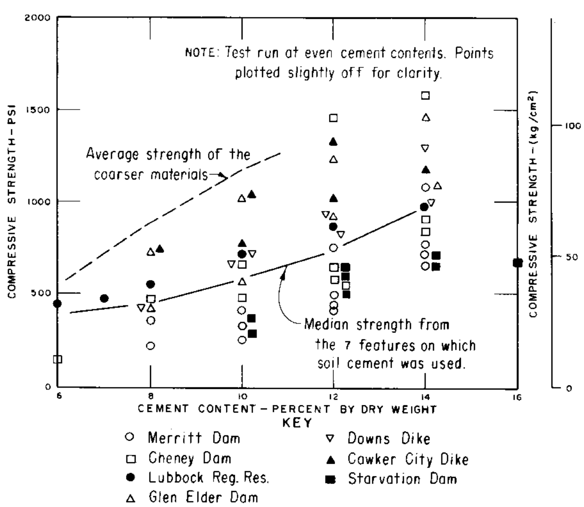

Figure 7. Summary of 7-day compressive strength test results on soil-cement.

A review of the summary figures shows that the increasing strength requirement is met in nearly all cases. The few cases where the strength did not increase were probably caused by normal variability of laboratory testing. These test results also show that the specified cement content was usually greater than that required to produce the compressive strengths discussed above. The strengths vary over a fairly wide range on the features using soil-cement as shown on

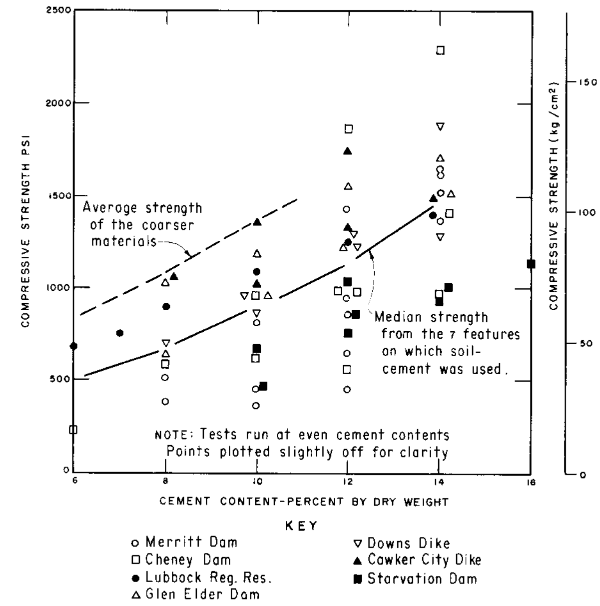

Figure 8. Summary of 28-day compressive strength test results on soil-cement.

Figures 7 and 8. However, most of the strengths at the specified cement content are above the minimums established on the Bonny Test Section. The median strength shown is about 750 psi (52.7 kg/sq cm) for the 7-day specimens and 1,150 psi (80.9 kg/sq cm) for the 28-day specimens at 12 percent cement. These values exceed the minimums by about 25 percent. Some of this increased strength may have been due to differences in testing methods as discussed below. Additional confirmation that better-graded materials would produce good quality soil-cement with less cement is also shown on Figures 7 and 8. The average strengths on these materials reach the minimums at about 7 percent cement by dry weight of soil. This is 3 to 4 percent cement less than needed for the fine silty sands to reach these same strengths.

All compression tests have been performed on cylindrical specimens but there has been some variation in the sizes used. The compressive strength tests for Bonny Test Section were performed on 2- by 2-inch (5.1-by 5.1-cm) specimens. Testing programs on Merritt Dam and Cheney Dam were performed with 2- by 2-inch (5.1- by 5.1-cm), 2.83- by 2.83-inch (7.2- by 7.2-cm), and 2.83- by 5.67-inch (7.2- by 14.4-cm) specimens. Since that time, all of the features have been tested using the 2.83- by 5.67-inch (7.2- by 14.4-cm) specimens although some 2- by 2-inch (5.1- by 5.1-cm) specimens were run for comparison. A comparison of specimen sizes and testing methods on a sample from Cheney Dam is given in Table 1.

Table 1

COMPARISON OF COMPRESSIVE STRENGTH TESTING METHODS

Specimen size (inches)	Capping material	Soaking prior to testing	Compressive strength (psi)	
			3-day	7-day
2.8 by 5.7	Sulfur	Not soaked	1,213	1,462
2.8 by 5.7	Sulfur	1 hour	864	1,123
2 by 2	Not capped	Not soaked	535	787
2 by 2	Not capped	1 hour	410	–
2.8 by 5.7	Sulfur	4 hours	822	1,094
2.8 by 5.7	Not capped	4 hours	814	1,021

Note: Tests run on Sample No. 25J-156, Cheney Dam, 12 percent cement, placed at optimum water content and 98 percent of Proctor dry density.

These data show that the 2.83- by 5.67-inch (7.2 by 14.4-cm) specimens give higher compressive strength results. Table 1 also shows the drop in compressive strength, on that sample at least, due to soaking the specimens prior to testing. The strengths shown on figures in the Appendix for 25J-156 are probably higher than they would have been if the specimens had been soaked. There seems to be little difference due to soaking for 1 or 4 hours which indicates there should be very little difference between soaking for 4 or 24 hours. Additional comparisons of compressive strength on different specimen sizes are shown on pages 74, 75, 78, and 79 of the Appendix for Glen Elder Dam and Starvation Dam. These results show divergent trends with higher strengths shown on 2.83- by 5.67-inch (7.2- by 14.4-cm) specimens on Glen Elder Dam but lower strengths on Starvation Dam.

Generally, it seems that the 2.83- by 5.67-inch (7.2- by 14.4-cm) specimens give higher strengths than the 2- by 2-inch (5.1- by 5.1-cm) specimens used on Bonny Test Section. Therefore, the higher strengths obtained at the specified cement contents on later features may not indicate as conservative an approach as it would seem.

Effects of Compaction, Time Delay, Water Content, Etc.

The test results discussed previously in this report were all performed under standard laboratory conditions. In order to estimate the effects of different conditions which might be encountered, series of laboratory tests were performed on soils selected from some of the features.

Some of the variations anticipated during construction were percent compaction, variation from optimum water content, and time delay from mixing to final compaction. The tests shown on the following figures were performed to check on these items.

Figure 9 shows average trends of compressive strength test results as effected by the percent compaction obtained. The results from which these trends were obtained are shown on pages 85 through 89 of the Appendix. These tests show that an increase in the compaction resulted in soil-cement of higher compressive strength. However, it was not considered practical to specify greater compaction in order to reduce the cement content. The construction procedures at most features have resulted in higher densities than the 98 percent of Proctor maximum dry density specified and this can be considered as extra quality above that required.

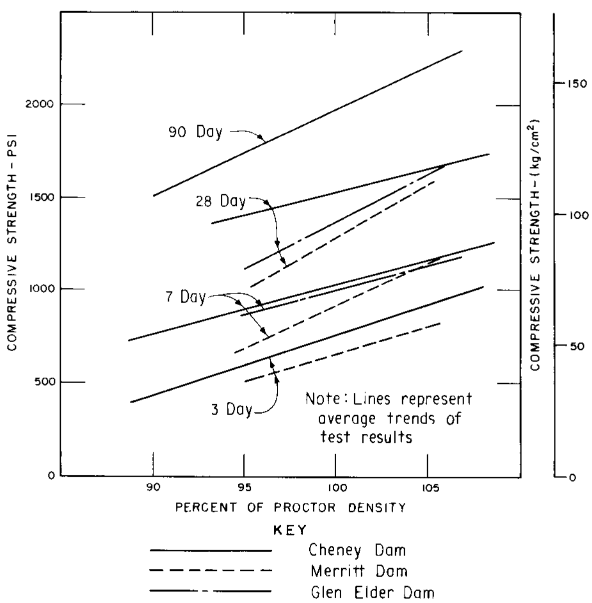

Figure 9. Effect of density on compressive strength of soil-cement.

Further indications of the increase in quality at higher densities are shown on Figure 10. These durability tests were run on material from Merritt Dam and show that the durability increases at higher densities. Although the losses are low, a significant decrease in the losses does occur at higher density.

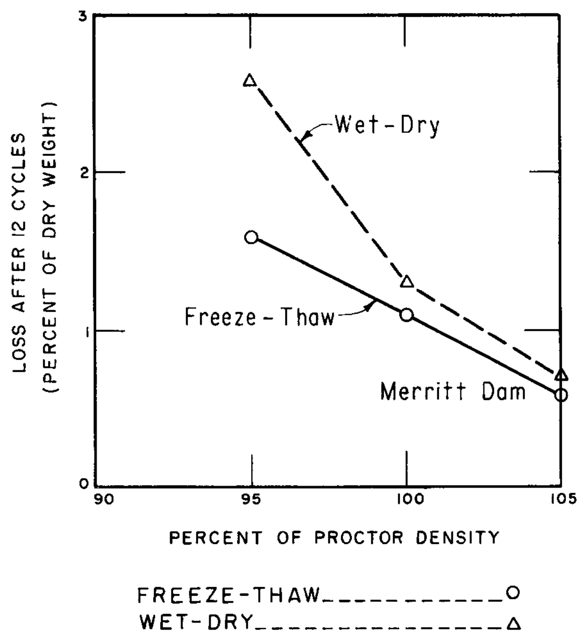

Figure 10. Effect of density on durability of soil-cement.

Figure 11 shows variations in compressive strength due to variations from optimum water content. Generally, the highest strengths are obtained at or near the optimum water content for the soil-cement mixture. During construction, the material is usually placed as near optimum as possible. However, as discussed later, placement conditions slightly dry of optimum usually work better. The general trends shown on Figure 11 and the more detailed results on pages 89 through 91 of the Appendix indicate that compaction of 1 to 2 percent dry of optimum does not greatly reduce the strength. Sufficient water would normally be available for complete hydration of the cement up to 4 percent or more dry of optimum. Therefore, the reduction in strength for mixing dry of optimum must indicate that the cement is not as thoroughly mixed at the lower water contents. Mixing above optimum water content probably increases the water-cement ratio to a point that it begins to reduce the strength.

It is impossible under construction conditions to compact the material immediately after mixing. Since the hydration of cement is a time-dependent reaction once contact is made with water, some change in the properties of soil-cement with time delay would be anticipated. Pages 92 and 93 of the Appendix show the results of compressive strength tests run on materials which were mixed and then had some time delay before being compacted. Some decrease in strength was found on the time delay specimens which was probably due to breaking down the aggregation of particles after some initial set. Figure 12 shows results at time delays of 1/4, 1/2, 1, 1-1/2, and 2 hours of material at 90° F temperature. Even at this elevated temperature, the greatest decrease seems to occur after 1/2 to 1 hour. This is confirmed by the data on Figure 13 which summarizes the results of compaction test results after

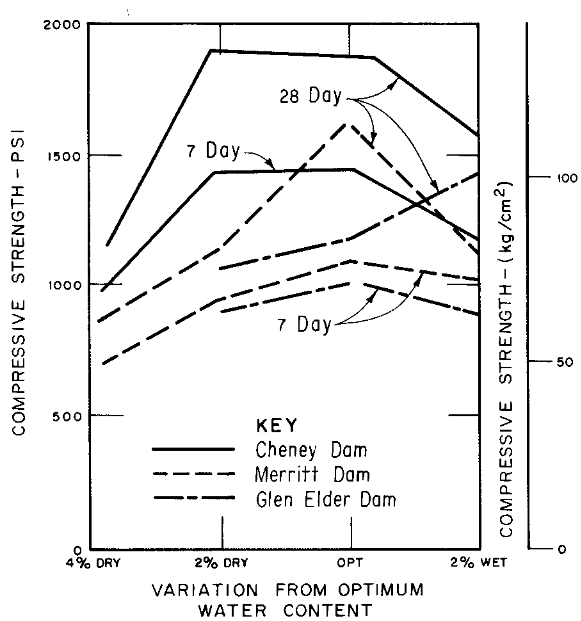

Figure 11. Effect of placement water content on compressive strength of soil-cement.

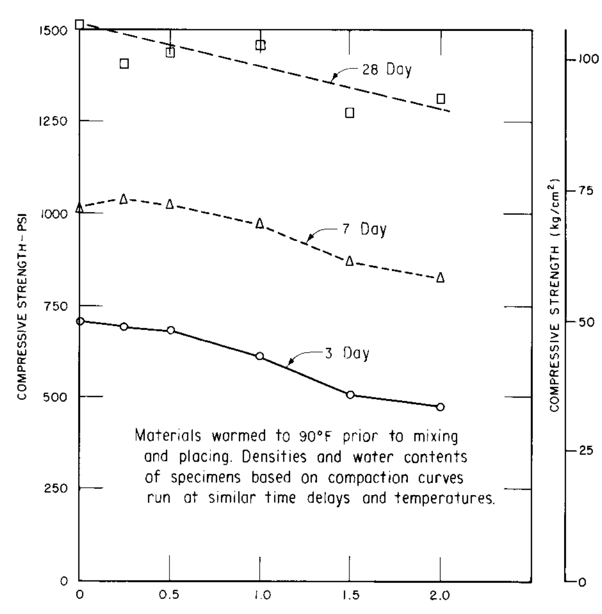

Figure 12. Effect of time delay on compressive strength of soil-cement.

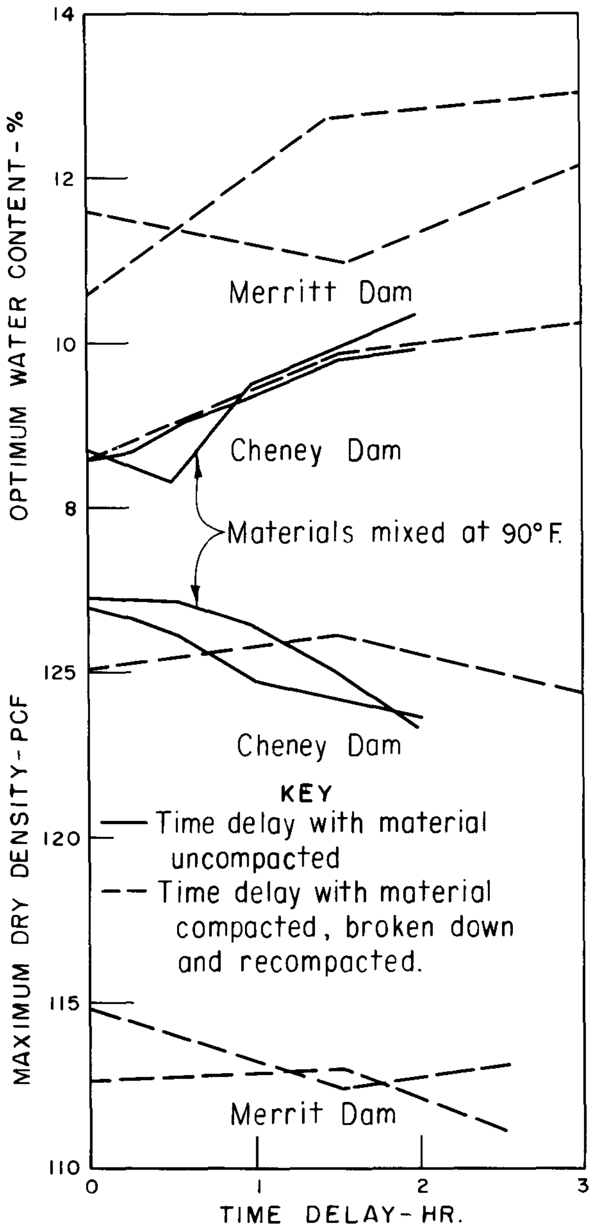

Figure 13. Effect of time delay on compaction of soil-cement.

time delays up to 3 hours. Most of these results show very little effect up to 30 minutes and some of the tests show little effect up to 1-1/2 hours.

CONSTRUCTION PROCEDURES

Detailed descriptions of the construction procedures used on Bonny Test Section are given in EM-652[1] and EM-630[2]. The pictorial summary of the Bonny Test Section construction in Figures 14 through 21 is presented to contrast the procedures used on the test section to those used on other features.

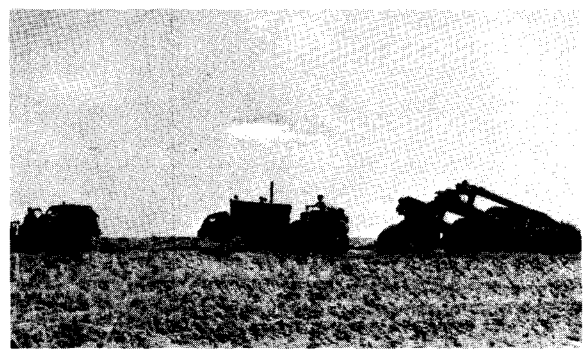

Figure 14. Spreading of untreated soil from the borrow area preparatory to constructing each new soil-cement layer, Bonny Test Section. Photo P331-700-31

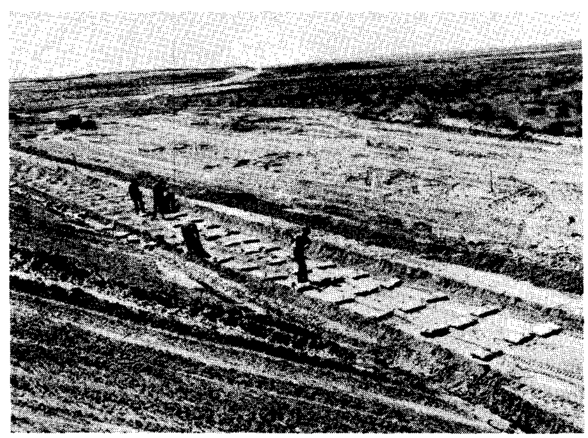

Figure 15. Dumping of portland cement from sacks which had been spaced to provide the required percentage of cement for the soil-cement, Bonny Test Section. Photo P794-701-549

Figure 16. Distribution of the dumped cement on soil layer with a spiked-tooth harrow prior to the start of the mixing in-place operation. Bonny Test Section. Photo P704-701-550

Figure 17. Mixing cement with soil by a tractor-drawn, self-powered, rotary-type mixer. Bonny Test Section. Photo P794-701-557

Figure 18. Applying water to soil-cement prior to wet mixing. Bonny Test Section. Photo P794-701-559

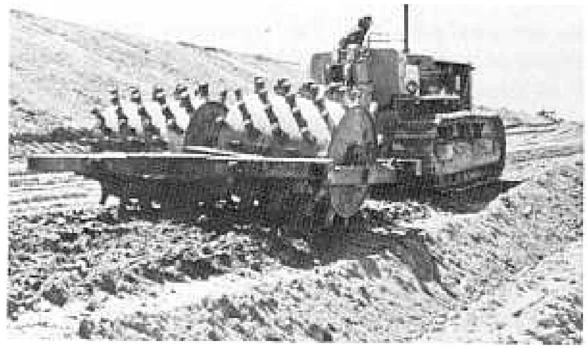

Figure 19. Initial compaction of the soil-cement with a sheepsfoot roller. Bonny Test Section. Photo P794-701-561

Figure 20. Final compaction by pneumatic-type rolling provided by a loaded truck. Bonny Test Section. Photo P331-700-26

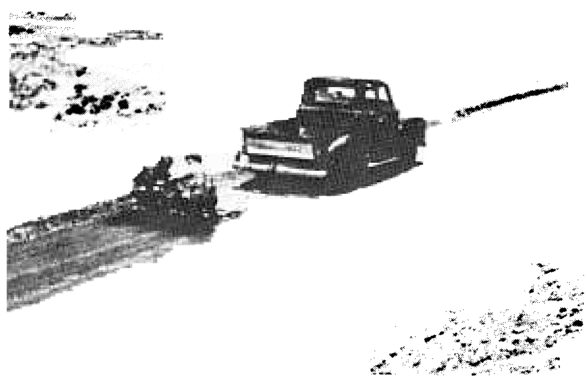

Figure 21. Light surface scarification of the completed soil-cement layer by a spiked-tooth harrow to increase bond to next layer to be placed. Bonny Test Section. Photo P331-700-27

Material Excavation and Stockpiling

The samples submitted for laboratory testing were considered to be representative of the material source to be used in the construction of soil-cement. Test results on the material submitted are used to determine the amount of cement to be added to produce satisfactory soil-cement. Therefore, during the construction of the facing, it is important that the material used is uniform and similar to that tested during the investigation program.

Since almost every natural deposit is variable to a certain extent, the construction procedures used must make some provision for mixing the material. Selective excavation in the borrow area and mixing the material in the stockpile is probably the most practical means of accomplishing this. If the material varies with depth, a full face cut should be made with the excavation machinery. However, if the material varies with the lateral extent of the borrow area, loads from alternate spots in the borrow area should also be mixed. After the material has been excavated, mixing can generally be accomplished by stockpiling and crossbucking the stockpile. For example, part of the stockpiling operating at Lubbock Regulating Reservoir is shown on Figure 22. The soil had been excavated at the borrow area and stockpiled there. It was then trucked to the reservoir site and dumped at the base of the stockpile to the left of the picture. The soil was then pushed up the stockpile with a bulldozer and the soil feed was charged at the top of the stockpile by the front-end loader. This operation resulted in a soil feed which was uniform in gradation and moisture content. Similar operations have been used on other features to achieve mixing.

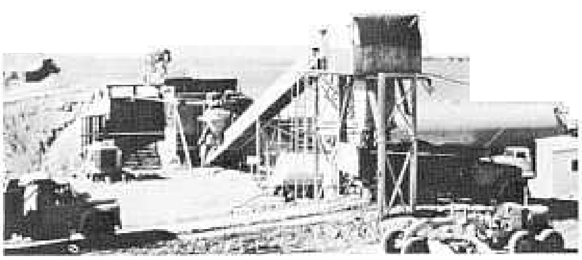

Figure 22. General view of soil stockpile and pugmill for mixing soil-cement—Lubbock Regulating Reservoir. Photo P719-D-58953

The borrow areas used by the Bureau to date have been above water table so the material could be excavated by normal excavation machinery. Materials below water table could be excavated by dragline or dredging equipment, but additional processing and mixing might be required. Materials from below water table would have to be stockpiled long enough to allow the excess water to drain. The mixing of the cement into the soil is generally more efficient if the soil is fairly dry so that soil and cement are mixed before the water content is brought near optimum.

Removal of oversize clay balls, if necessary, prior to proportioning the soil will result in a more consistent soil feed but it is more difficult to do during the stockpiling operation. On most jobs, the scalping screen used to remove these clay balls is placed right after the soil proportioning device. If the soil contains a fairly constant amount of clay balls, the soil feed past the scalping screen should still be fairly constant. However, if the clay ball content varies appreciably, the soil feed past the scalping screen may also vary.

Proportioning and Mixing of Materials

Proper proportioning and mixing of soil, cement, and water is essential to producing a consistent soil-cement. The specifications for soil-cement construction require that the material be mixed in a stationary mixing plant. Either continuous flow or individual batching types are permitted, but all the soil-cement construction to date has used the continuous flow type of plant.

A schematic drawing of a typical proportioning and mixing operation as shown on Figure 23 and the main components are discussed in the following paragraphs.

Different devices have been used or attempted to provide a constant flow of soil to the mixing plant. The most successful device seems to be the reciprocating plate feeder and it has been used on most of the Bureau soil-cement jobs. As the name implies, the soil is fed by a plate which moves back and forth. On the forward stroke, the plate carries a ribbon of soil out through the orifice at the front of the feeder; as the plate makes its backward stroke, the soil on the plate is discharged onto the conveyor belt. Since the plate feeder discharges soil on only half of its cycle, there is some variation along the conveyor belt. On some projects, this variation has been leveled off by using a hopper on the conveyor belt which levels the material into an even ribbon of soil. A variation of the reciprocating plate is the double reciprocating plate feeder. This device feeds from a double bin so that when one side of the feeder is discharging soil, the

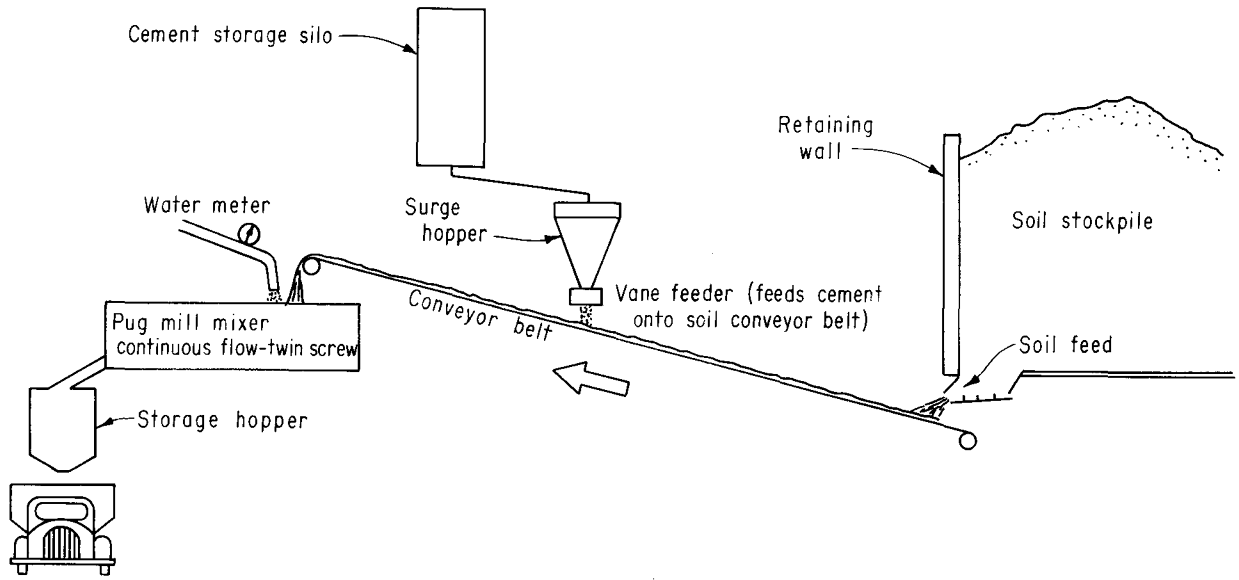

Figure 23. Soil-cement processing.

other side is not and vice versa. Therefore, the total soil flow is a more constant flow.

Some attempts have been made to use a simple strike-off gate on a conveyor belt to proportion the soil. Although this should produce a constant flow of soil, it has not worked well in practice. Soil clods tend to hang up at the gate and restrict the opening and at times the soil seems to roll or ball as it passes the gate resulting in a smaller ribbon of soil on the belt.

Another device used to feed the soil was a tread feeder. This is quite similar to the strike-off gate on a conveyor belt except it has flights or treads instead of a smooth belt. This device was used on Downs Dike construction and worked satisfactorily.

Since these devices are all essentially volume measuring systems, it is apparent that the soil supply to the feeder must be uniform if a constant weight per minute is to be delivered. Variations in soil gradation result in variation of the density of the soil being discharged. Variations in moisture content, even on soils of similar gradation, also result in density variations due to the bulking effects of moisture on sandy soils. This variation causes soil-cement wetter than desired since a lower weight of dry soil at a higher water content is delivered and water at a constant rate is added in the mixer.

A vane feeder has been used to proportion the cement on the features built by the Bureau. This device has a rotating cylinder with compartments formed by 12 vanes on the circumference. The feed rate can be changed by changing the speed at which the cylinder turns. Since this device is also a volume measuring system, it is apparent that the cement must be delivered to the vane at a constant density. The most effective way of doing this seems to be with a small surge hopper above the vane feeder. The level of cement in the surge hopper is kept fairly constant with use of a pressure sensitive switch in the surge hopper to regulate the flow from the main cement silos. Aeration is used in the surge tank to minimize arching of the cement and permit smooth flow.

Figures 22 and 24 show mixing plant setups for Lubbock Regulating Reservoir and Cheney Dam. As shown on the figure, the cement is usually added to the soil on the conveyor. A small plow at the vane feeder gives a furrow for the cement and this furrow is blinded after the cement has been added to prevent the wind from blowing the cement off the belt. Figures 22 and 24 can be compared to Figures 14 and 15, to see the difference in construction procedures from Bonny Test Section.

Since the soil feed and cement feed measure volumes, they must be calibrated to proportion the material by weight. Both feeds are calibrated running material through them and weighing the soil and cement separately. The cement vane feeder can be calibrated over the range of cement supply necessary by varying the speed of rotation. This allows adjustments for

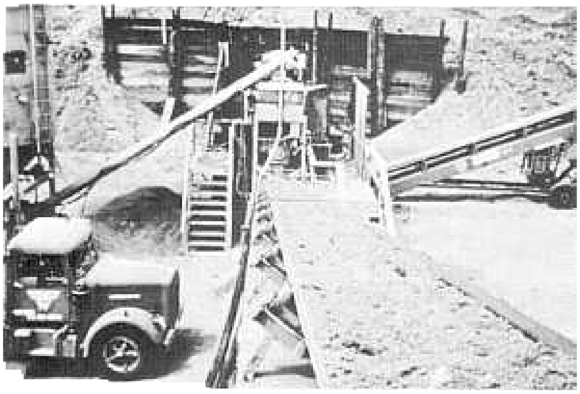

Figure 24. Cement vane feeder and soil feed belt to pugmill—Cheney Dam. Photo P719-D-58954

slight variations from time to time in the soil feed rate. The specifications require that the measuring devices should be accurate to within 2 percent. The vane feeder is usually calibrated at the beginning of the job and checked occasionally as the job progresses. Check calibrations do not normally change the calibration curve. The soil feed is calibrated at the beginning of the job and usually checked a number of times a day during the job. These additional checks are obtained as part of the construction control procedures and they will be explained later in that part of the report.

The soil and cement are charged into the mixer. The mixer used on all the Bureau jobs to date has been a twin shaft pugmill. Other types of mixing equipment would probably also perform satisfactorily. The inside of a pugmill mixer is shown on Figure 25. The shafts rotate in opposite directions and the soil-cement is moved through the mixer by the pitch of the paddles.

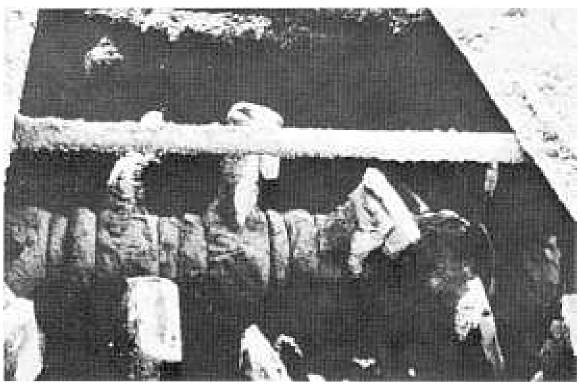

Figure 25. Inside of pugmill, shafts turn in opposite directions while spray bars supply water—Cheney Dam. Photo P835-D-58951

Although the specifications usually require a minimum mixing time of 30 seconds, a shorter mixing time seems to mix the materials sufficiently in most cases. As long as the soil, cement, and moisture are thoroughly mixed into a homogeneous material, shorter mixing times are accepted.

As noted on Figure 25, the water required to bring the soil-cement to the desired water content is supplied in the pugmill. Generally no water is supplied at the front of the mixer so the soil and cement are mixed prior to adding water. The water is sprayed onto the soil-cement as it is being mixed and water is very well distributed by the time the soil-cement leaves the mixer. The amount of water to be added during mixing can be determined from the water content of the soil and the desired water content of the soil-cement. If the mixing plant has an accurate water meter, adjustments of the water content are quite easily made. Satisfactory moisture control has been exercised on some jobs without a water meter by making approximate adjustments of the waterline valve opening.

Figures 16 through 18 show the mixing operation on Bonny Test Section.

Transporting and Placing

As the material is mixed, it is discharged into trucks for transportation to the placement area. On Bureau jobs, the mixing plant has been located close to the placement area which results in a short haul time. Therefore, special protective measures for the soil-cement being transported are usually not required. On all Bureau jobs to date, the truck hauling the soil-cement has been driven up on the previous lift of soil-cement to dump the material. This requires the construction of temporary approach ramps to the layer being placed. Care must be exercised at the top of these ramps so the ramp does not get so thin that it does not protect the previous layers of soil-cement. Current specifications require at least an 18-inch (0.46-m) thickness at the top of the ramp to prevent the heavily loaded trucks from breaking up the edge of the previous lifts.

The fresh soil-cement is dumped into a spreader which is pushed with a crawler tractor. The use of the spreader results in a smooth lift of uniform width and depth. Figure 26 shows a schematic diagram of the placement operation. Figures 27 and 28 show pictures of the spreading operation and the resulting loose lift. This type of spreading equipment has been used on all to date in order to maintain close control of lift thickness and width.

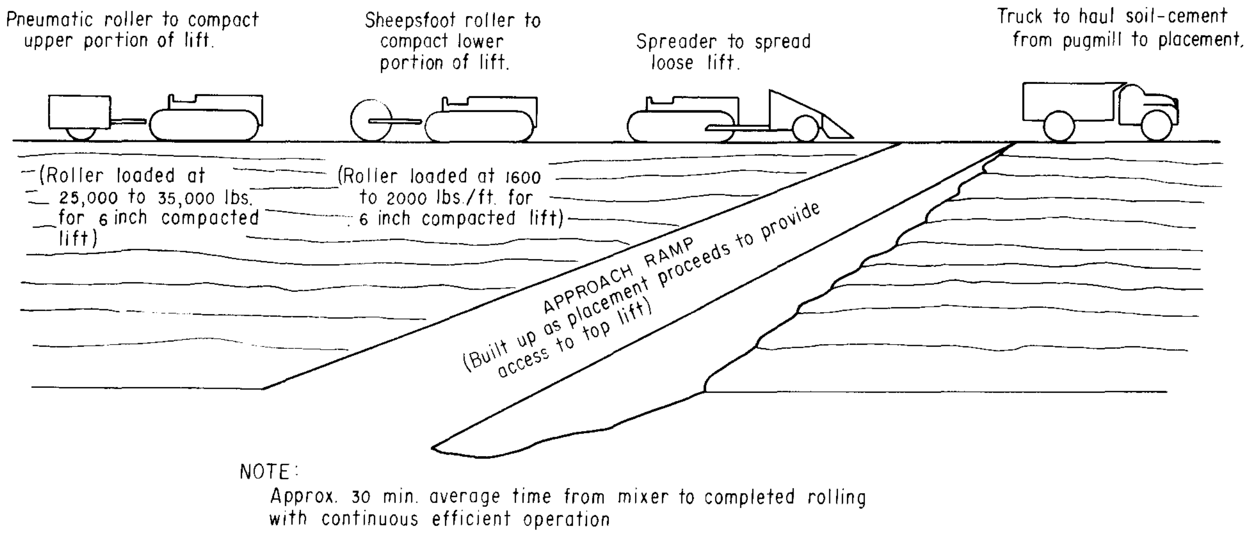

Figure 26. Soil-cement placing operation.

The soil-cement facings have been specified as a horizontal width of 8 feet (2.4 m) or normal thicknesses of 24 or 36 inches (0.61 to 0.91 m) in some cases. This results in a rather narrow working area for operation of construction equipment without considerable overbuild. In order to increase the working width, the contractor is permitted to place the material on a slope not to exceed 8:1. Slopes flatter than 8:1 are used in most cases as it is not necessary to use an 8:1 slope to obtain working widths of over 10 feet (3.0 m). For example, if the slope of the dam is 3:1, a placement slope of 10:1 gives a working width of about 11 feet (3.4 m) which is adequate for most construction equipment. A slight slope on the placement area results in a greater working width without endangering the construction personnel or equipment.

Compaction

Since cement hydration changes the characteristics of fresh soil-cement quite rapidly, the compaction must be accomplished as soon as possible after the material is spread. The specifications require that the material be spread within 30 minutes after mixing. After the material is spread, compaction must be completed within 1 hour and cannot be left more than 30 minutes without having some operation performed on it.

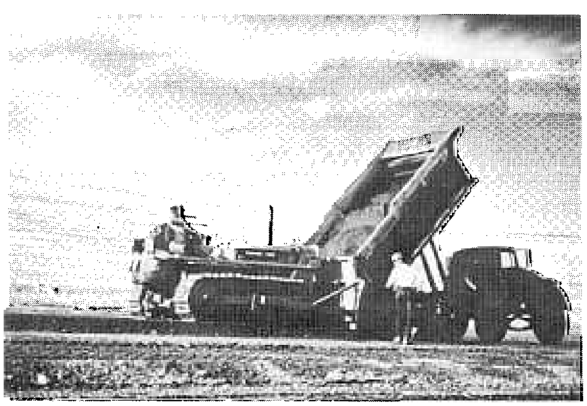

Figure 27. Spreading soil-cement on placement area—Merritt Dam. Photo P719-D-58952

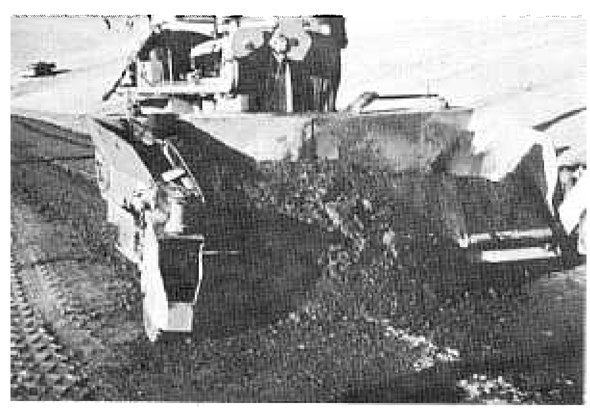

Figure 28. Closeup view of spreader box used to place even, uniform lift—Merritt Dam. Photo P719-D-58949

On compacted earthwork, successive layers are compacted together by the use of tamping rollers. However, on soil-cement construction, the entire layer must be compacted before the time limits stated above are exceeded. In almost all cases, this requires that the entire lift be compacted because successive lifts cannot be placed rapidly enough to compact them together. This has been accomplished by a combination of sheepsfoot and pneumatic-tired rolling on all Bureau jobs except the Merritt Dam Modifications and some test sections on other jobs.

The sheepsfoot rolling is used to compact the lower portion of the lift. The roller weights must be adjusted at the beginning of the job to produce the best compaction with the material being used. The best compaction seems to be when the sheepsfoot begins to walk out toward the end of the required number of passes. At this weight, the roller is heavy enough to compact the material but not so heavy that it continues to penetrate the material compacted on previous passes. The required weight per foot depends on the material type and lift thickness. On Glen Elder Dam, Cawker City Dike, and Starvation Dam, a self-propelled roller with 11-inch-long (27.8-cm) tamping teeth and pneumatic front tires was used. This roller is shown on Figure 32. Because of the longer tooth length, a thicker lift was used at roller weights of 2,600 to 3,950 pounds per foot (3,870 to 5,880 kg/m). On the other jobs, a 6-inch (15.2-cm) compacted lift was used and the roller was loaded from about 1,600 to 2,000 pounds per foot (2,380 to 2,980 kg/m). These weights probably correspond to the 200 psi (14.1 kg/sq cm) knob pressure used on the Bonny Test Section.[1] These rollers were towed by a crawler tractor and are shown on Figures 29, 30, and 31.

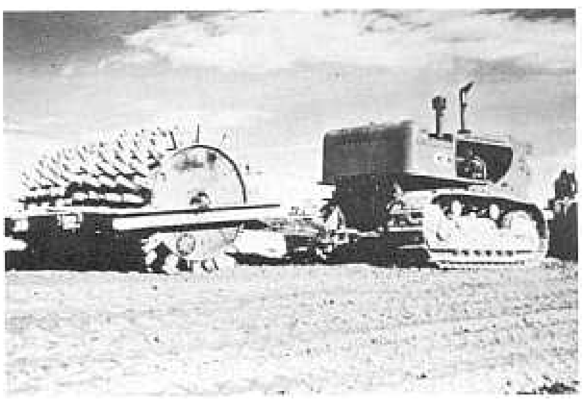

Figure 29. Sheepsfoot roller used to compact lower portion of the lift—Merritt Dam. Photo P719-D-58950

A pneumatic-tired roller is used to compact the upper portion of the lift. As with the sheepsfoot roller, the weights must be adjusted at the beginning of the job to obtain the best compaction. The weight of the roller is usually increased to just under the weight which causes the material to squeeze or creep under the roller. If the roller cannot be loaded that heavily, the weight is considered adequate if acceptable densities are being obtained. A towed-type roller such as shown on Figure 30 was used on Merritt Dam and Cheney Dam but on

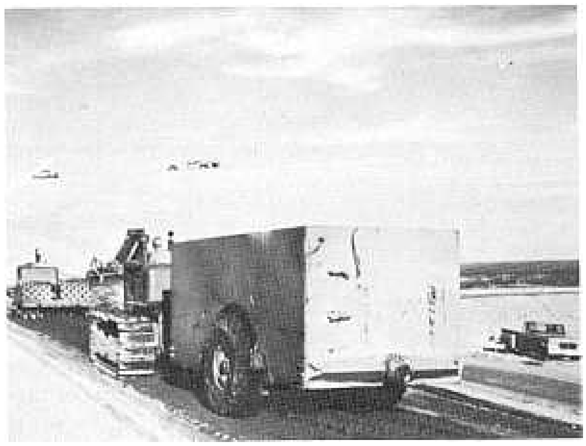

Figure 30. Pneumatic roller used to compact upper portion of lift—Merritt Dam. Photo P719-D-58947

the rest of the jobs, self-propelled rollers were used. The towed rollers were large four-wheel rollers and were loaded to about 28,000 pounds (12,700 kg) total load. The self-propelled rollers were two-axle types with the wheels on one axle in staggered positions relative to the wheels on the other axle. Rollers of this type are shown on Figures 31, 32, and 33. The smooth wheels and the overlap design on the self-propelled rollers result in a smoother finished surface than with the towed roller. Note on Figure 30, the slight ridges of soil-cement caused by the tire tread dry rapidly and also interfere with the cleanup in preparation for the next lift. The loaded weight of the self-propelled rollers varied from 25,000 to 35,000 pounds (11,400 to 15,900 kg) which is in the same range as the total weight used on the towed rollers. The self-propelled rollers were 7- and 11-wheel rollers so the wheel weight was somewhat less.

The results of placement moisture at or above optimum can be seen on Figure 31. The rutting during pneumatic-tire rolling shown behind the roller is usually due to placement water contents near or above

Figure 31. General view of placing and rolling operations—Lubbock Regulating Reservoir. Photo P662-525-5763

optimum. The fine sand materials which have been used for soil-cement become somewhat spongy at that water content and make it difficult to compact them to a smooth surface. The surfaces shown on Figures 32 and 33 are much smoother and show little evidence of rutting or surface cracking. The pneumatic rolling on Bonny Test Section was accomplished with a loaded truck (Figure 20).

Figure 32. General view of placing and rolling operations—Cawker City Dike. Photo P495-731-460

As previously stated, the soil-cement on the Merritt Dam Modifications was compacted by pneumatic-tire rolling only. This modification was made on the right abutment which had not been protected by soil-cement as part of the original construction. The slope of the upstream face is 10:1 and it was covered with two 6-inch (15.2-cm) layers of soil-cement. This is the only slope facing constructed by the Bureau which was laid parallel to the slope surface instead of in the stairstep fashion up the slope as shown in Figure 32. To avoid joints, the soil-cement was laid as a series of continuous slabs. Before the time had expired for compaction of one strip of soil-cement, another strip was laid next to it so the two could be compacted together (Figure 33). No specific difficulties were encountered in compacting the material with pneumatic compaction only.

Figure 33. Placing and rolling second lift—Merritt Dam upstream slope modification. Photo P719-701-7

Two test sections were constructed on Glen Elder Dam to evaluate the use of pneumatic rolling only. The first section was compacted with the same procedures as used on the rest of the job. That is: 11-1/2-inch (29-cm) loose lift compacted to about 8 inches (20 cm) with eight passes of the sheepsfoot roller and six passes of the pneumatic-tired roller. The second section was placed with a 9-inch (23-cm) loose lift compacted by 10 passes of the pneumatic-tired roller. The following conclusions were made on these test sections: (1) there was very little difference in the densities obtained, (2) pneumatic-tired rolling only resulted in more creep and more wheel rutting, (3) more cleanup was required due to wheel rutting and greater number of lifts, (4) little difference in the bond strengths was evidenced in record coring, and (5) because more time is required for the greater number of lifts, cost savings by using pneumatic rolling only are minimal. This last conclusion was probably true in the case of thicker lifts used on Glen Elder Dam but probably would not be true where a 6-inch (15-cm) lift was used with the smaller sheepsfoot rollers.

Curing and Preparation for Next Lift

The specifications have required that the soil-cement surfaces be kept continuously moist until the next layer of soil-cement was placed. However, moist curing in excess of 7 days was not required. Side slopes of permanently exposed material were required to be kept moist for 7 days. Sealing compounds or moist earth covers have also been used to maintain a moist condition for 7 days on permanently exposed surfaces.

Surfaces which are to receive an overlying layer of soil-cement must be kept clean and moist in order for the next layer to have an opportunity to bond. These surfaces are normally cleaned by power brooming to remove all loose and uncemented material ahead of the next placement, as shown on Figure 34. Usually by the time this brooming was done, the layer being cleaned had set sufficiently so the broom did not affect the surface of the soil-cement. Laboratory testing seemed to indicate that removing the compaction plane at the top of the lift would increase the bond strength. This removal would result in a surface similar to a fresh-cut surface and could be compared to removing the laitance layer in concrete work. Brooming was specified after compaction to remove the compaction plane on Glen Elder Dam, Downs Dike, Cawker City Dike, and Starvation Dam. The time of the brooming was dependent on the set time of the soil-cement and was done when the soil-cement had set sufficiently to prevent removing a large amount of material.

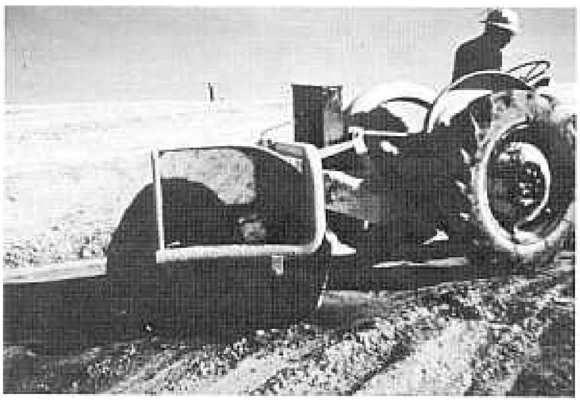

Figure 34. Power brooming to clean surface prior to placement of next lift—Lubbock Regulating Reservoir. Photo P719-D-58948

Figures 35 and 36 show watering operations using fine sprays to keep soil-cement moist. The use of excessive water, especially immediately after placement and until initial set, could be detrimental to developing bond to the next lift. Applying excess water at this time can be envisioned as increasing the water-cement ratio of the fresh soil-cement or possibly washing the cement from the soil particles near the surface.

Figure 35. Watering inplace soil-cement—Starvation Dam. Photo P66-D-66533

Figure 36. Watering inplace soil-cement with side boom spray—Lubbock Regulating Reservoir. Photo P719-D-66534

CONSTRUCTION CONTROL PROCEDURES

Proportioning and Mixing

The most critical construction control part of the mixing operation is to ascertain that the materials are being proportioned properly. Gradation tests are performed on the stockpile material in addition to visual observation of the excavation and stockpiling operations to see that uniform material of the desired gradation is being delivered.

The soil feed and cement feed are set to proportion the cement to the required percentage based on the dry weight of soil. As previously stated, the calibration curve on the cement vane feeder can be used to adjust the cement feed. Setting the soil feed is generally subject to greater variations since gradation or moisture changes in the soil can cause variations. As long as the same material and feeder setting is maintained, the feed rate will stay about the same. The soil feed rate can be obtained by running a timed load of soil only through the plant and weighing it. This requires shutting down production and is usually done only at the beginning of the day or at other times when production is disrupted. A check on the soil feed rate can be obtained by weighing a timed truck load of soil-cement. The dry weight of the material can be determined by obtaining the water content of the material and the amount of cement in the soil-cement is obtained from the cement feed rate in pounds per minute. This calculation assumes, of course, that the cement feeder is feeding the amount obtained from the calibration curve but this seems to be a reasonable assumption if calibration checks show the device is reliable. These checks can be made at any time but they are usually made at the time density and compaction tests are made.

Calibration and check tests on the soil feed require moisture content determinations whether soil only or soil-cement weights are obtained. In order for the tests to have any benefit for control purposes, this moisture content must be available quickly. Hotplate moistures have been used in some cases and the carbide moisture tester has also been used. This device measures the amount of acetylene gas produced by the reaction of the water in the soil with calcium carbide reagent added to the soil. This device is fast and has been found to be satisfactory for control purposes.

Visual observation of the soil-cement as it is discharged from the mixer is normally adequate to determine if it is properly mixed. The soil-cement should be uniform in color and texture. Very little difficulty has been encountered in obtaining thorough mixing.

Density and Compaction Testing

Density tests are made in the compacted material in the same manner as for normal earthwork. However, since the properties of the soil-cement material are time dependent, the compaction test is not performed on the material taken from the density hole. Material for the compaction test is obtained before the material is compacted on the placement. The spot where the material was obtained is marked so the field density test can be performed at the same location. If the material is obtained at the mixing plant prior to spreading, the spot is marked where the truckload is spread. A Rapid Method Compaction Test is performed on the material at about the same time the material is being compacted on the placement. The field density test results are obtained after compaction and compared to Rapid Method Test results to obtain the percent compaction and variation from optimum.

Compressive strength test specimens are remolded to the fill wet density as soon as the fill wet density is available. The material for these specimens is obtained at the same time the compaction material is obtained and stored in a sealed container until the density test results are available.

Chemical Cement Content Determination

Since the proportioning of the soil and cement are subject to some variations, some check on the cement content of the soil-cement as mixed was desired. A study by the Applied Sciences Branch indicated that a volumetric determination of the total calcium was the most rapid.[17] This method requires the determination of the amount of calcium in the soil and cement as well as the soil-cement. It is only recommended where the soil being used has small amounts of acid-soluble calcium.[17]

Chemical cement content determinations were made on the features listed in the Introduction except Merritt Dam Modification and Starvation Dam. These tests were not performed at Starvation Dam due to the high salt content of the soil. The calcium content of the soils used at Lubbock Regulating Reservoir and Downs Dike also had high calcium contents and the results of the chemical cement content tests showed greater than normal variations.

Construction Control Reports

Construction control testing is reported on Form No. 7-1737. This form gives the location of each record test and the results of the compaction and density tests. The time of plant mixing, elapsed times for completion of laboratory and field compaction, and field density test are recorded. The results of compressive strength control specimens are also recorded as they become available. Some of these results are not available until after the report has been submitted and subsequent reports are therefore submitted. An example of the report form with data recorded is shown in the Appendix.

A brief summary of construction control results from the facings constructed by the Bureau are summarized on Table 2. A more complete summary giving the averages for each month during construction is shown on pages 94 through 95 of the Appendix. A review of these results shows that the density of the soil-cement has averaged 98 to 100 percent of Proctor maximum

Table 2

SUMMARY OF CONSTRUCTION CONTROL RESULTS
Weighted Averages from Entire Job

Feature	Percent Proctor Density	Variation from optimum (percent)	Compressive strength (psi)	
			7-day	28-day
Merritt Dam	102.1	0.3 dry	1,360	1,815
Cheney Dam	98.8	0.3 dry	1,199	1,497
Lubbock Regulating Reservoir	100.0	0.3 dry	834	1,134
Glen Elder Dam	100.8	0.7 dry	854	1,071
Down Dike	99.4	0.2 dry	748	1,201
Cawker City Dike	99.7	0.4 dry	962	1,287
Merritt Dam Modification	100.0	1.0 dry	1,006	1,556
Starvation Dam	98.1	1.6 dry	769	905

dry density. The water content has ranged mostly from optimum to 1.0 percent dry of optimum water content. These control results show that the construction procedures used have resulted in satisfactory placement conditions. The averages of the 7- and 28-day compressive strength specimens are also shown on Table 2. As previously stated, these specimens are placed at fill moisture and density conditions. Therefore, a comparison of the construction control test results should give an indication if the constructed soil-cement has the strength anticipated in the investigations stage. These averages are also plotted on pages 69 through 79 of the Appendix so they can be readily compared to the test results of the investigation stage.

The construction control specimens showed somewhat higher strengths than anticipated from the initial laboratory testing for Merritt Dam, Cheney Dam, and Starvation Dam. The construction control results from the other features show strengths about as anticipated or somewhat less. Some of the lower strengths might result from a time delay from mixing to compacting as indicated on Figure 12. The general conclusion to be drawn from construction control results is that soil-cement of at least the quality indicated by the investigation tests has been produced by the construction procedures used.

RECORD CORING

After the completion of the soil-cement facings, record core holes were drilled on all the features except the Merritt Dam Modifications. Observation and testing of these cores allows an evaluation of the soil-cement as it actually exists in the facing. The drilling also determines the thickness of the soil-cement facings at the drill hole locations. On most of the features, reference points have been established at the record core hole locations. A steel reinforcing rod set flush with the surface and grouted in place gives a quick visual determination of the amount eroded from the surface of the soil-cement since the reference point was installed. These reference points are also used as a basis for detailed profiles of the soil-cement surface. If later inspections of the soil-cement indicate excessive wear at locations other than the reference points, an estimate of the wear can be made by comparison to the original profiles.

Unconfined compression tests and durability tests are performed on representative sections of the record cores. These test specimens are normally soaked in water for at least 7 days prior to the beginning of the test to rewet the soil-cement if it has dried. The core barrel used to obtain most of these samples has a bit which cuts a core approximately 2.8 inches (7.1 cm) in diameter. This is the same diameter used for standard laboratory compression specimens. The record cores selected for compression testing are cut to a length-diameter ratio of about 2 prior to soaking in water. The specimens are capped prior to testing. The specimens selected for durability testing are tested the same as the standard laboratory test specimens after the curing period. The 2.8-inch (7.1-cm) diameter specimen is smaller than the standard 1/30-cubic-foot (944-cc) specimen used for the standard durability test.

Table 3

SUMMARY OF TESTS ON RECORD CORES

Feature	Compressive strength (psi)		Durability losses (percent)	
	Record cores	28-day construction control	Wet-dry	Freeze-thaw
Merritt Dam	930	1,815	0.7	0.8
Cheney Dam	1,241	1,497	0.8	0.9
Lubbock Regulating Reservoir	782	1,134	1.5	4.3
Glen Elder Dam	1,478	1,071	0.7	0.8
Downs Dike	1,665	1,201	0.8	1.6
Cawker City Dike	1,493	1,287	0.5	0.7
Starvation Dam	905	905	0.8	2.3

However, the 2.8-inch (7.1-cm) specimen has about 15 percent more surface area for the same volume and is a more severe durability test.

The average of test results on the record cores for each feature are shown on Table 3, and the average of the 28-day construction-control specimens is shown for comparison. As shown by this comparison, the compressive strengths obtained on the record cores are lower than the 28-day construction-control tests except at the Glen Elder Unit features (Glen Elder Dam, Downs Dike, and Cawker City Dike).

The cores tested all appeared to be in good condition. However, some fine hairline cracking during the coring operation may have caused some reduction in strength without causing visual disturbance. The core holes at Lubbock Regulating Reservoir were drilled normal to the slope which caused the core to be taken at an angle to the top of each soil-cement layer. This may have contributed to the low strength of these record cores.

The results of the durability tests are also shown on Table 3. A review of these test results shows that the average losses on the durability test specimens is very low on all the features except for the freeze-thaw tests on Lubbock Regulating Reservoir. The average loss on freeze-thaw tests from the record cores on the slope protection on Lubbock Regulating Reservoir was 4.3 percent, about 1.5 percent higher than that shown on the laboratory investigation testing program. Comparison of the rest of record core test results with the results shown on Figures 4 and 5, or the plots of durability test data on pages 47 through 63 of the Appendix, shows that the record cores in most cases showed losses less than the investigation tests. This indicates that the soil-cement should be resistant to the destructive effects of wetting and drying and freezing and thawing.

A small percentage of the contact planes between lifts encountered during record coring were recovered as bonded lifts. This may not indicate a total lack of bonding between layers but it does indicate the bonds are not strong enough to withstand the stresses caused in drilling. The bonded layers recovered have all been on features where power brooming was used to remove the smooth compaction plane. However, the percentage of bonded layers recovered on these features was too small to be considered conclusive proof that the brooming causes much greater bond strengths. Direct shear tests have been performed on bonded lifts from Glen Elder Dam and Starvation Dam. These test results are shown on pages 100 and 103 of the Appendix and show that on the bonded layers tested, the strength is nearly as high as the material above or below the contact between lifts. This indicates that the drilling may be breaking many of the bonded layers if they are weaker than the rest of the layer and the percentage of layers bonded to some degree may be higher than indicated.

Apparently some bonded layers have been recovered from the Bonny Test Section. The construction procedure used on the section may have concentrated cement near the bottom of the mixing depth if the soil was dry and allowed the cement to "sift" down as the material was being mixed. This may have increased the bonding where the material was mixed to the full depth of the layer. In other cases, wave action undercut the layers and this may have been due to the material not being mixed to the full depth of the layer.[2]

PERFORMANCE

The performance of a product during service is always the final criteria on whether it is acceptable. Most of the soil-cement facings in service to date have performed very well.

Bonny Test Section has now been in place for nearly 20 years. The conclusions stated in Report EM-630[2] still seem to be valid and the satisfactory performance of this test section was the basis for the use of soil-cement on the more recent features. Figure 37 shows a general view of Bonny Test Section in 1966, 15 years after construction. This picture shows the rather irregular stairstep pattern caused by erosion of the improperly compacted material at the edge of the lifts. During construction, an attempt was made to remove these edges by blading but the soil-cement was too hard. Experience has shown that these irregularities are desirable to break up and reduce the runup of waves.[2]

Figure 37. View of Bonny Test Section—October 1966, 15 years after construction. Photo PX-D-58945

Figures 38 through 41 show pictures of the facing at Cheney Dam after 3 years of service. Most of the wear shown on Figures 38 and 39 has been on the edges of the compacted lifts and was anticipated. During the construction on Cheney Dam, Merritt Dam, and Lubbock Regulating Reservoir, the edges of the lifts were compacted by rolling the outside edges first. This resulted in a feathered or triangular edge on each lift and a rather smooth surface on the soil-cement. However, these feathered edges were not compacted as thoroughly as the rest of the layer where the material was more restrained and these feathered edges could be

Figure 38. View of Cheney Dam at Sta. 67+50 showing normally anticipated wear pattern—October 1966, 3 years after construction. Photo P835-D-56104

Figure 39. View of Cheney Dam at Sta. 40+00 showing normally anticipated wear pattern—October 1966, 3 years after construction. Photo P835-D-56102

expected to break off. The photographs show that this was the major part of the wear to the time the features were inspected. The edges of the lifts on Glen Elder Dam, Cawker City Dike, and Starvation Dam were not compacted in the same manner (see Figure 32). The stairstep appearance should develop quite quickly on these features.

Figures 40 and 41 show areas on the Cheney Dam facing which have moderate breakage. This breakage was reported to have occurred during a storm in March of 1966 when the reservoir was about at the level of the eroded lifts (elevation 1413). Winds in excess of 50 mph (80 km/hr) with waves of 6 feet (1.8 m) breaking directly on the soil-cement were reported. A survey of the facing showed that some overbuild had occurred and this was in the area that the overbuild was being

Figure 40. View of Cheney Dam at Sta. 110+00 showing moderate breakage of facing—October 1966, 3 years after construction. Photo P835-D-56108

Figure 41. View of Cheney Dam at Sta. 85+75 showing moderate breakage of facing—October 1966, 3 years after construction. Photo P835-D-56106

corrected. This correction would create cantilevers of somewhat greater length and apparently they were not strong enough to resist the severe wave action. The survey also indicated that there was still a normal thickness of about 2 feet (0.6 m) in the broken areas. Similar breakouts were reported at elevation 1421 during an inspection in 1969. These apparently were not in areas of significant overbuild and these areas were repaired in the spring of 1970. Figure 42 shows the repair procedures used.

Another severe storm occurred in March of 1971 when the water surface in the reservoir was at elevation 1421.4. Although the damage caused by this storm did not appear to be severe above the water surface, an inspection by an underwater team indicated damage below the water surface. It was decided to lower the water surface in the fall of 1971 and make a complete

Cheney Dam, Wichita Project, Kansas. Approximate Station 87+26, area of soil-cement to be repaired. Lake elevation at 1418.41. One-quarter-inch-diameter rebars were grouted into 1/2-inch-diameter holes drilled into the soil-cement. Photo P835-D-68953

Cheney Dam, Wichita Project, Kansas. Protective surface repair on Cheney Dam. Material was 2,500-pound transit-mix concrete shaped to configuration of soil-cement. Photo P835-D-68954

Figure 42. Repair of soil-cement, Cheney Dam.

inspection and repair the damaged areas. The water surface was lowered to elevation 1413 which exposed considerable damage to the soil-cement between elevations 1415 and 1420. Most of this damage occurred between Stations 60+00 and 110+00. These stations are near the center of the dam where maximum wave action might be expected to occur with the winds blowing generally down the length of the reservoir. Another underwater inspection at that time indicated that no serious damage had occurred below elevation 1413. Figure 43 shows a general view of some of the most severely damaged area. As shown on the photo, some of the soil-cement lifts are broken off to a nearly vertical face through the entire facing, and in some places the soil-cement forms an overhang. About 300 feet (91 m) of the soil-cement facing had been completely removed exposing the Zone 1 embankment materials. Figure 44 shows one of these areas where the embankment is exposed. Another 600 feet (183 m) had less than 12 inches (30.5 cm) of soil-cement remaining over the embankment material.

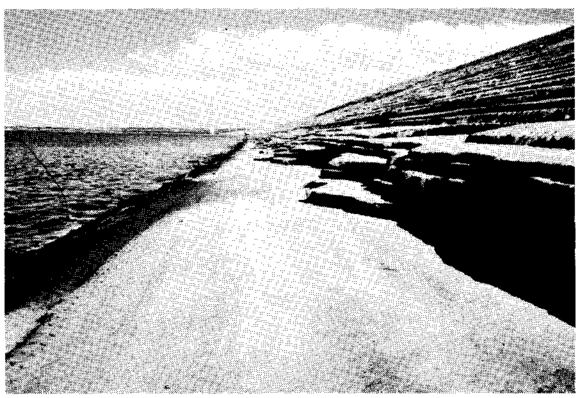

Figure 43. General view of Cheney Dam from Station 88+00 looking east. View shows some of the area of most severe breakage—October 1971, 8 years after construction. Photo P835-D-70305

Figure 44. Closeup view of Cheney Dam at Station 95+90 showing Zone 1 material exposed—October 1971, 8 years after construction. Photo P835-D-70510

The damaged areas of the soil-cement facing are being repaired by about the same procedures shown in Figure 42. Reinforcing steel will be used to tie the patches to the existing soil-cement facing. In addition, reinforcing steel mesh will be embedded in the patch.

Figure 45 shows one of the patches placed in the spring of 1970 at Station 85+80. These patches seem to have survived the storm in March 1971 very well and it was decided to use the same general scheme of repair. It was estimated that the concrete needed to repair the damaged areas would be 2,000 cubic yards (1,530 cu m).

Figure 45. View of patch on Cheney Dam at Station 85+80—October 1971, 1-1/2 years after patch was placed. Photo P835-D-70306.

Figures 46, 47, and 48 show the appearance of Merritt Dam in the fall of 1968, about 6 years after construction. This is considered to be in generally excellent condition. The normal water surface has been at and above the light colored area shown in Figure 46. Below that elevation, the facing had been continually

Figure 46. General view of Merritt Dam, Sta. 18+00 looking west—September 1968, 6 years after construction. Photo P637-D-66530

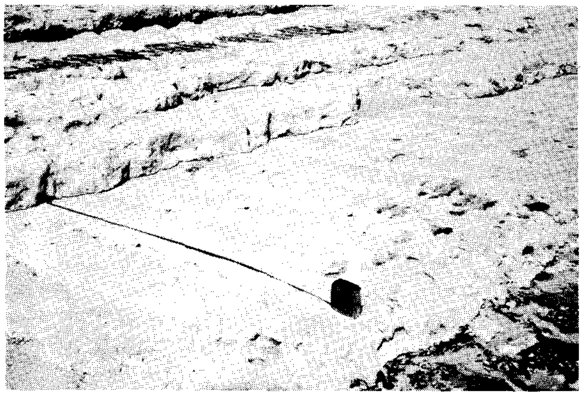

Figure 47. Closeup of Merritt Dam showing erosion of feathered edge—September 1968, 6 years after construction. Photo P637-D-66531

Figure 48. General view of Merritt Dam, Sta. 27+00 looking east—September 1968, 6 years after construction. Photo P637-D-66532

covered by water and almost no erosion had taken place as evidenced by the asphaltic curing compound still being intact on the surface. Above that elevation, the stairstep pattern up the slope was beginning to develop but it did not appear that the erosion had progressed past the feathered edges.

Figures 49, 50, and 51 show the appearance of Lubbock Regulating Reservoir in March of 1968, about 1 year after construction. Very little wear had occurred on this feature. The fetch on this reservoir is much less than on the dams faced with soil-cement and the rubble broken loose on the feathered edges does not get completely swept down the slope. Some weathering in place is shown on Figure 49 but this seems to be superficial and is not considered serious.

Figure 49. General view of Lubbock Regulating Reservoir showing weathered material, Sta. 51—March 1968, 1 year after construction. Photo P719-D-66536

Figure 50. General view of Lubbock Regulating Reservoir, from outlet structure—March 1968, 1 year after construction. Photo P719-D-60589

Figure 51. View of Lubbock Regulating Reservoir showing erosion of feathered edge—March 1968, 1 year after construction. Photo P719-D-66535

Poor bond between layers may also cause somewhat more severe breakage during severe wave action. The bond between layers does not seem to have been very good on the first soil-cement features constructed by the Bureau. As previously mentioned, the removal of the smooth compaction plane at the top of each lift seemed to increase the bonding in laboratory tests. It is too early to tell if this has resulted in significantly increased bonding under field construction conditions. Other means of increasing the bond, such as applying a cement paste between lifts and varying curing conditions, are being investigated.

The short time of exposure on most of the soil-cement facings constructed since Bonny Test Section does not allow a complete projection of performance. Additional evaluations should be made and published as more performance data become available.

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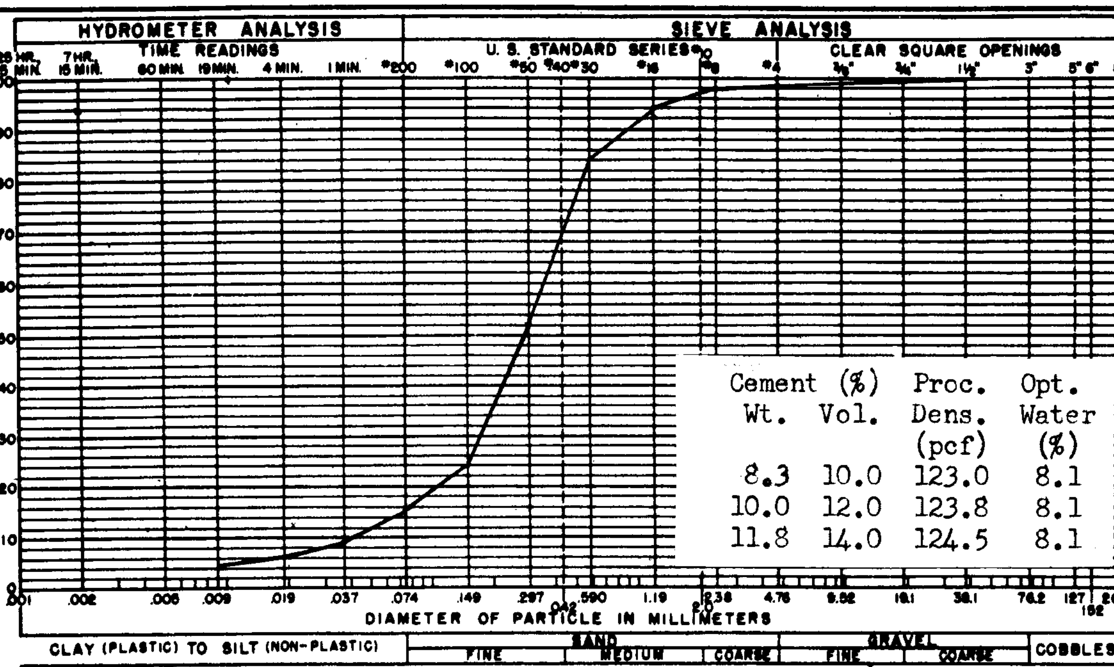
APPENDIX

TEST RESULTS



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
8.3	10.0	123.0	8.1
10.0	12.0	123.8	8.1
11.8	14.0	124.5	8.1

CLASSIFICATION SYMBOL SM

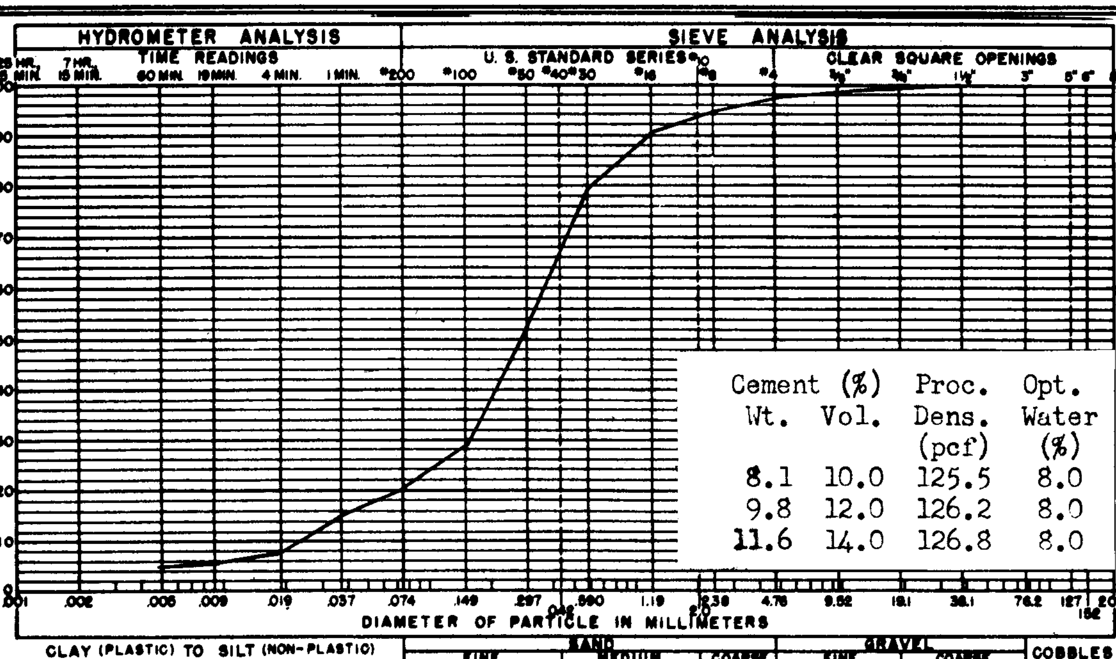
SPECIFIC GRAVITY ______

NOTES: Bonny Test Sect.

ATTERBERG LIMITS
- LIQUID LIMIT ______
- PLASTICITY INDEX ______
- SHRINKAGE LIMIT ______

GRAVEL 1 %
SAND 83 %
SILT TO CLAY 16 %

LABORATORY SAMPLE No. 13N-420 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
8.1	10.0	125.5	8.0
9.8	12.0	126.2	8.0
11.6	14.0	126.8	8.0

CLASSIFICATION SYMBOL SM

SPECIFIC GRAVITY ______

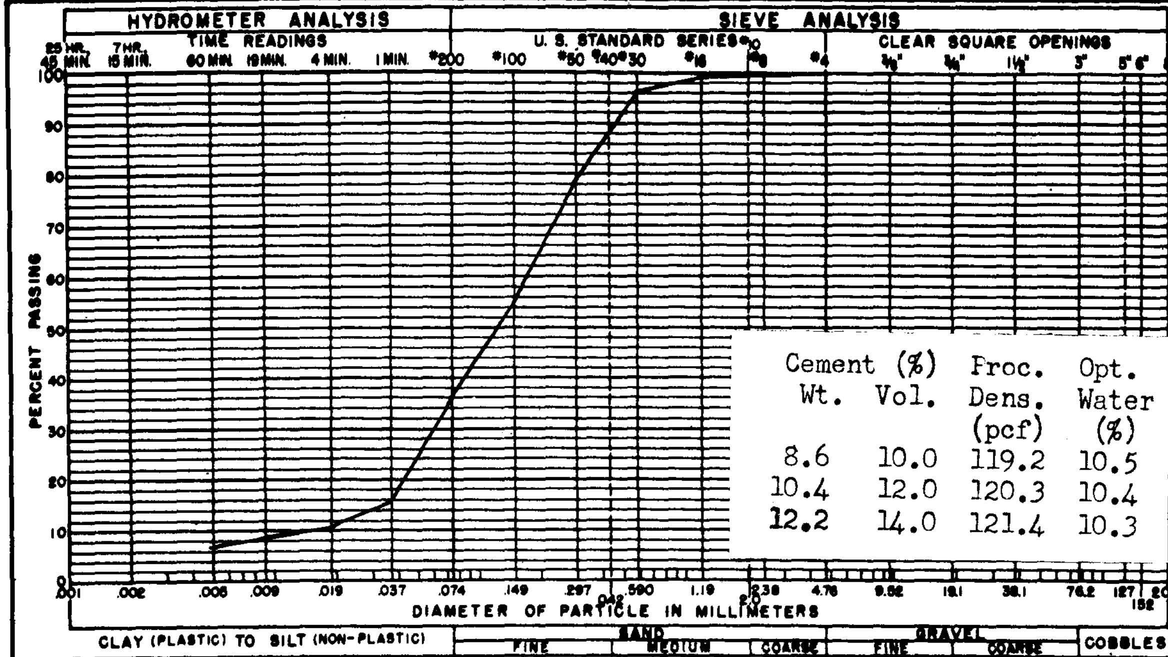
NOTES: Bonny Test Section
Type B

ATTERBERG LIMITS
- LIQUID LIMIT ______
- PLASTICITY INDEX ______
- SHRINKAGE LIMIT ______

GRAVEL 2 %
SAND 78 %
SILT TO CLAY 20 %

LABORATORY SAMPLE No. 13N-421 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.

INDEX TEST SUMMARY



Cement (%) Wt.	Cement (%) Vol.	Proc. Dens. (pcf)	Opt. Water (%)
8.6	10.0	119.2	10.5
10.4	12.0	120.3	10.4
12.2	14.0	121.4	10.3

CLASSIFICATION SYMBOL SM

SPECIFIC GRAVITY ______

NOTES Bonny Test Section Type A

ATTERBERG LIMITS

LIQUID LIMIT ______

PLASTICITY INDEX ______

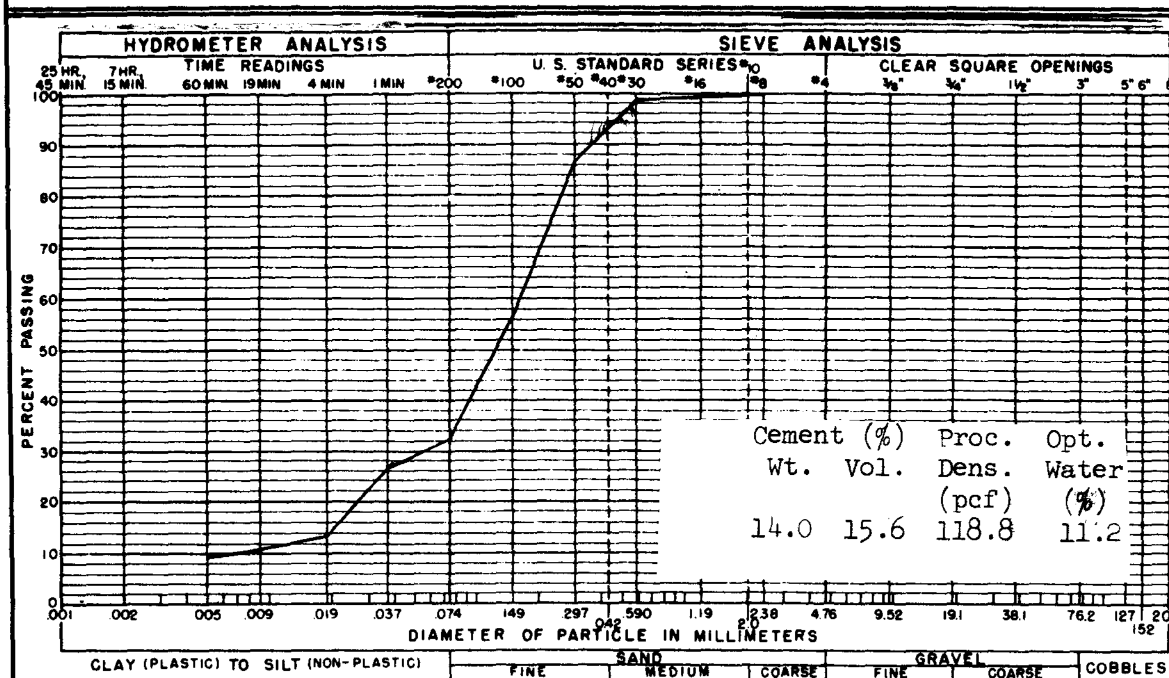
SHRINKAGE LIMIT ______

GRAVEL 0 %

SAND 64 %

SILT TO CLAY 36 %

LABORATORY SAMPLE No. 13N-422 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.



Cement (%) Wt.	Cement (%) Vol.	Proc. Dens. (pcf)	Opt. Water (%)
14.0	15.6	118.8	11.2

CLASSIFICATION SYMBOL SM

SPECIFIC GRAVITY 2.63

NOTES: Merritt Dam during construction

ATTERBERG LIMITS

LIQUID LIMIT ______

PLASTICITY INDEX ______

SHRINKAGE LIMIT ______

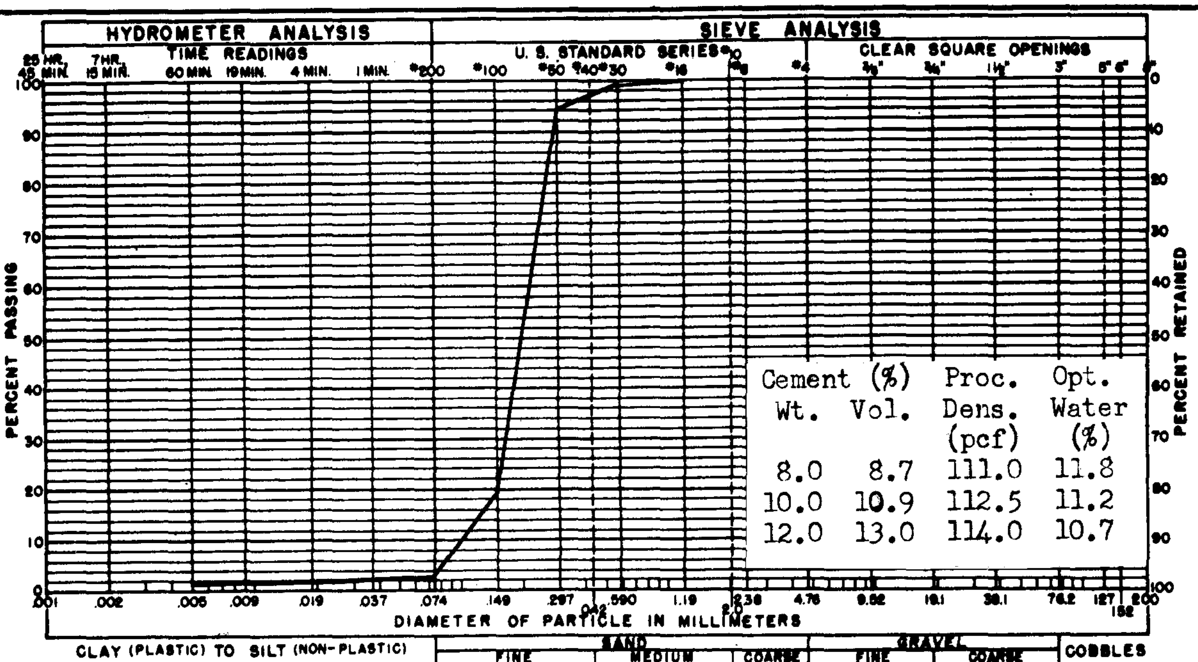
GRAVEL 0 %

SAND 67 %

SILT TO CLAY 33 %

LABORATORY SAMPLE No. 15R-132 FIELD DESIGNATION ______ EXCAVATION No Stockpile DEPTH ______ FT.

INDEX TEST SUMMARY



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
8.0	8.7	111.0	11.8
10.0	10.9	112.5	11.2
12.0	13.0	114.0	10.7

CLASSIFICATION SYMBOL SP

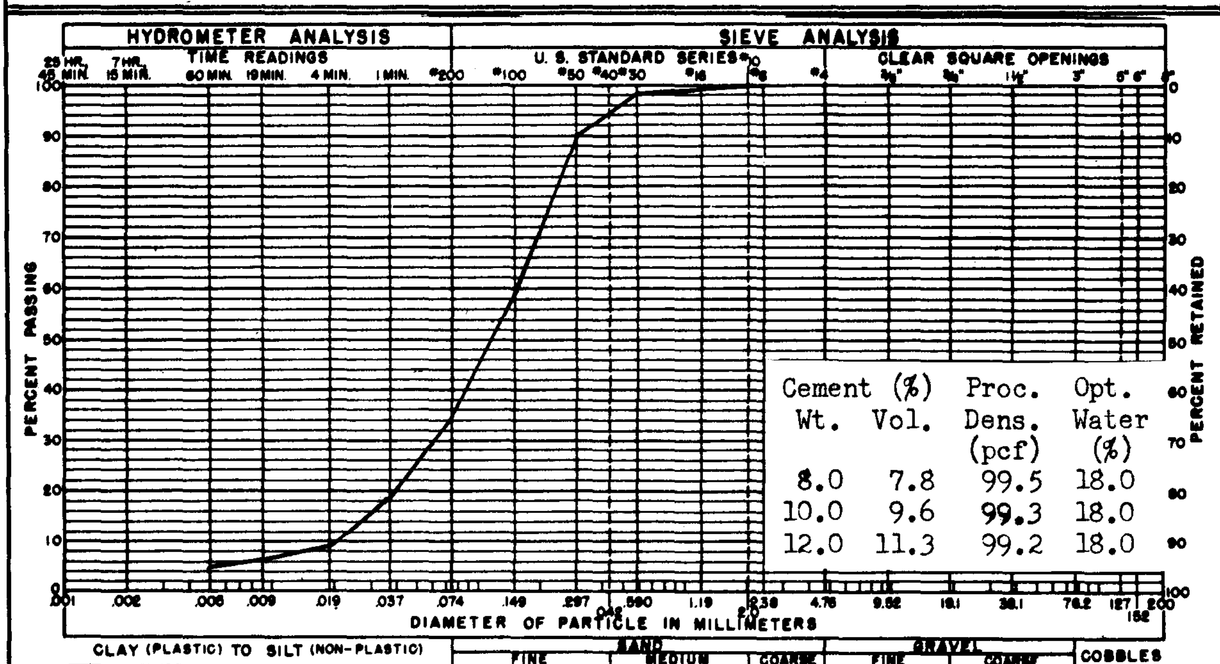
SPECIFIC GRAVITY ______

NOTES: Merritt Dam

ATTERBERG LIMITS
LIQUID LIMIT ______
PLASTICITY INDEX ______
SHRINKAGE LIMIT ______

GRAVEL 0 %
SAND 97 %
SILT TO CLAY 3 %

LABORATORY SAMPLE No. 15R-49 FIELD DESIGNATION ______ EXCAVATION No. TP-A Left Abut. DEPTH 0-31.0 FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
8.0	7.8	99.5	18.0
10.0	9.6	99.3	18.0
12.0	11.3	99.2	18.0

CLASSIFICATION SYMBOL SM

SPECIFIC GRAVITY ______

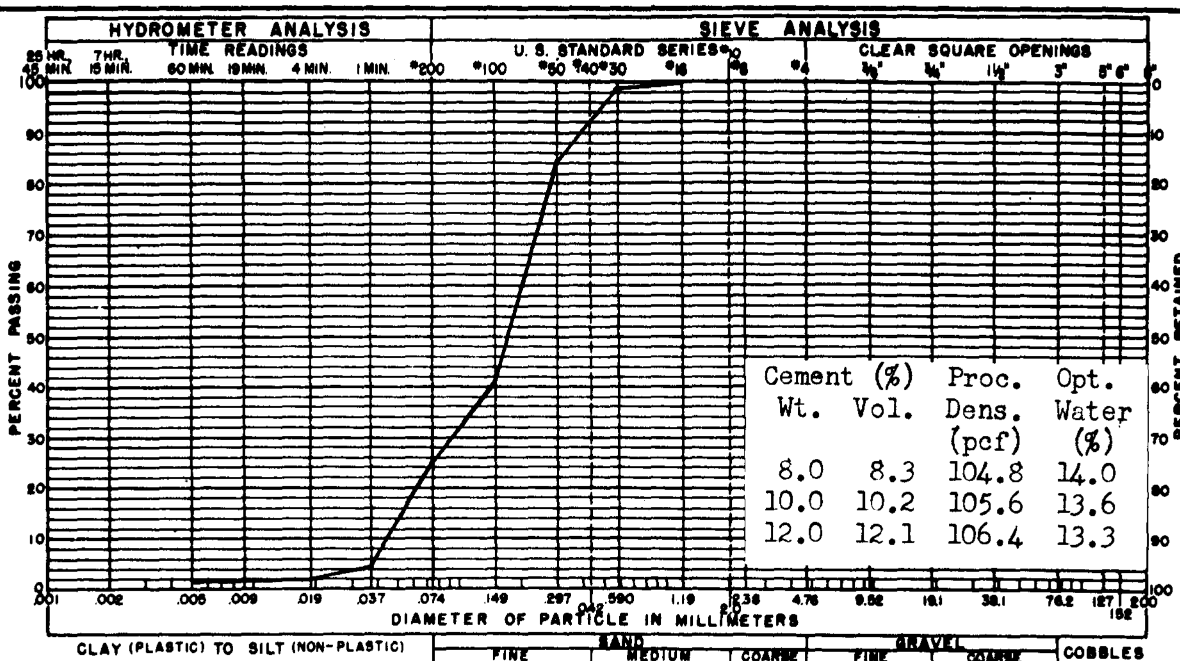
NOTES: Merritt Dam

ATTERBERG LIMITS
LIQUID LIMIT ______
PLASTICITY INDEX ______
SHRINKAGE LIMIT ______

GRAVEL 0 %
SAND 65 %
SILT TO CLAY 35 %

LABORATORY SAMPLE No. 15R-X59 FIELD DESIGNATION ______ EXCAVATION No. AH-301A DEPTH 0-7.0 FT.

INDEX TEST SUMMARY



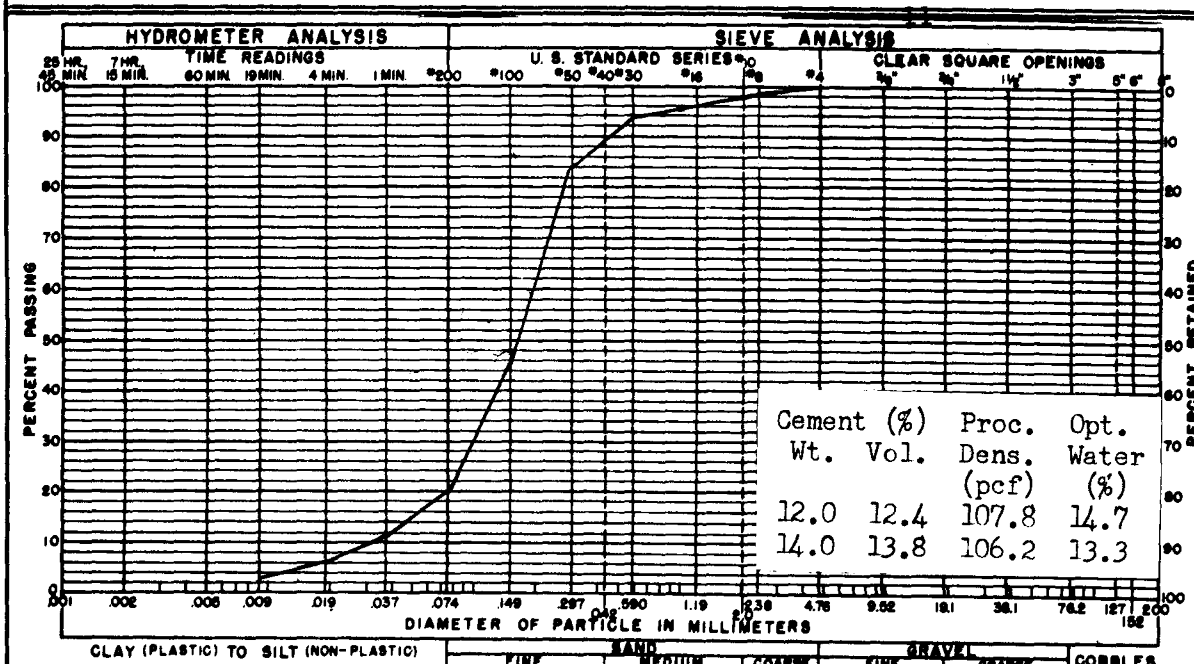
Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
8.0	8.3	104.8	14.0
10.0	10.2	105.6	13.6
12.0	12.1	106.4	13.3

GRAVEL 0%
SAND 75%
SILT TO CLAY 25%

CLASSIFICATION SYMBOL SM
SPECIFIC GRAVITY
NOTES: Merritt Dam

ATTERBERG LIMITS
LIQUID LIMIT
PLASTICITY INDEX
SHRINKAGE LIMIT

LABORATORY SAMPLE No. 15R-X60 FIELD DESIGNATION ______ EXCAVATION No. 768' West from DH-11 DEPTH 1.5-7.8 FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
12.0	12.4	107.8	14.7
14.0	13.8	106.2	13.3

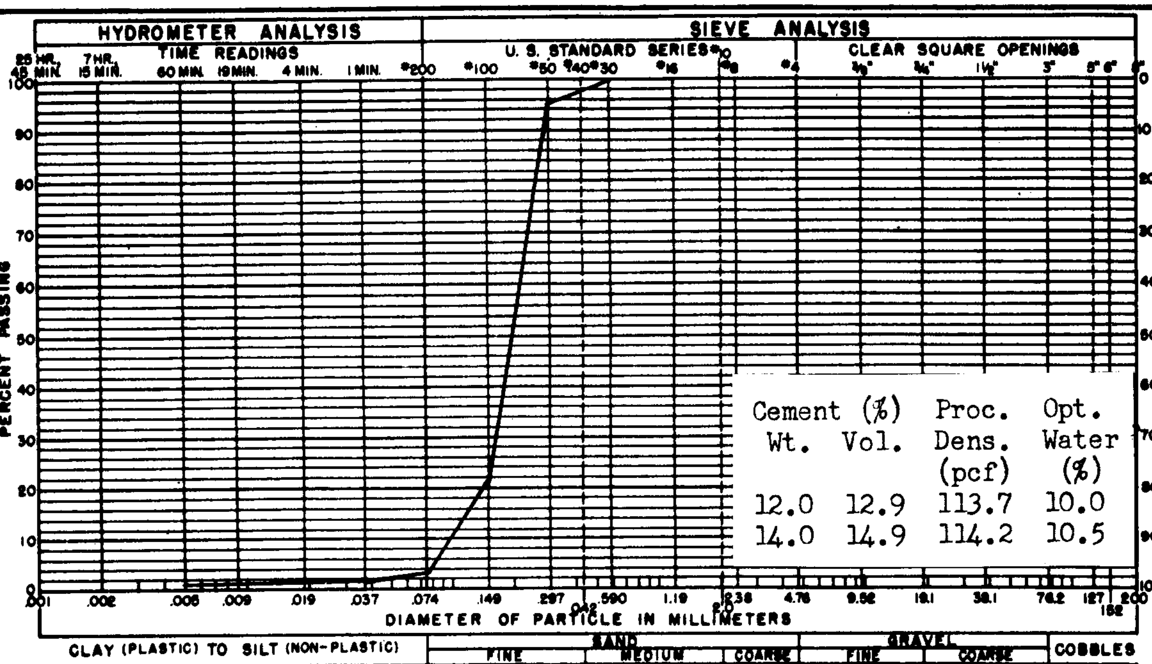
GRAVEL 0%
SAND 81%
SILT TO CLAY 19%

CLASSIFICATION SYMBOL SM
SPECIFIC GRAVITY
NOTES: Merritt Dam

ATTERBERG LIMITS
LIQUID LIMIT
PLASTICITY INDEX
SHRINKAGE LIMIT

LABORATORY SAMPLE No. 15R-110 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.

INDEX TEST SUMMARY



Cement (%) Wt.	Cement (%) Vol.	Proc. Dens. (pcf)	Opt. Water (%)
12.0	12.9	113.7	10.0
14.0	14.9	114.2	10.5

GRAVEL 0 %
SAND 97 %
SILT TO CLAY 3 %

CLASSIFICATION SYMBOL SP

SPECIFIC GRAVITY ______

NOTES Merritt Dam

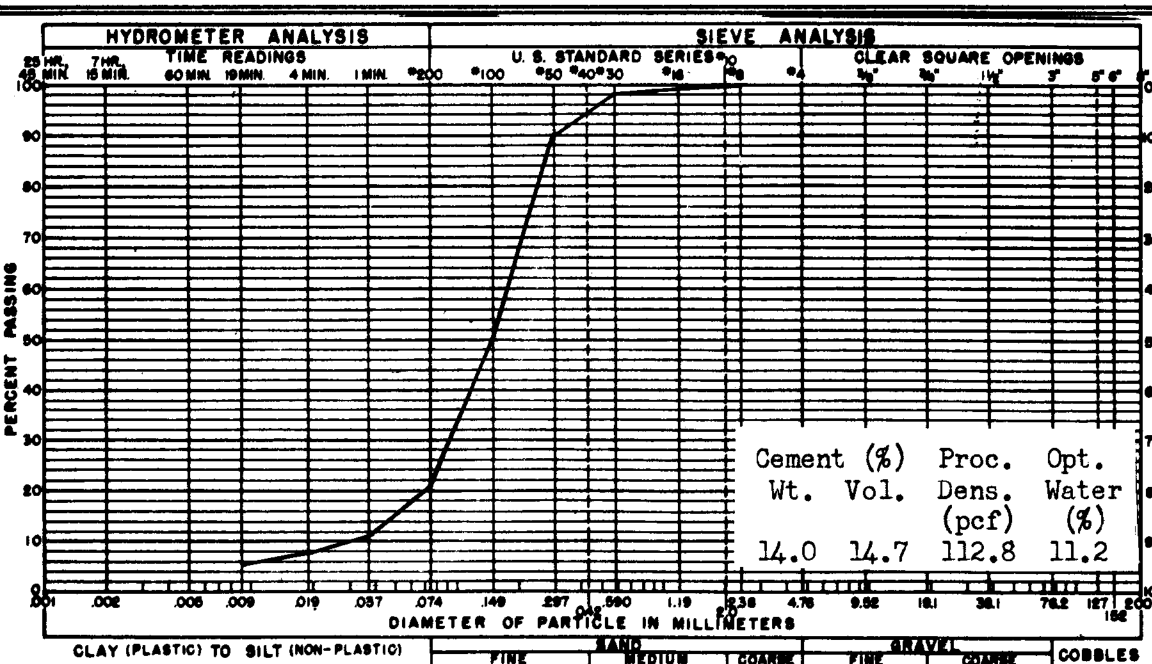
ATTERBERG LIMITS

LIQUID LIMIT ______

PLASTICITY INDEX ______

SHRINKAGE LIMIT ______

LABORATORY SAMPLE No. 15R-111 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.



Cement (%) Wt.	Cement (%) Vol.	Proc. Dens. (pcf)	Opt. Water (%)
14.0	14.7	112.8	11.2

GRAVEL 0 %
SAND 79 %
SILT TO CLAY 21 %

CLASSIFICATION SYMBOL SM

SPECIFIC GRAVITY ______

NOTES: Merritt Dam

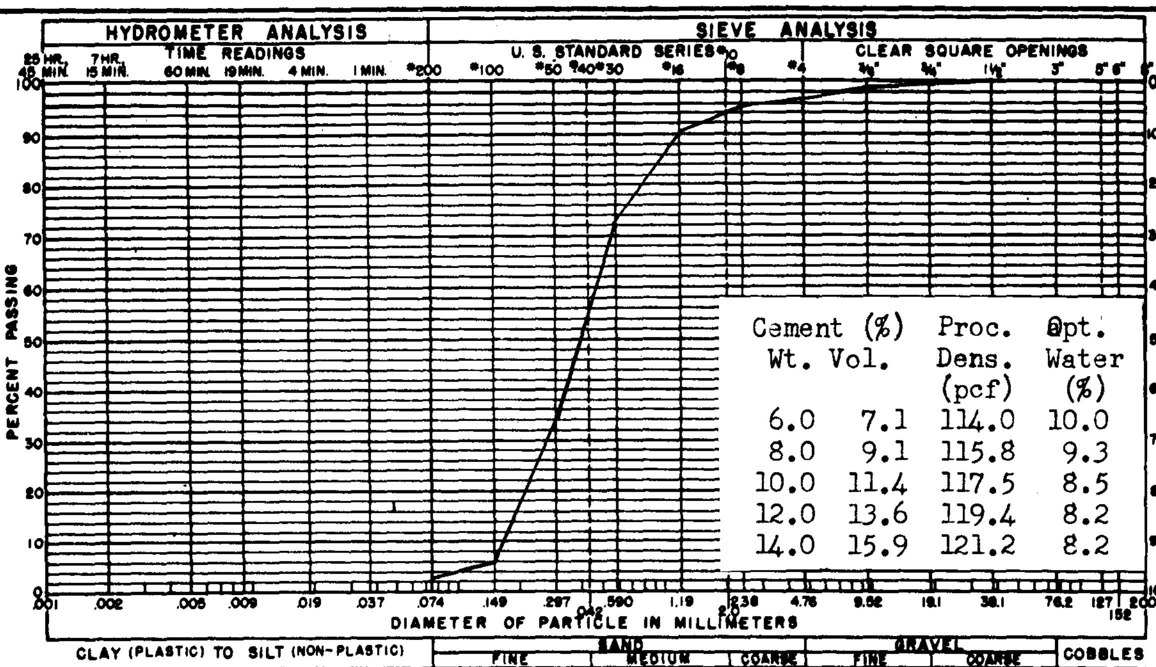
ATTERBERG LIMITS

LIQUID LIMIT ______

PLASTICITY INDEX ______

SHRINKAGE LIMIT ______

LABORATORY SAMPLE No. 15R-118 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
6.0	7.1	114.0	10.0
8.0	9.1	115.8	9.3
10.0	11.4	117.5	8.5
12.0	13.6	119.4	8.2
14.0	15.9	121.2	8.2

CLASSIFICATION SYMBOL SP

SPECIFIC GRAVITY ______

NOTES Cheney Dam

ATTERBERG LIMITS

LIQUID LIMIT ______

PLASTICITY INDEX ______

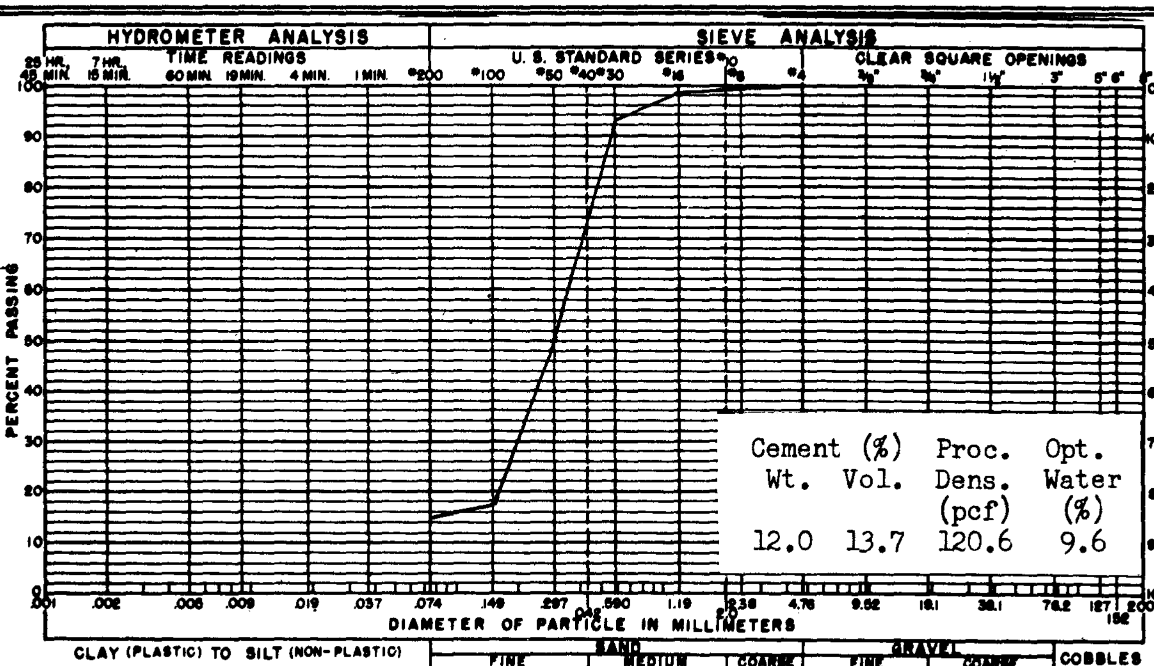
SHRINKAGE LIMIT ______

GRAVEL 3 %

SAND 94 %

SILT TO CLAY 3 %

LABORATORY SAMPLE No. 25J-X108 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
12.0	13.7	120.6	9.6

CLASSIFICATION SYMBOL SM

SPECIFIC GRAVITY ______

NOTES: Cheney Dam

ATTERBERG LIMITS

LIQUID LIMIT ______

PLASTICITY INDEX ______

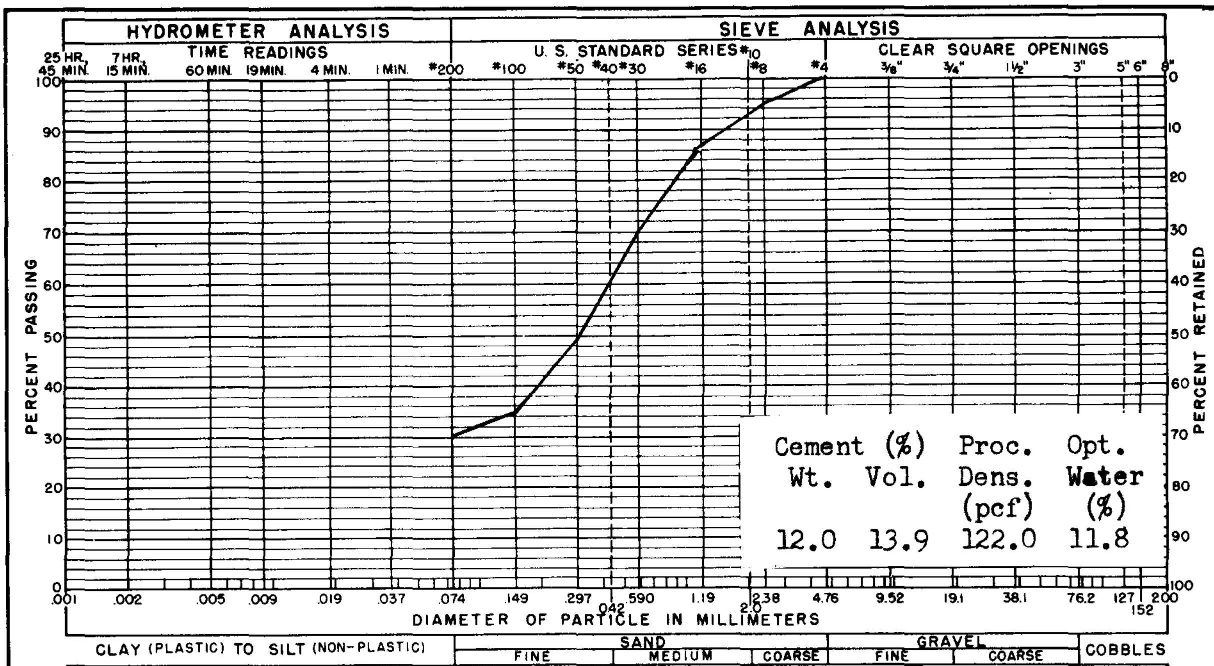
SHRINKAGE LIMIT ______

GRAVEL 0 %

SAND 85 %

SILT TO CLAY 15 %

LABORATORY SAMPLE No. 25J-X147 FIELD DESIGNATION ______ EXCAVATION No. TP-2 DEPTH 13.0-18.0 FT.



Cement (%) Wt.	Cement (%) Vol.	Proc. Dens. (pcf)	Opt. Water (%)
12.0	13.9	122.0	11.8

CLASSIFICATION SYMBOL SM

SPECIFIC GRAVITY ______

NOTES: Cheney Dam

ATTERBERG LIMITS

LIQUID LIMIT ______

PLASTICITY INDEX ______

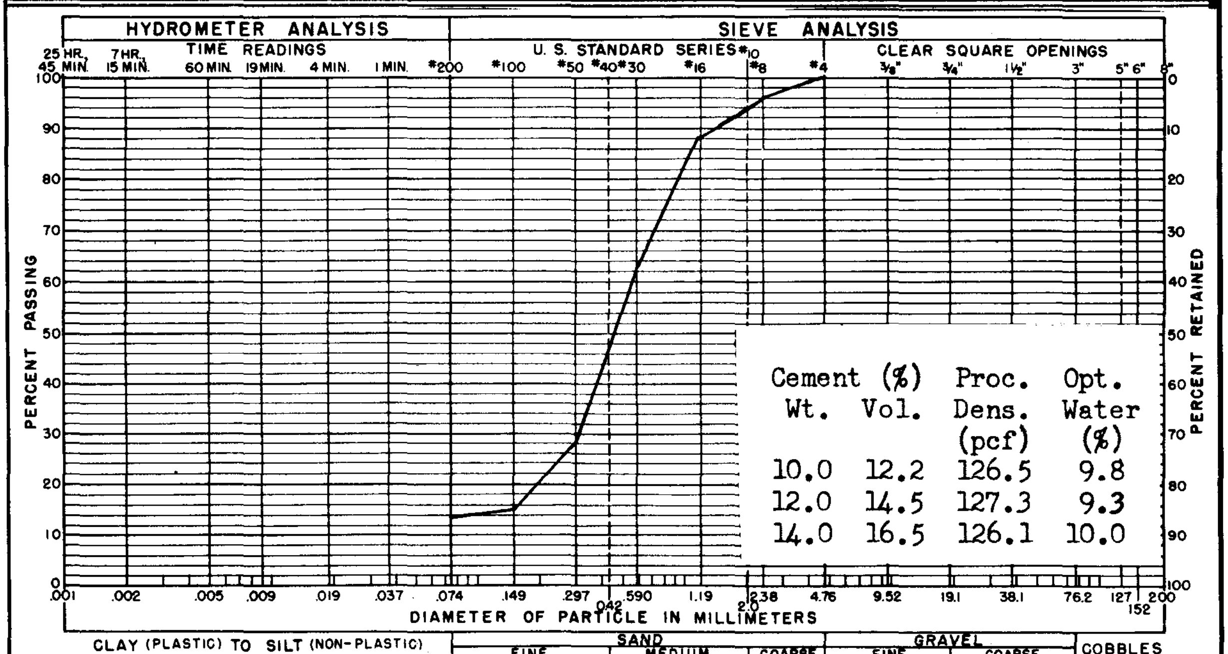
SHRINKAGE LIMIT ______

GRAVEL 0 %

SAND 70 %

SILT TO CLAY 30 %

LABORATORY SAMPLE No. 25J-149 FIELD DESIGNATION ______ EXCAVATION No. AH-265 DEPTH 0-8.0 FT.



Cement (%) Wt.	Cement (%) Vol.	Proc. Dens. (pcf)	Opt. Water (%)
10.0	12.2	126.5	9.8
12.0	14.5	127.3	9.3
14.0	16.5	126.1	10.0

CLASSIFICATION SYMBOL SM

SPECIFIC GRAVITY ______

NOTES: Cheney Dam

ATTERBERG LIMITS

LIQUID LIMIT ______

PLASTICITY INDEX ______

SHRINKAGE LIMIT ______

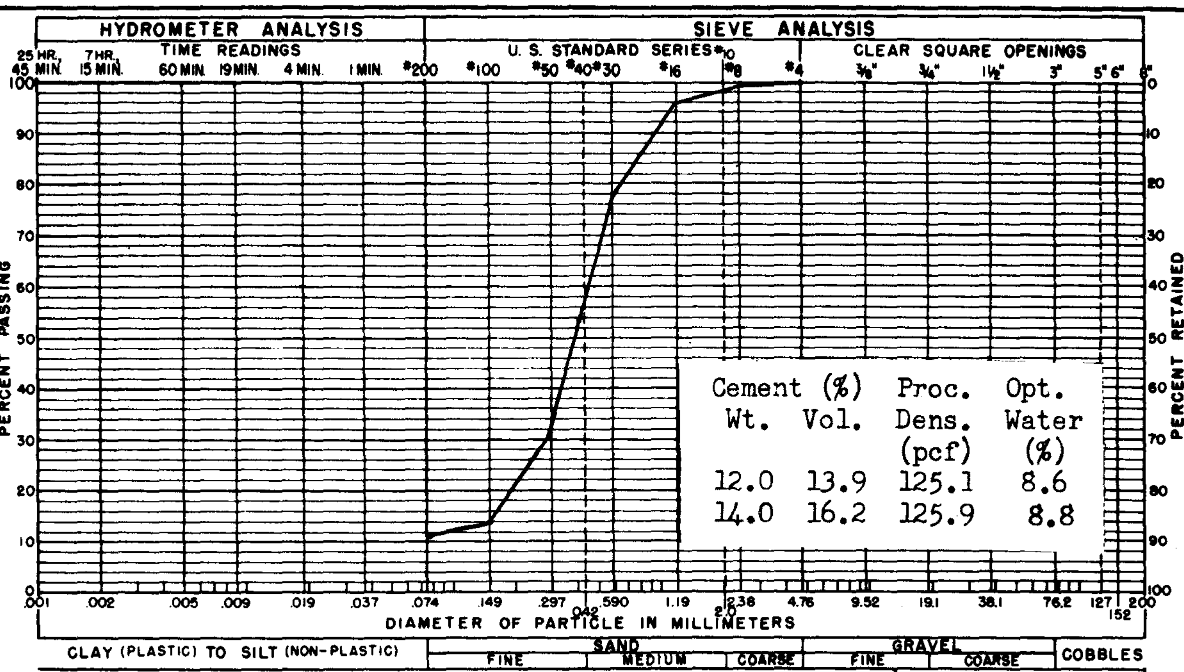
GRAVEL 0 %

SAND 86 %

SILT TO CLAY 14 %

LABORATORY SAMPLE No. 25J-X150 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.

INDEX TEST SUMMARY



Cement (%) Wt.	Cement (%) Vol.	Proc. Dens. (pcf)	Opt. Water (%)
12.0	13.9	125.1	8.6
14.0	16.2	125.9	8.8

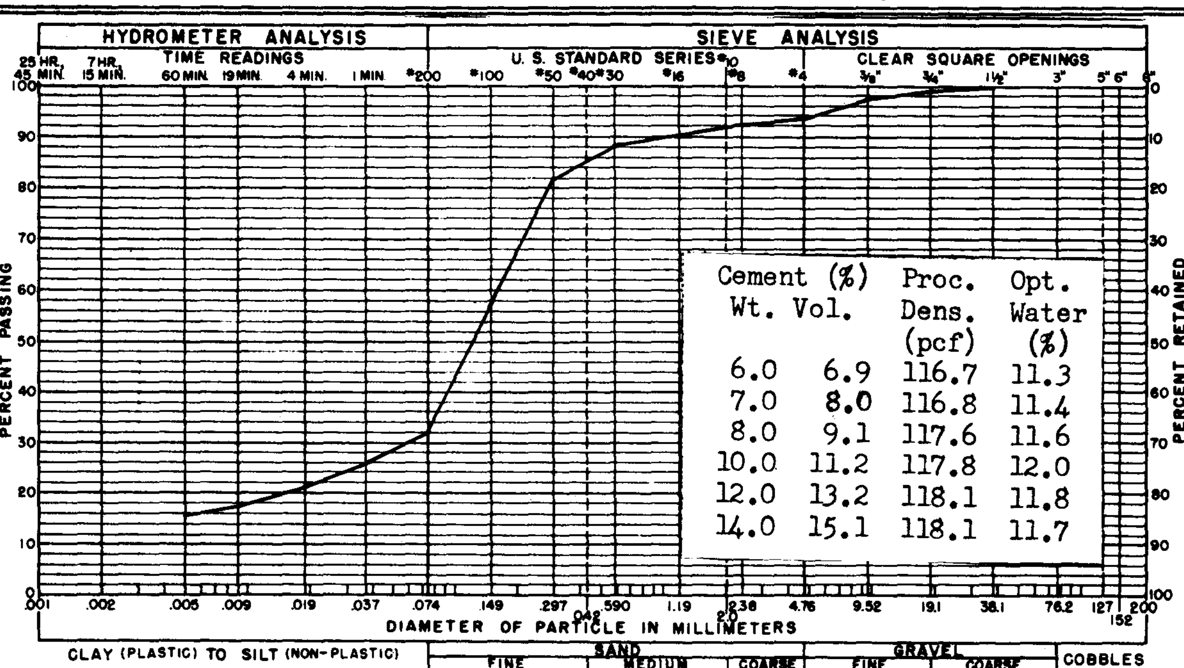
GRAVEL 0 %
SAND 91 %
SILT TO CLAY 9 %

CLASSIFICATION SYMBOL SM-SP
SPECIFIC GRAVITY ______
NOTES: Cheney Dam

ATTERBERG LIMITS
LIQUID LIMIT ______
PLASTICITY INDEX ______
SHRINKAGE LIMIT ______

N384,500

LABORATORY SAMPLE No. 25J-156 FIELD DESIGNATION ______ EXCAVATION No. E2198, 250 DEPTH 1.5-8.4 FT.



Cement (%) Wt.	Cement (%) Vol.	Proc. Dens. (pcf)	Opt. Water (%)
6.0	6.9	116.7	11.3
7.0	8.0	116.8	11.4
8.0	9.1	117.6	11.6
10.0	11.2	117.8	12.0
12.0	13.2	118.1	11.8
14.0	15.1	118.1	11.7

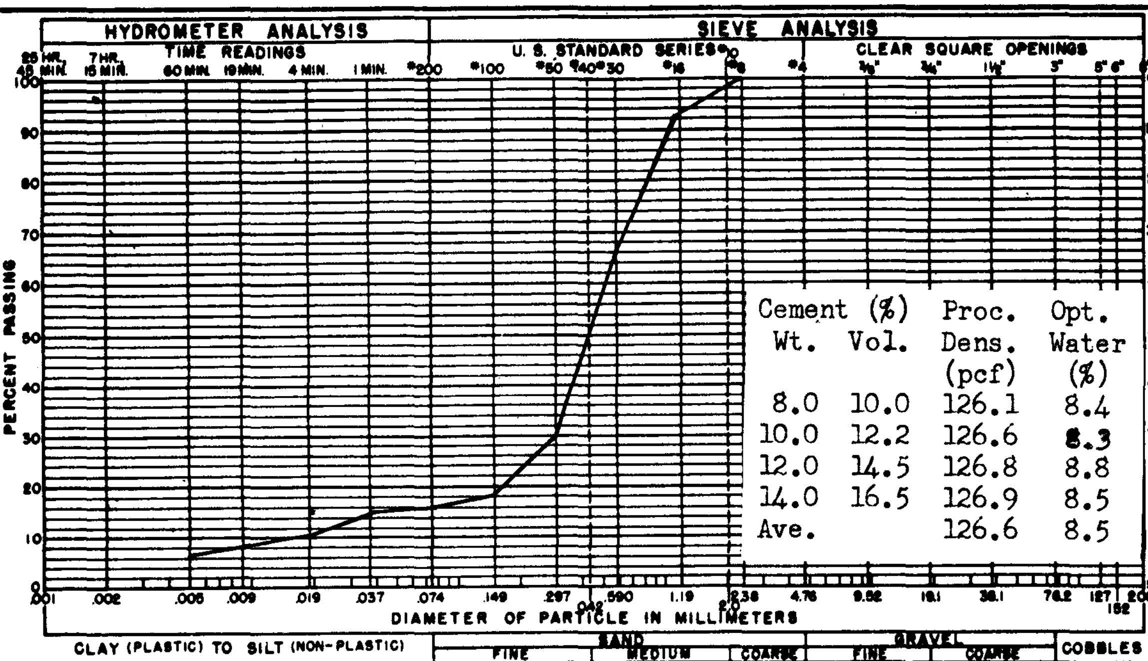
GRAVEL 7 %
SAND 61 %
SILT TO CLAY 32 %

CLASSIFICATION SYMBOL SM-SC
SPECIFIC GRAVITY ______
NOTES: Lubbock Reg. Res.

ATTERBERG LIMITS
LIQUID LIMIT 22
PLASTICITY INDEX 5
SHRINKAGE LIMIT ______

Mixture of TP-3,4,6, and 11

LABORATORY SAMPLE No. 39U-X16 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
8.0	10.0	126.1	8.4
10.0	12.2	126.6	8.3
12.0	14.5	126.8	8.8
14.0	16.5	126.9	8.5
Ave.		126.6	8.5

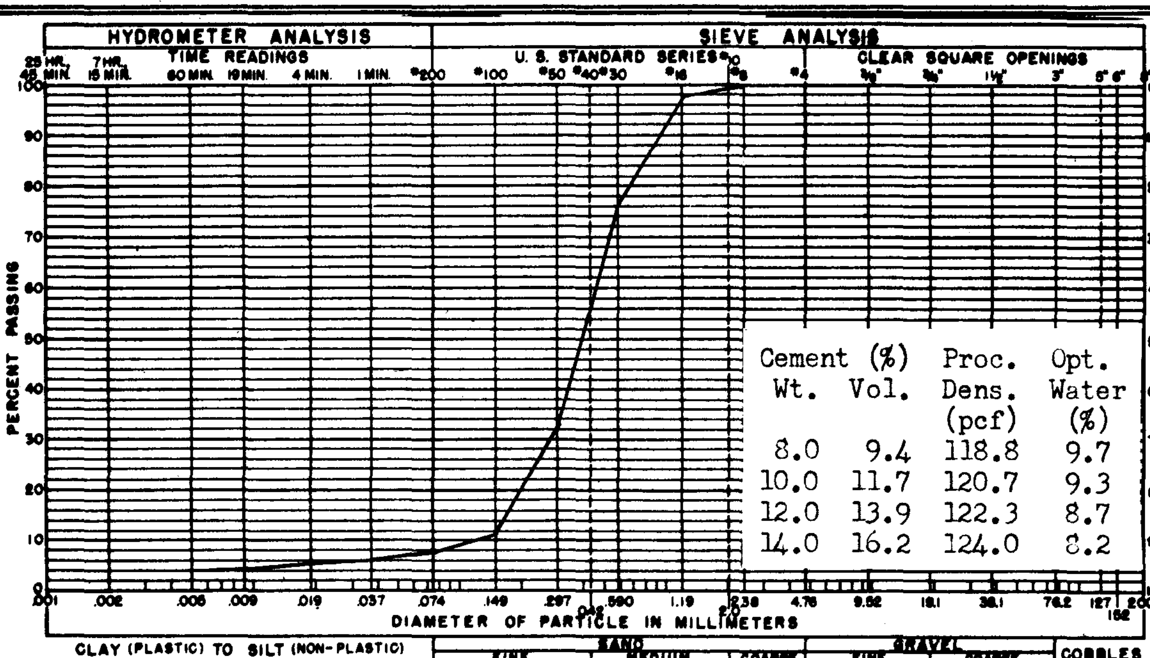
GRAVEL 0 %
SAND 84 %
SILT TO CLAY 16 %

CLASSIFICATION SYMBOL SM
SPECIFIC GRAVITY ____
NOTES Glen Elder Dam

ATTERBERG LIMITS
LIQUID LIMIT ____
PLASTICITY INDEX ____
SHRINKAGE LIMIT ____

Mixture of AH-1 through AH-6

LABORATORY SAMPLE No. 18C-X223 FIELD DESIGNATION ____ EXCAVATION No. ____ DEPTH 0-4.0 FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
8.0	9.4	118.8	9.7
10.0	11.7	120.7	9.3
12.0	13.9	122.3	8.7
14.0	16.2	124.0	8.2

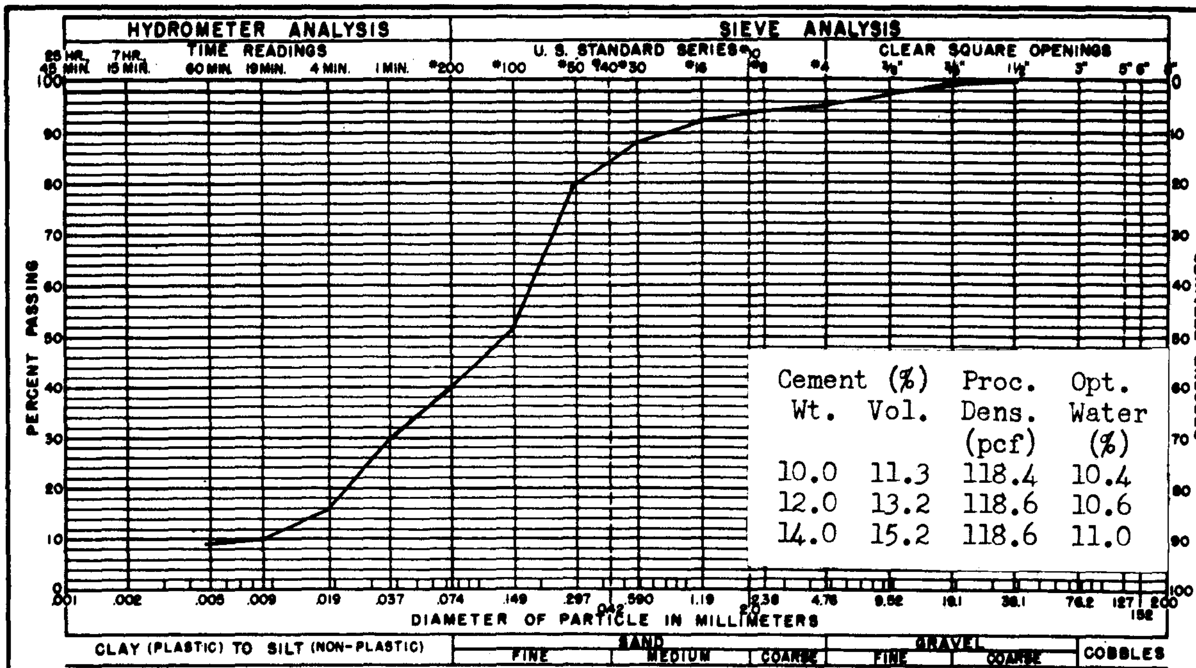
GRAVEL 0 %
SAND 92 %
SILT TO CLAY 8 %

CLASSIFICATION SYMBOL SP-SM
SPECIFIC GRAVITY ____
NOTES: Glen Elder Dam

ATTERBERG LIMITS
LIQUID LIMIT ____
PLASTICITY INDEX ____
SHRINKAGE LIMIT ____

Mixture of AH P-1 through AH P-6

LABORATORY SAMPLE No. 18C-X224 FIELD DESIGNATION ____ EXCAVATION No. ____ DEPTH 0-4.0 FT.



Cement (%) Wt.	Cement (%) Vol.	Proc. Dens. (pcf)	Opt. Water (%)
10.0	11.3	118.4	10.4
12.0	13.2	118.6	10.6
14.0	15.2	118.6	11.0

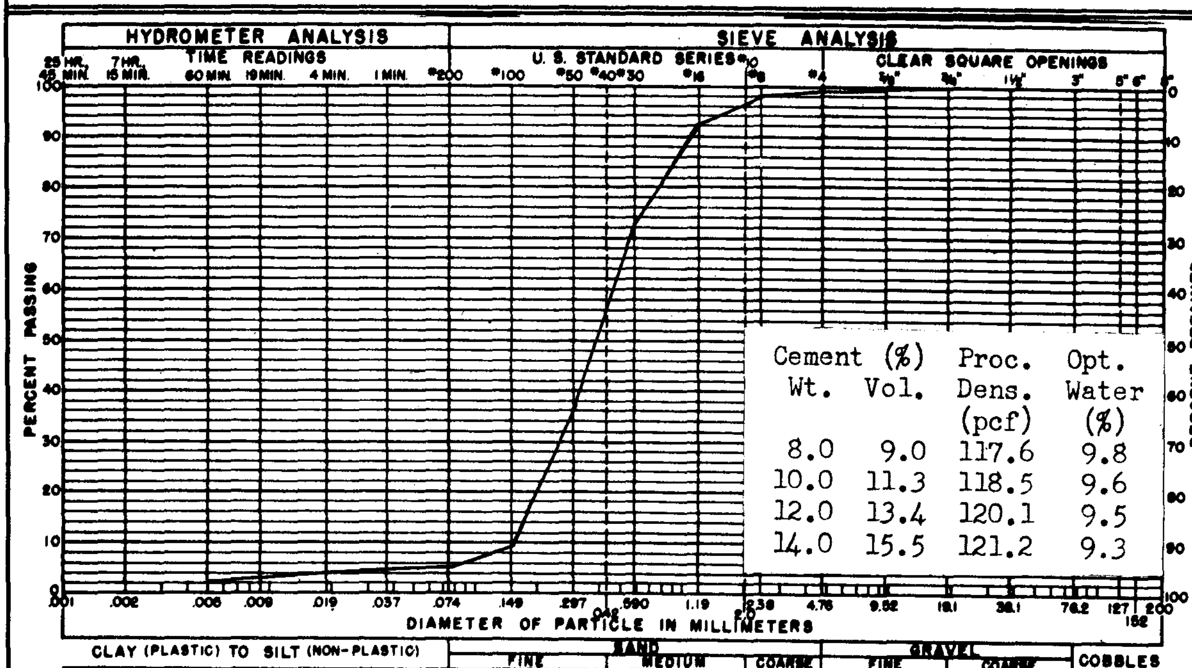
GRAVEL 5 %
SAND 55 %
SILT TO CLAY 40 %

CLASSIFICATION SYMBOL SM
SPECIFIC GRAVITY ______
NOTES: Downs Dike

ATTERBERG LIMITS
LIQUID LIMIT ______
PLASTICITY INDEX ______
SHRINKAGE LIMIT ______

Barnes Pit Overburden

LABORATORY SAMPLE No. 40Y-41 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH 0-5.0 FT.



Cement (%) Wt.	Cement (%) Vol.	Proc. Dens. (pcf)	Opt. Water (%)
8.0	9.0	117.6	9.8
10.0	11.3	118.5	9.6
12.0	13.4	120.1	9.5
14.0	15.5	121.2	9.3

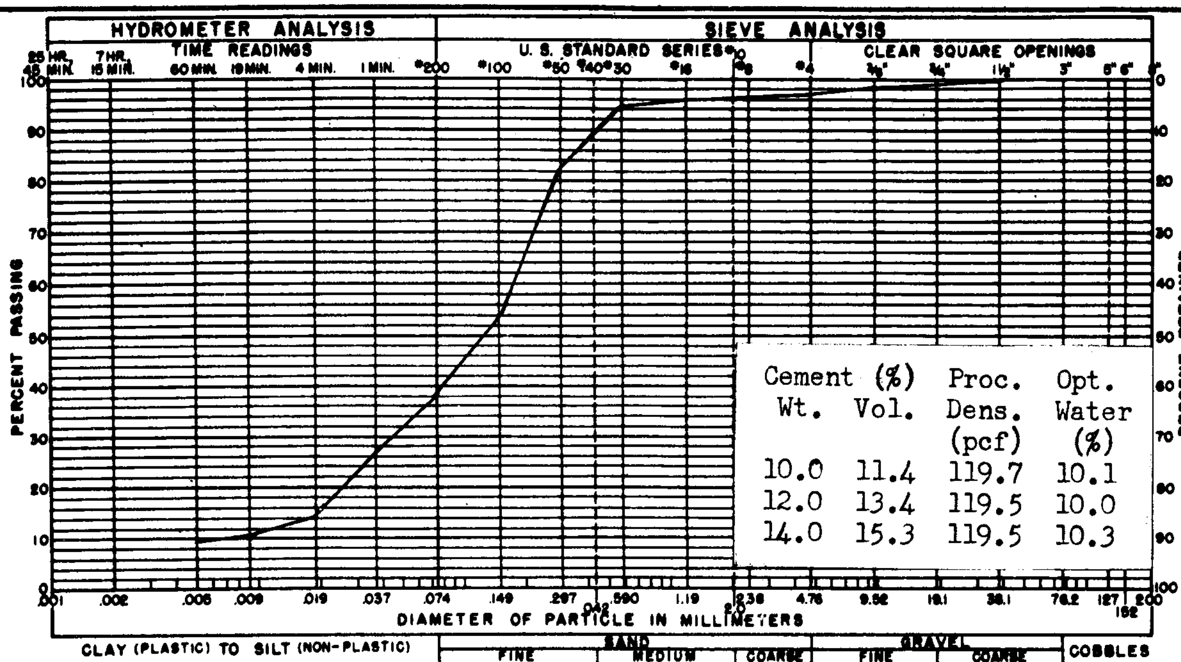
GRAVEL 1 %
SAND 94 %
SILT TO CLAY 5 %

CLASSIFICATION SYMBOL SP-SM
SPECIFIC GRAVITY ______
NOTES: Downs Dike

ATTERBERG LIMITS
LIQUID LIMIT ______
PLASTICITY INDEX ______
SHRINKAGE LIMIT ______

Barnes Pit Stockpile

LABORATORY SAMPLE No. 40Y-42 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH 0-30.0 FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
10.0	11.4	119.7	10.1
12.0	13.4	119.5	10.0
14.0	15.3	119.5	10.3

CLASSIFICATION SYMBOL SM

SPECIFIC GRAVITY ______

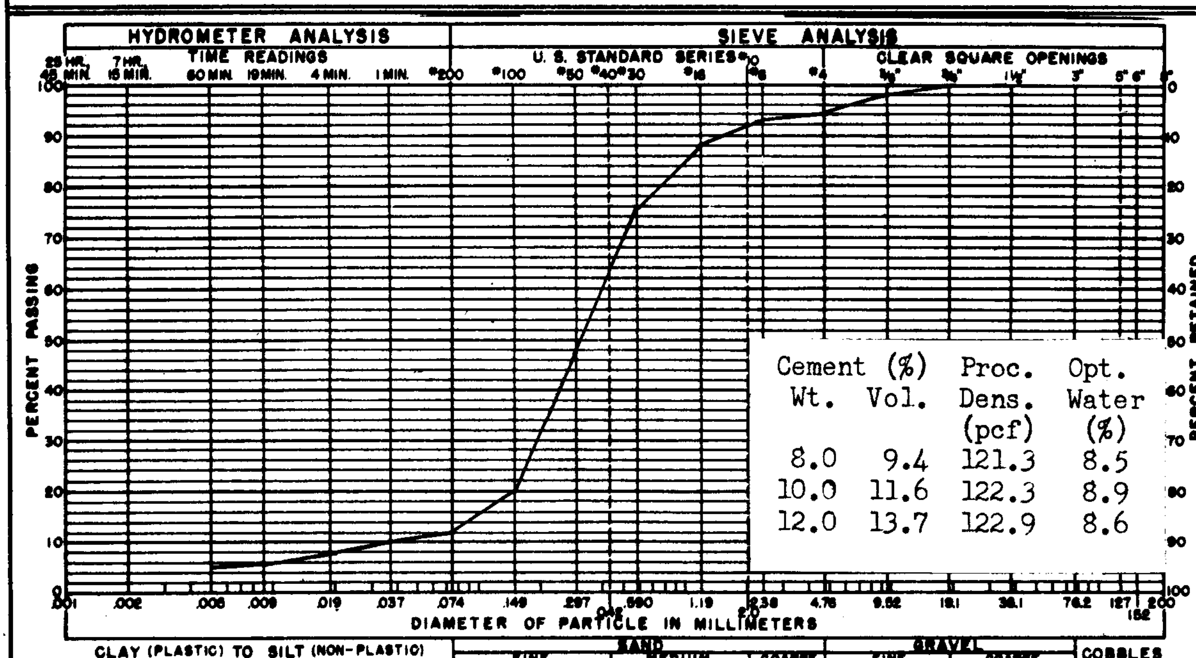
NOTES: Cawker City Dike

ATTERBERG LIMITS
LIQUID LIMIT ______
PLASTICITY INDEX ______
SHRINKAGE LIMIT ______

GRAVEL 3 %
SAND 58 %
SILT TO CLAY 39 %

Shearer Pit Overburden

LABORATORY SAMPLE No. 40M-134 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH 0-5.0 FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
8.0	9.4	121.3	8.5
10.0	11.6	122.3	8.9
12.0	13.7	122.9	8.6

CLASSIFICATION SYMBOL SW-SM

SPECIFIC GRAVITY ______

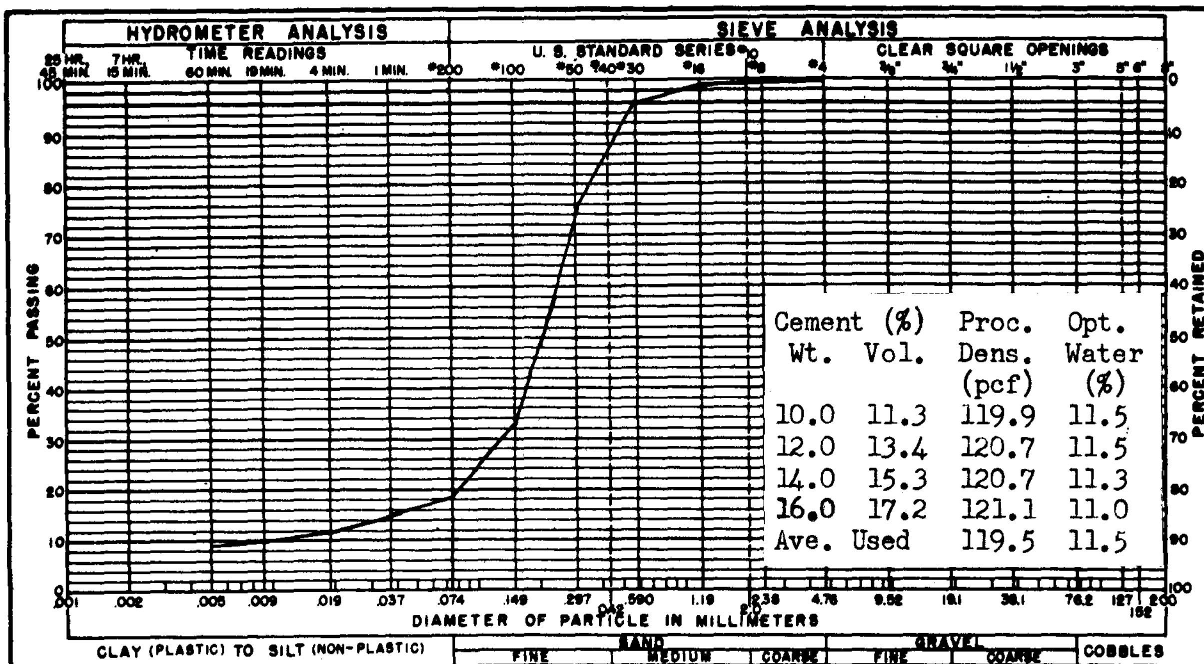
NOTES: Cawker City Dike

ATTERBERG LIMITS
LIQUID LIMIT ______
PLASTICITY INDEX ______
SHRINKAGE LIMIT ______

GRAVEL 5 %
SAND 83 %
SILT TO CLAY 12 %

Shearer Pit

LABORATORY SAMPLE No. 40M-135 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH 5.0-20.0 FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
10.0	11.3	119.9	11.5
12.0	13.4	120.7	11.5
14.0	15.3	120.7	11.3
16.0	17.2	121.1	11.0
Ave. Used		119.5	11.5

GRAVEL 0 %
SAND 81 %
SILT TO CLAY 19 %

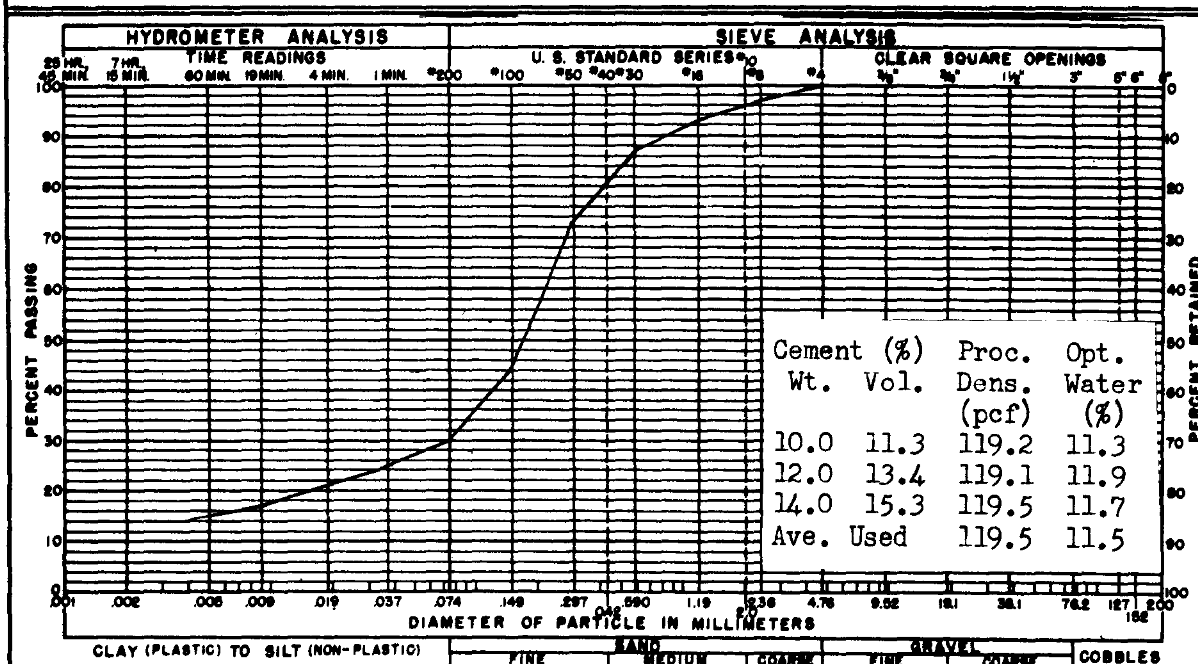
CLASSIFICATION SYMBOL SM

SPECIFIC GRAVITY

NOTES: Starvation Dam

ATTERBERG LIMITS
LIQUID LIMIT
PLASTICITY INDEX
SHRINKAGE LIMIT

LABORATORY SAMPLE No. 37Q-X69 FIELD DESIGNATION EXCAVATION No. DEPTH FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
10.0	11.3	119.2	11.3
12.0	13.4	119.1	11.9
14.0	15.3	119.5	11.7
Ave. Used		119.5	11.5

GRAVEL 0 %
SAND 70 %
SILT TO CLAY 30 %

CLASSIFICATION SYMBOL SM

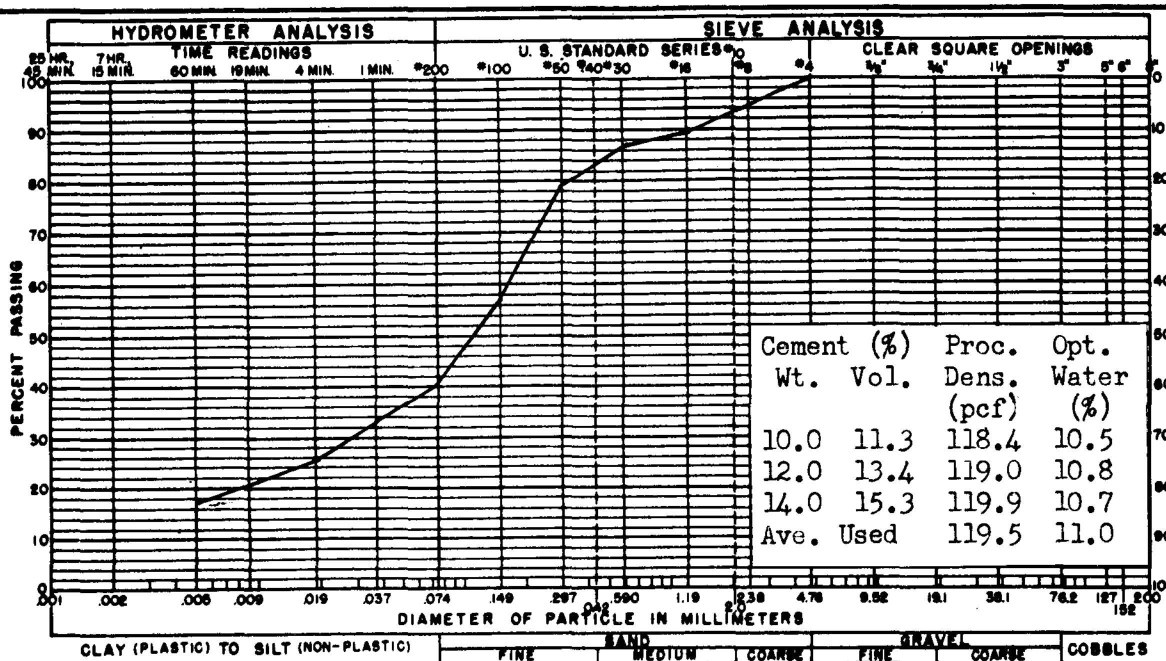
SPECIFIC GRAVITY

NOTES: Starvation Dam

ATTERBERG LIMITS
LIQUID LIMIT
PLASTICITY INDEX
SHRINKAGE LIMIT

LABORATORY SAMPLE No. 37Q-X70 FIELD DESIGNATION EXCAVATION No. DEPTH FT.

INDEX TEST SUMMARY



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
10.0	11.3	118.4	10.5
12.0	13.4	119.0	10.8
14.0	15.3	119.9	10.7
Ave.	Used	119.5	11.0

CLASSIFICATION SYMBOL SM

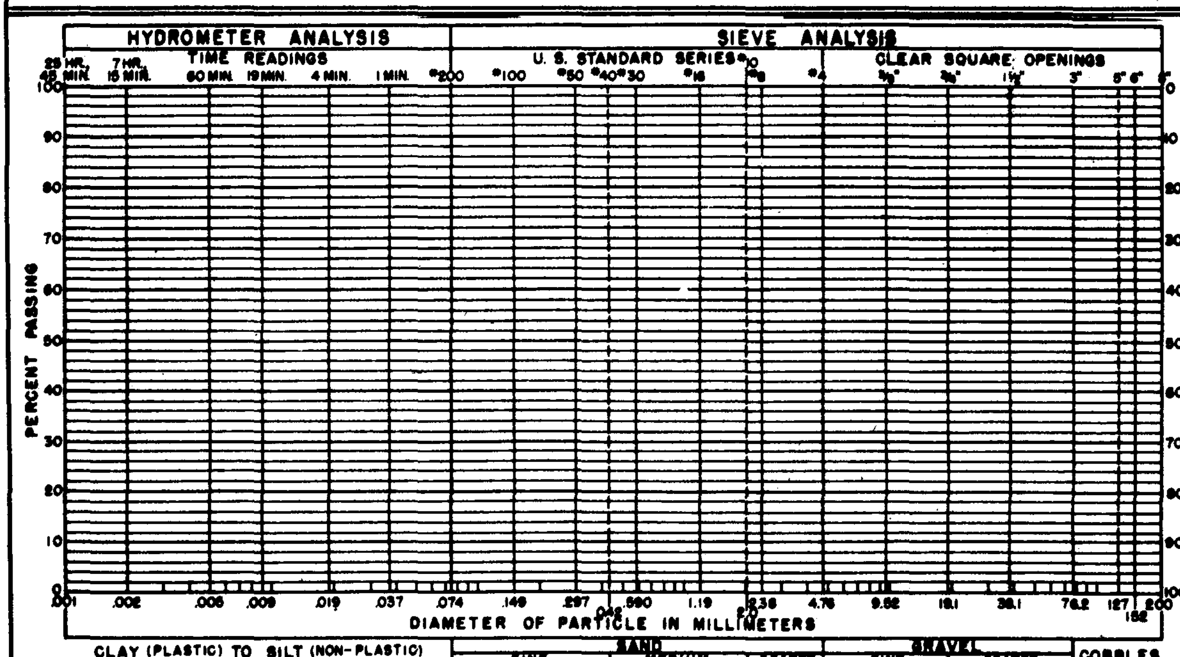
SPECIFIC GRAVITY ______

NOTES: Starvation Dam

ATTERBERG LIMITS
- LIQUID LIMIT ______
- PLASTICITY INDEX ______
- SHRINKAGE LIMIT ______

GRAVEL .... 0 %
SAND .... 60 %
SILT TO CLAY ...... 40 %

LABORATORY SAMPLE No. 370-X71 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.



CLASSIFICATION SYMBOL ______

SPECIFIC GRAVITY ______

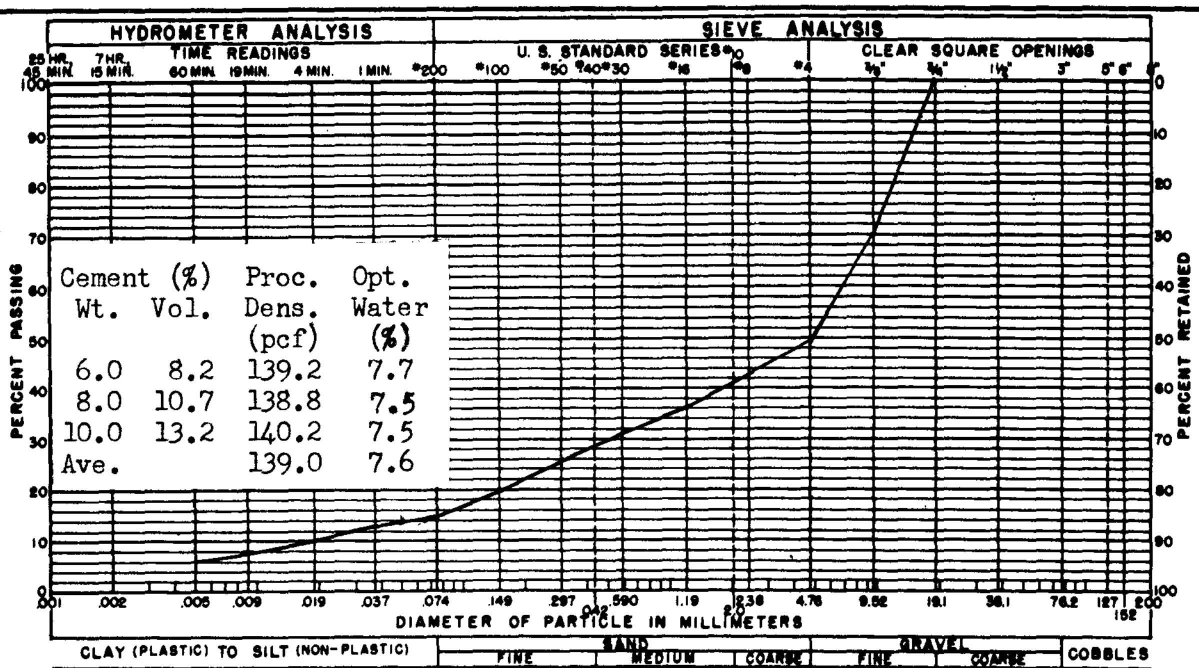
NOTES: ______

ATTERBERG LIMITS
- LIQUID LIMIT ______
- PLASTICITY INDEX ______
- SHRINKAGE LIMIT ______

GRAVEL .... ___ %
SAND .... ___ %
SILT TO CLAY ...... ___ %

LABORATORY SAMPLE No. ______ FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.

INDEX TEST SUMMARY



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
6.0	8.2	139.2	7.7
8.0	10.7	138.8	7.5
10.0	13.2	140.2	7.5
Ave.		139.0	7.6

CLASSIFICATION SYMBOL GM

SPECIFIC GRAVITY ______

ATTERBERG LIMITS

LIQUID LIMIT ______

PLASTICITY INDEX ______

SHRINKAGE LIMIT ______

GRAVEL 50 %

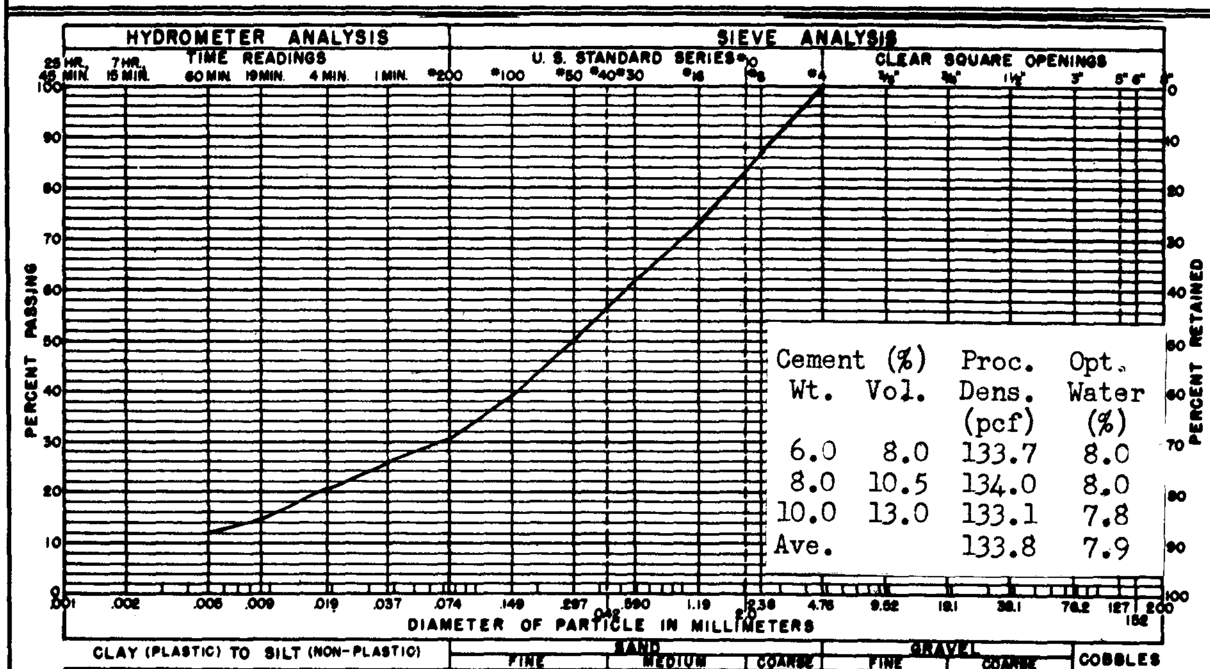
SAND 35 %

SILT TO CLAY 15 %

NOTES: Little Panoche Creek Det. Dam

Composite of APPY-360 0-6', -361 0-6', -362 2-8'

LABORATORY SAMPLE No. 41P-X140 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.



Cement (%) Wt.	Vol.	Proc. Dens. (pcf)	Opt. Water (%)
6.0	8.0	133.7	8.0
8.0	10.5	134.0	8.0
10.0	13.0	133.1	7.8
Ave.		133.8	7.9

CLASSIFICATION SYMBOL SM

SPECIFIC GRAVITY ______

ATTERBERG LIMITS

LIQUID LIMIT ______

PLASTICITY INDEX ______

SHRINKAGE LIMIT ______

GRAVEL 0 %

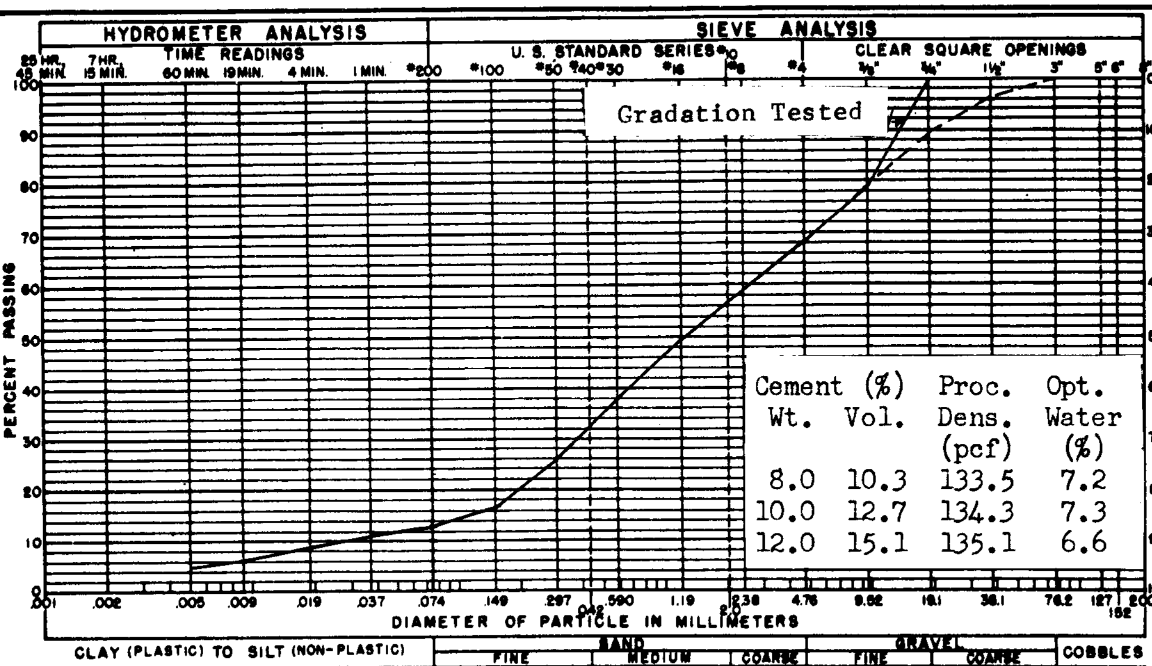
SAND 70 %

SILT TO CLAY 30 %

NOTES: Little Panoche Creek Det. Dam

#4 fraction of 41P-X140

LABORATORY SAMPLE No. 41P-X141 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.



CLASSIFICATION SYMBOL GM

SPECIFIC GRAVITY ______

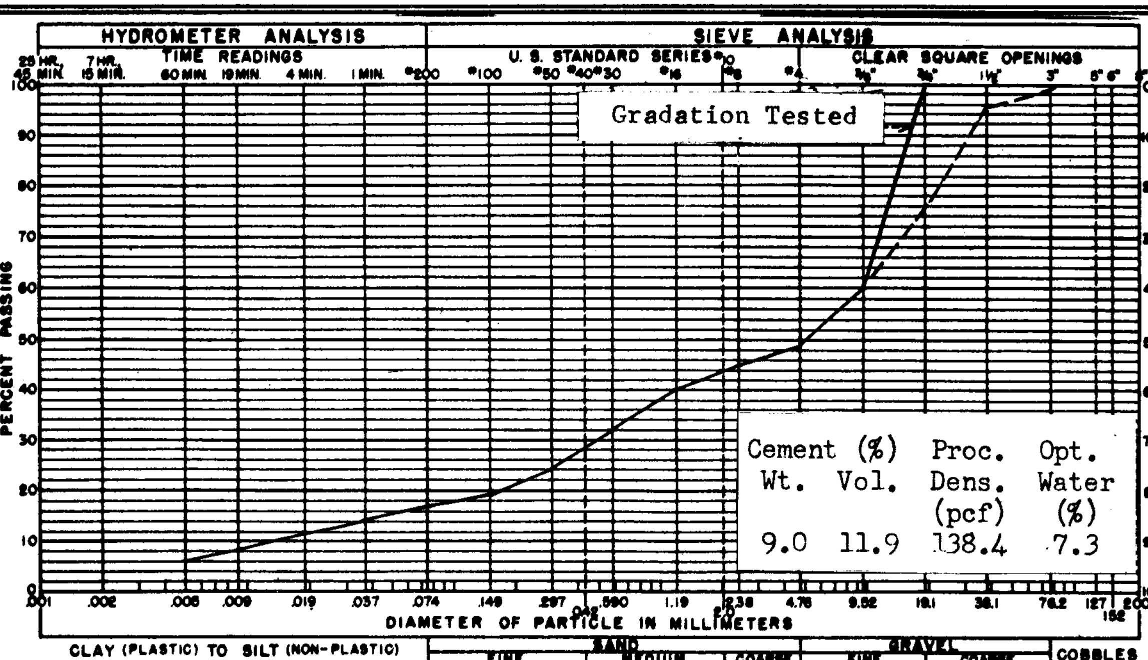
NOTES: Red Bluff Res.

ATTERBERG LIMITS
LIQUID LIMIT ______
PLASTICITY INDEX ______
SHRINKAGE LIMIT ______

GRAVEL 32 %
SAND 54 %
SILT TO CLAY 14 %

Composite of TP-101, -102, -103, 106, -107, -108

LABORATORY SAMPLE No. 40U-X22 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.



CLASSIFICATION SYMBOL GC

SPECIFIC GRAVITY ______

NOTES: Red Bluff Res.

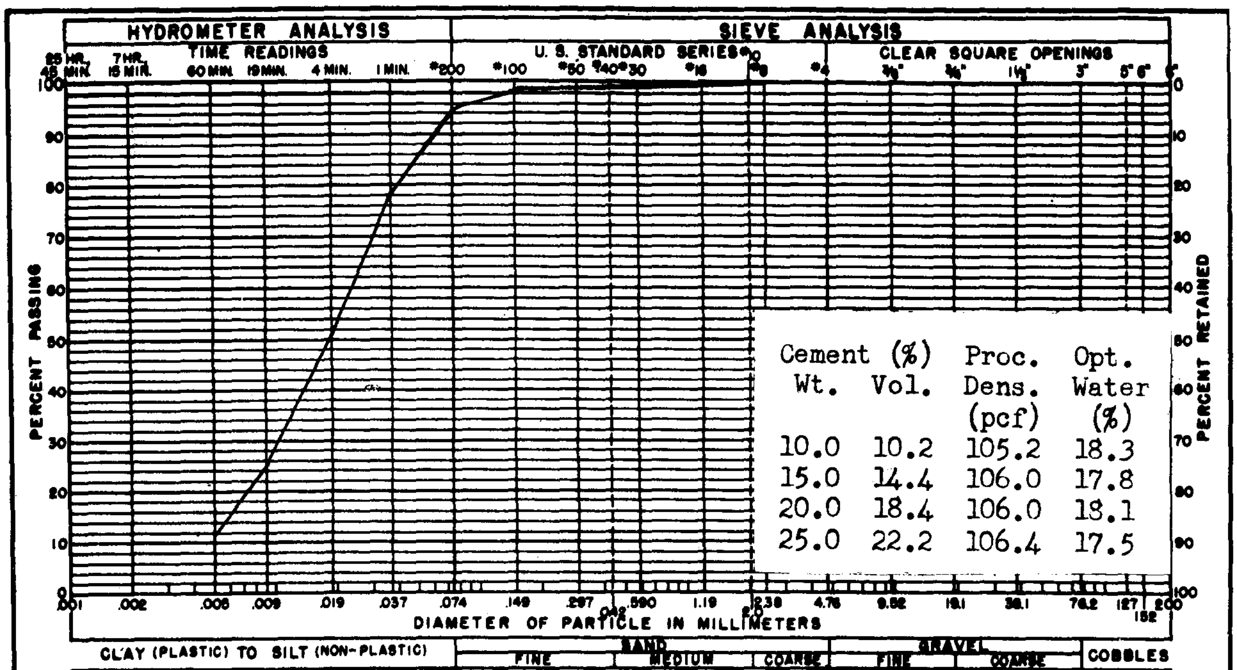
ATTERBERG LIMITS
LIQUID LIMIT 23
PLASTICITY INDEX 8
SHRINKAGE LIMIT ______

GRAVEL 51 %
SAND 32 %
SILT TO CLAY 17 %

80% Tp-506 and 509 20% from Player's Plant

LABORATORY SAMPLE No. 40U-38 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.

INDEX TEST SUMMARY



Cement (%) Wt.	Cement (%) Vol.	Proc. Dens. (pcf)	Opt. Water (%)
10.0	10.2	105.2	18.3
15.0	14.4	106.0	17.8
20.0	18.4	106.0	18.1
25.0	22.2	106.4	17.5

CLASSIFICATION SYMBOL ML

SPECIFIC GRAVITY ______

NOTES: Conconully Dam

ATTERBERG LIMITS

LIQUID LIMIT 25

PLASTICITY INDEX 2

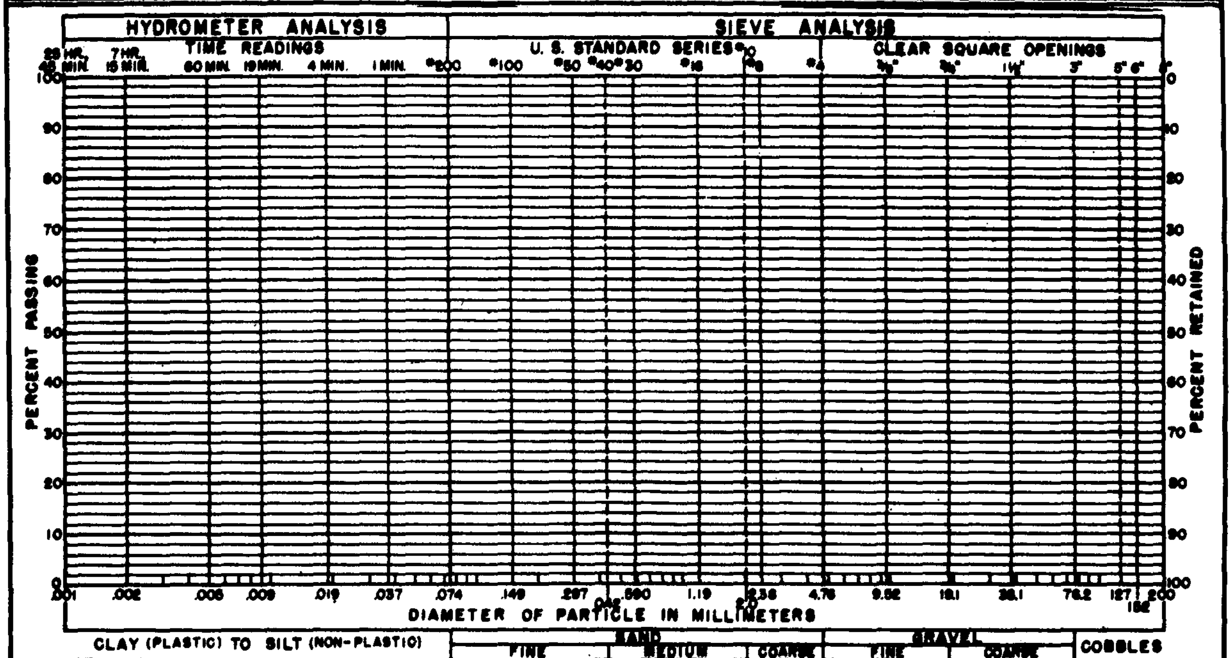
SHRINKAGE LIMIT ______

GRAVEL .... 0 %

SAND .... 5 %

SILT TO CLAY ...... 95 %

LABORATORY SAMPLE No. 47P-1 FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.



CLASSIFICATION SYMBOL ______

SPECIFIC GRAVITY ______

NOTES: ______

ATTERBERG LIMITS

LIQUID LIMIT ______

PLASTICITY INDEX ______

SHRINKAGE LIMIT ______

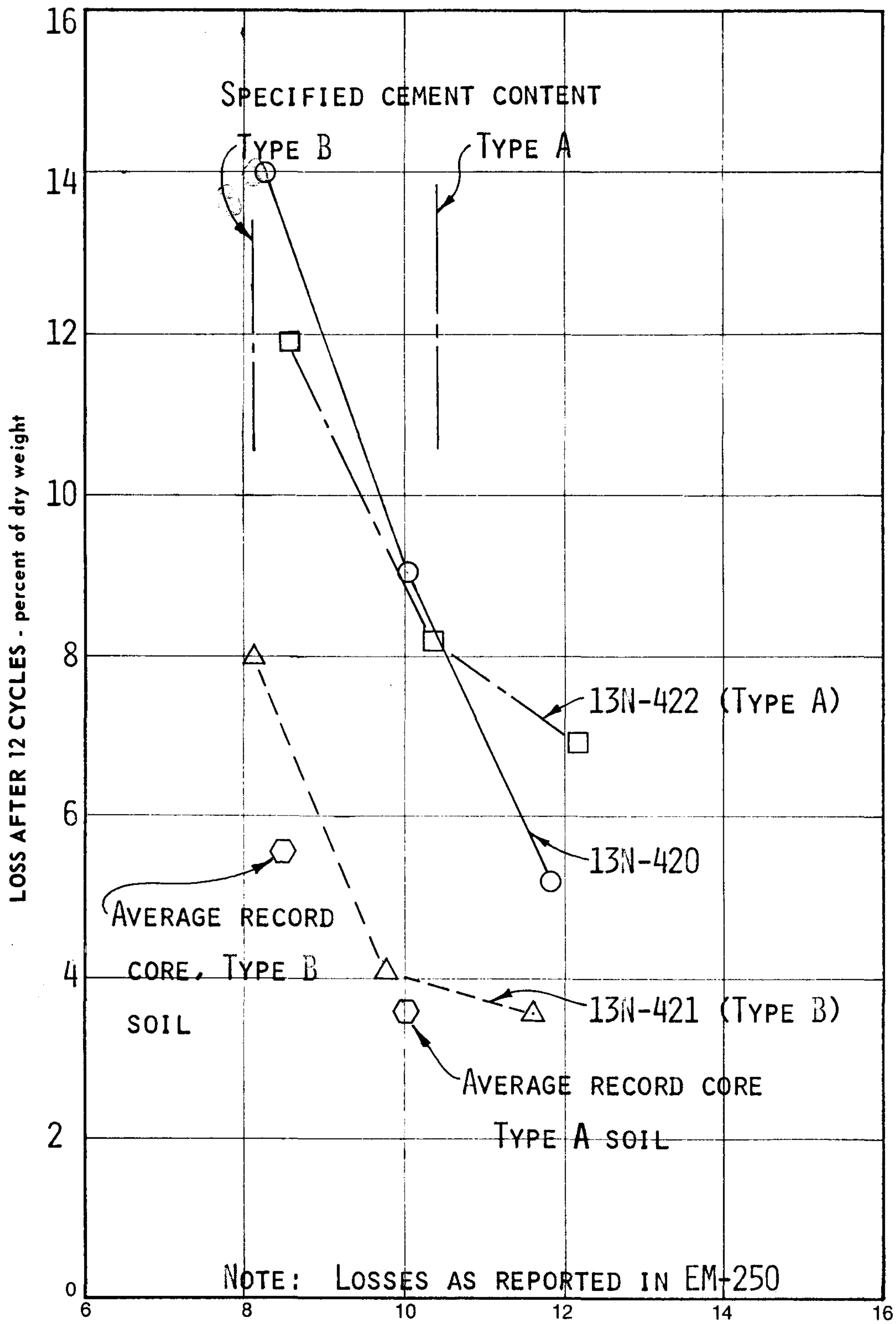
GRAVEL .... ___ %

SAND .... ___ %

SILT TO CLAY ...... ___ %

LABORATORY SAMPLE No. ______ FIELD DESIGNATION ______ EXCAVATION No. ______ DEPTH ______ FT.

SOIL-CEMENT DURABILITY TEST RESULTS

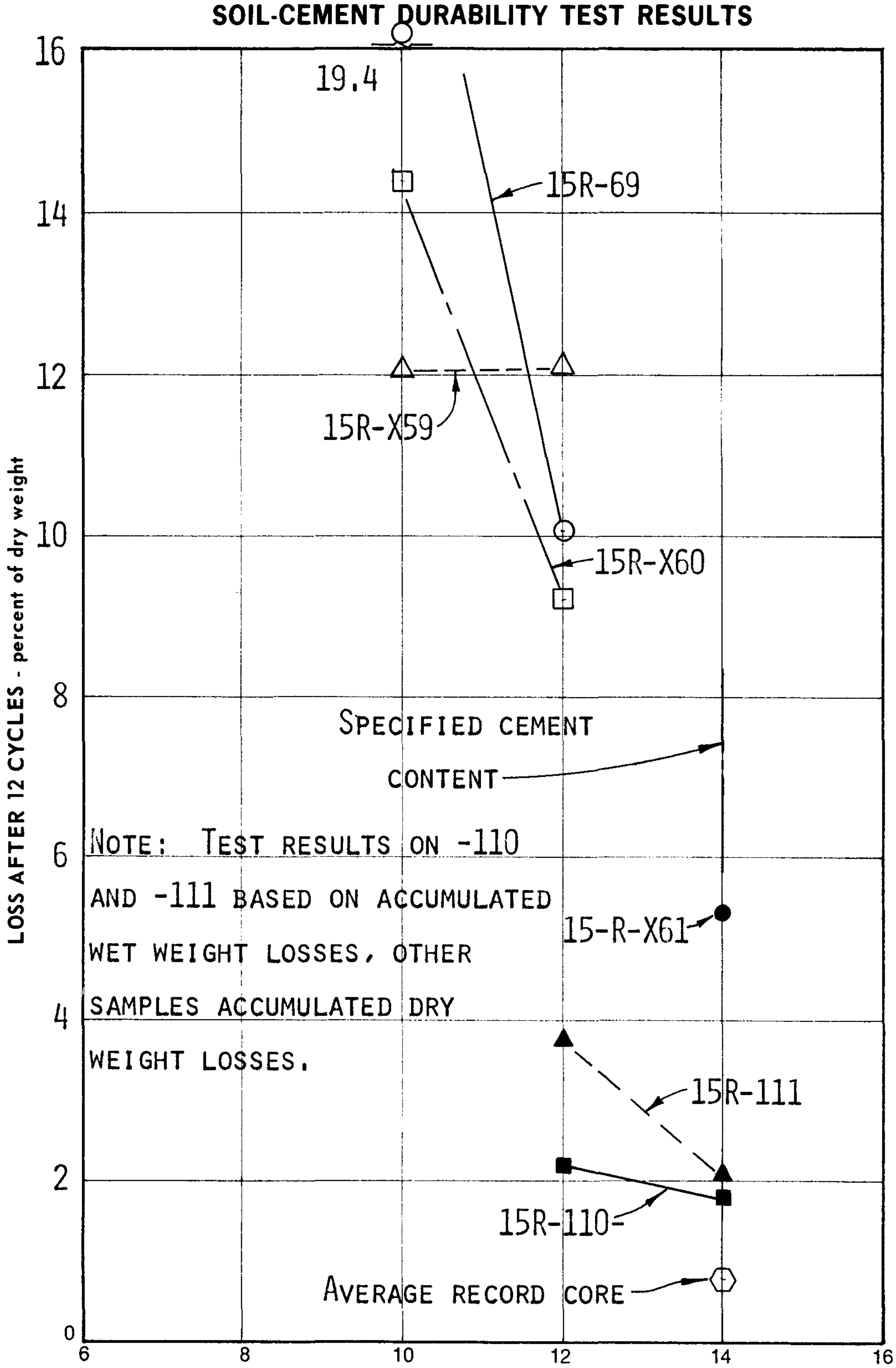


CEMENT CONTENT - percent by dry weight of soil

FREEZE-THAW _X_ WET-DRY ____

BONNY TEST SECTION

SOIL-CEMENT DURABILITY TEST RESULTS

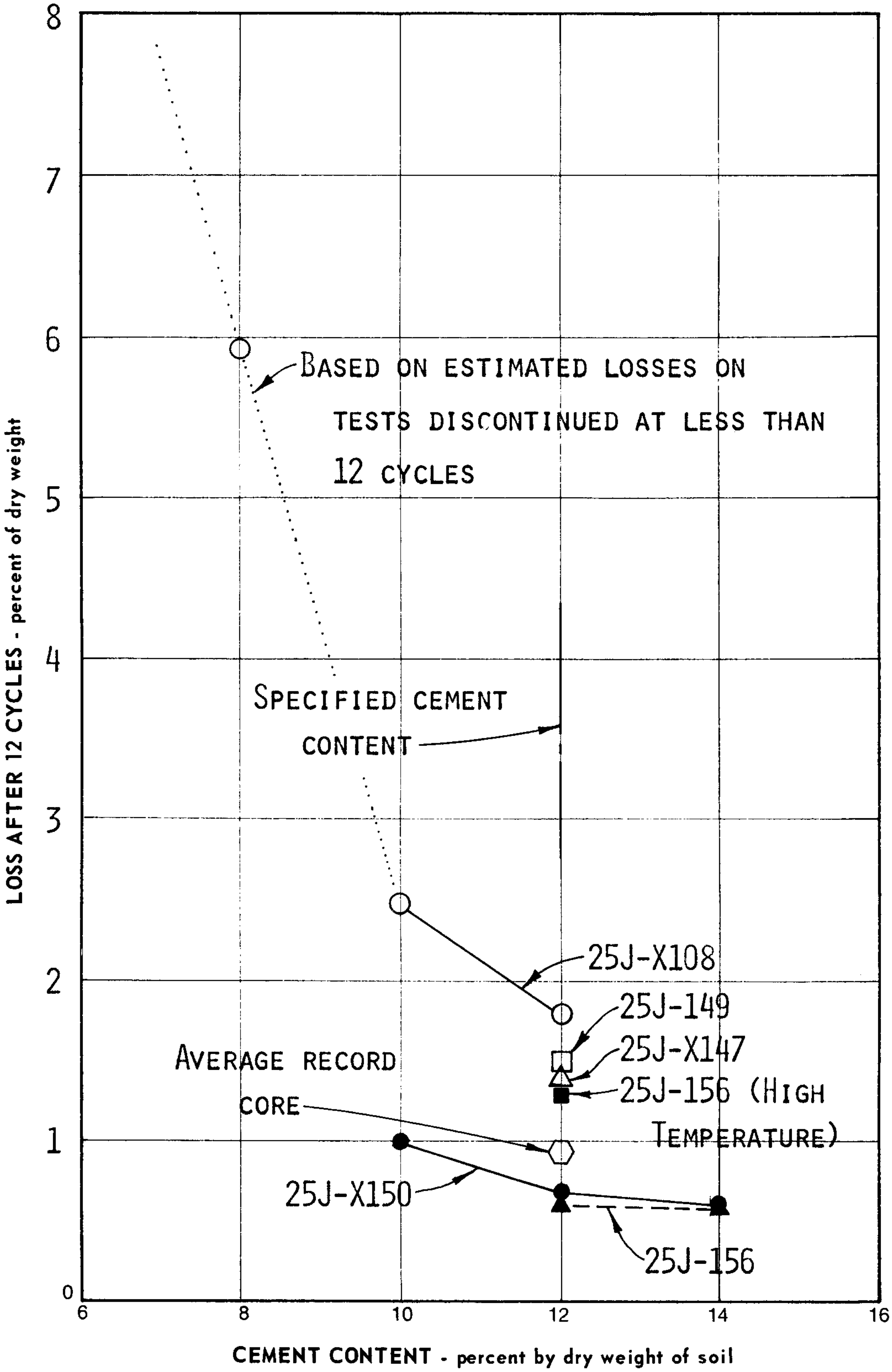


CEMENT CONTENT - percent by dry weight of soil

FREEZE-THAW X WET-DRY ____

MERRITT DAM

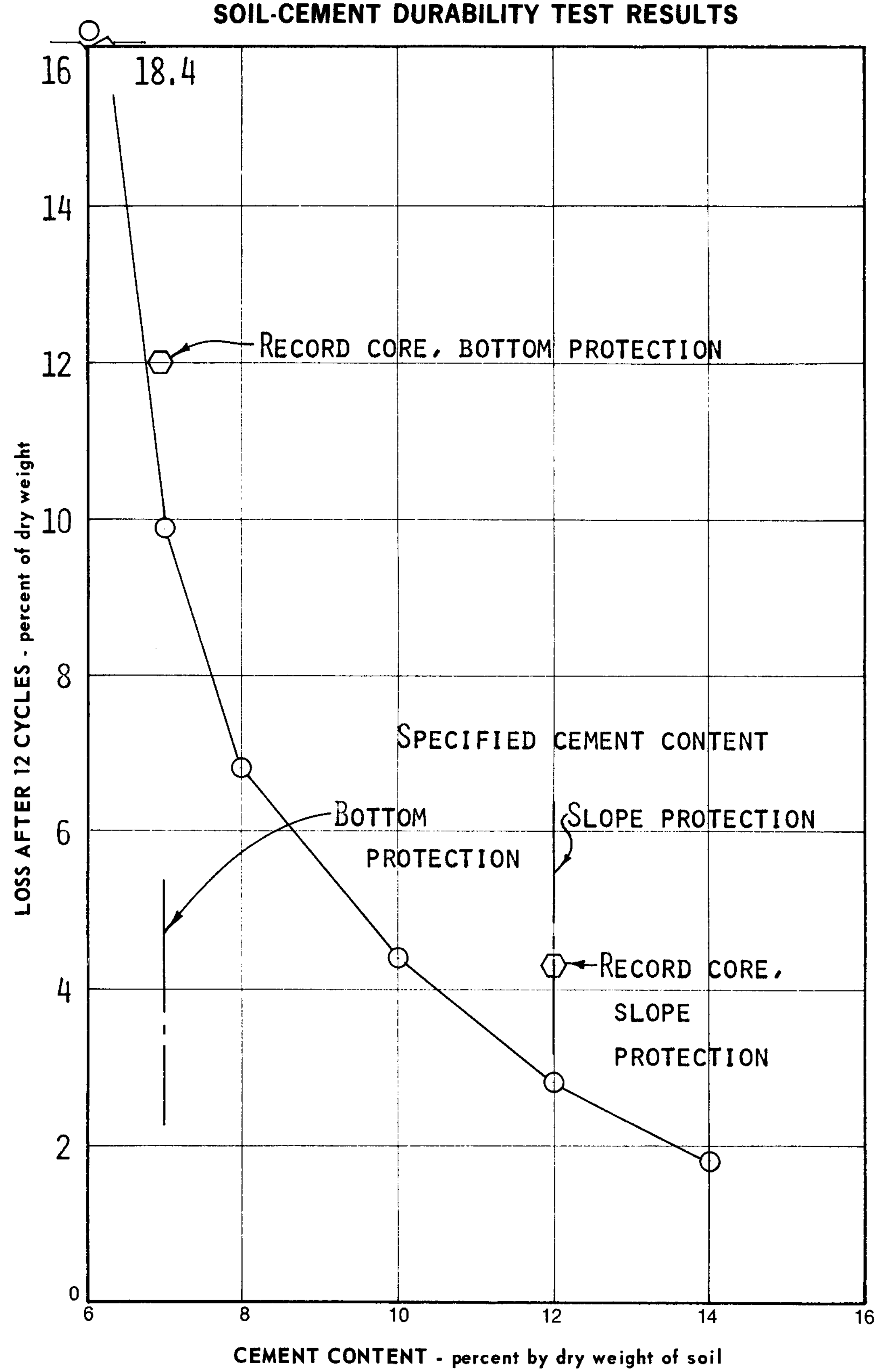
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW X WET-DRY ____

CHENEY DAM

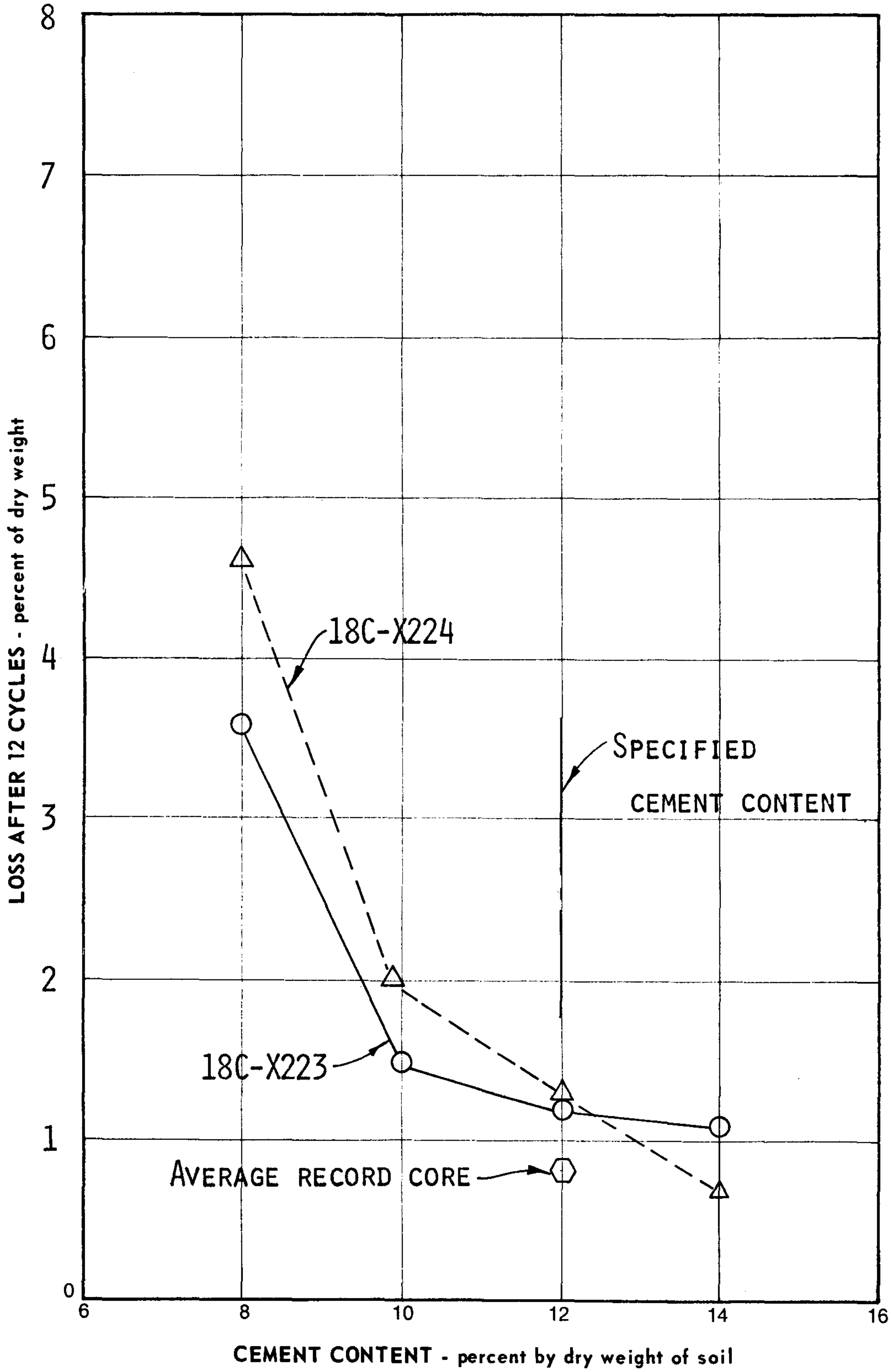
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW __X__ WET-DRY ____

LUBBOCK REGULATING RESERVOIR

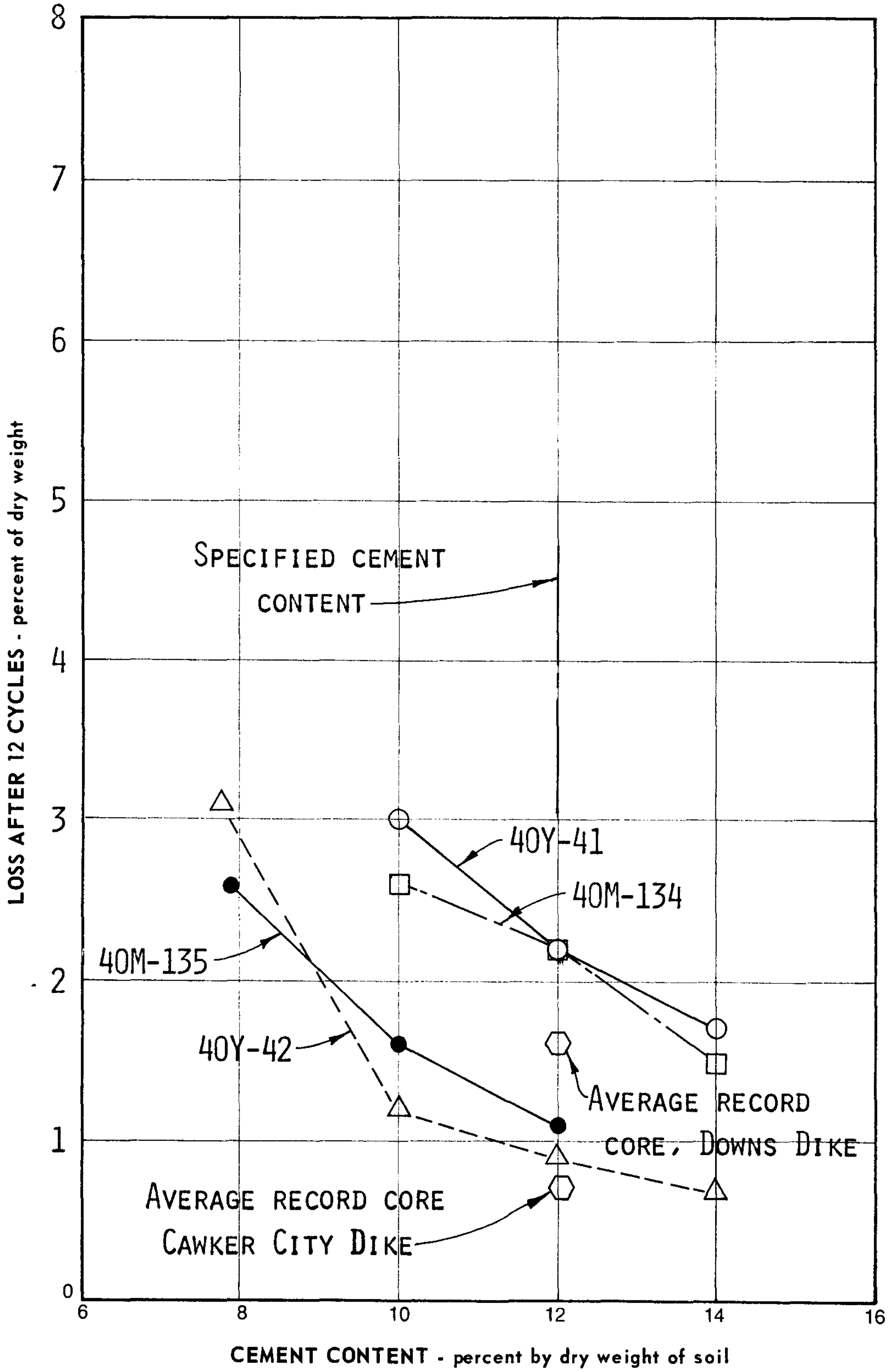
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW X WET-DRY ____

GLEN ELDER DAM

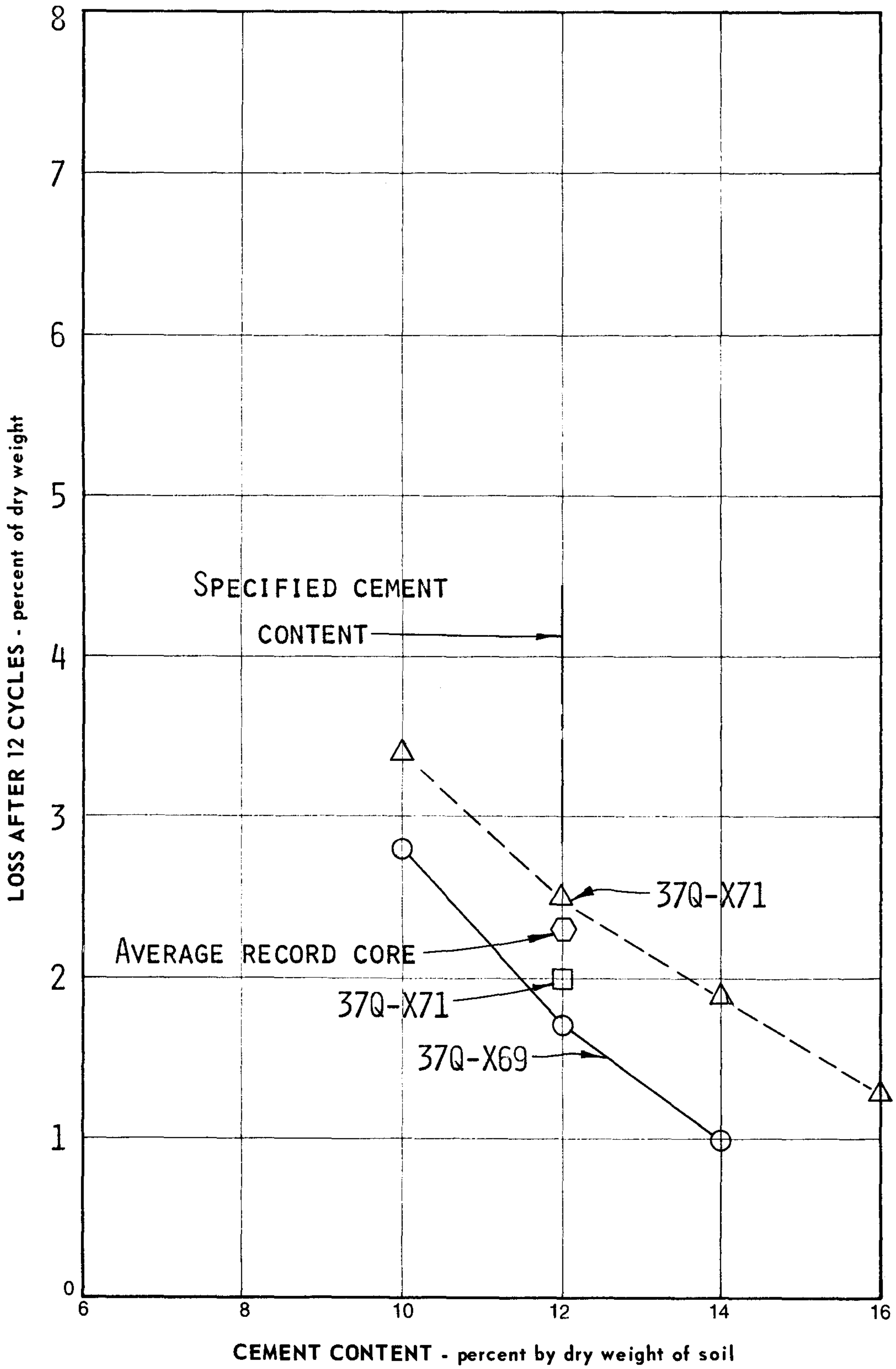
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW X WET-DRY____

DOWNS DIKE 40Y
CAWKER CITY DIKE 40M

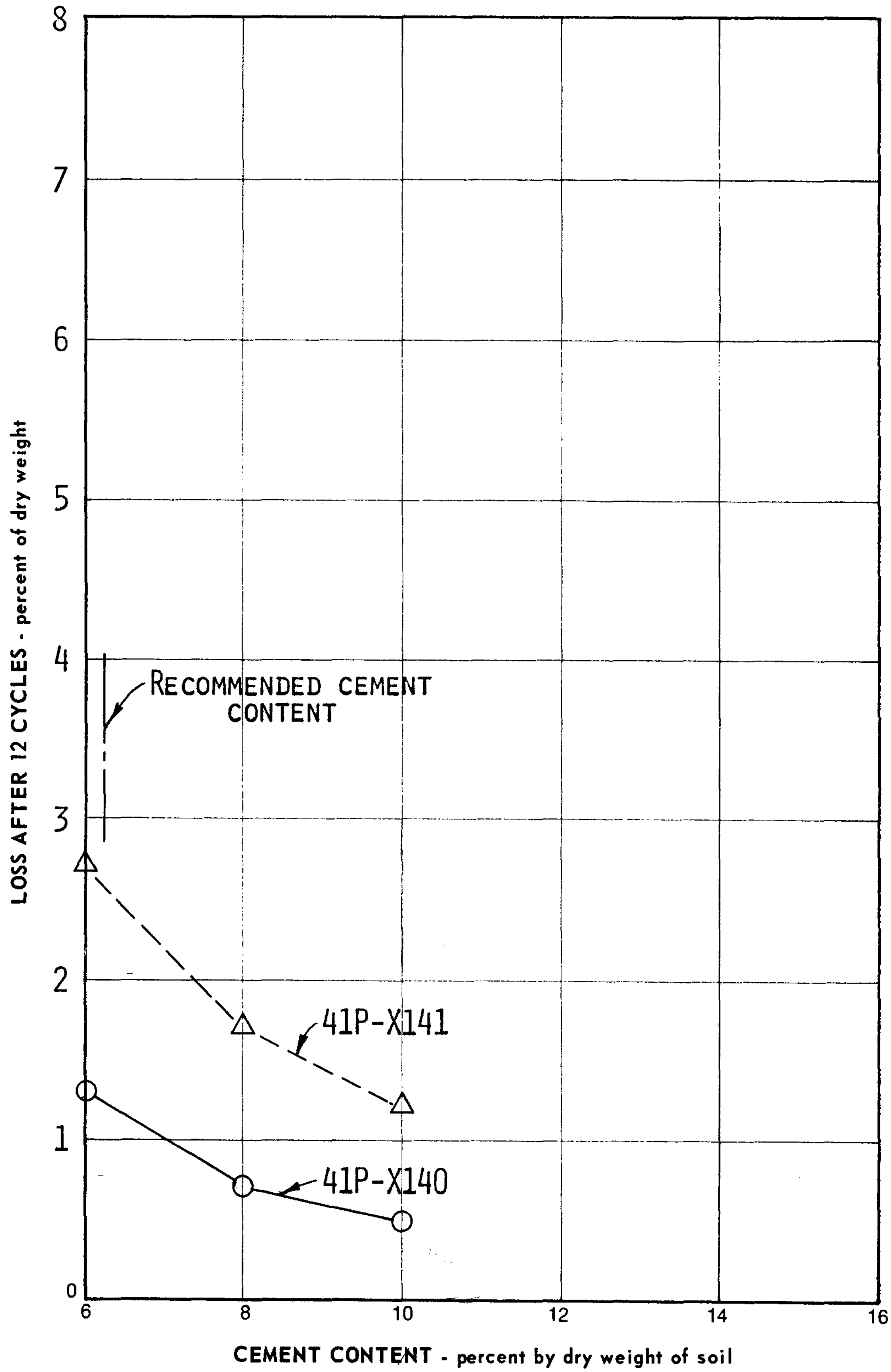
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW X WET-DRY ____

STARVATION DAM

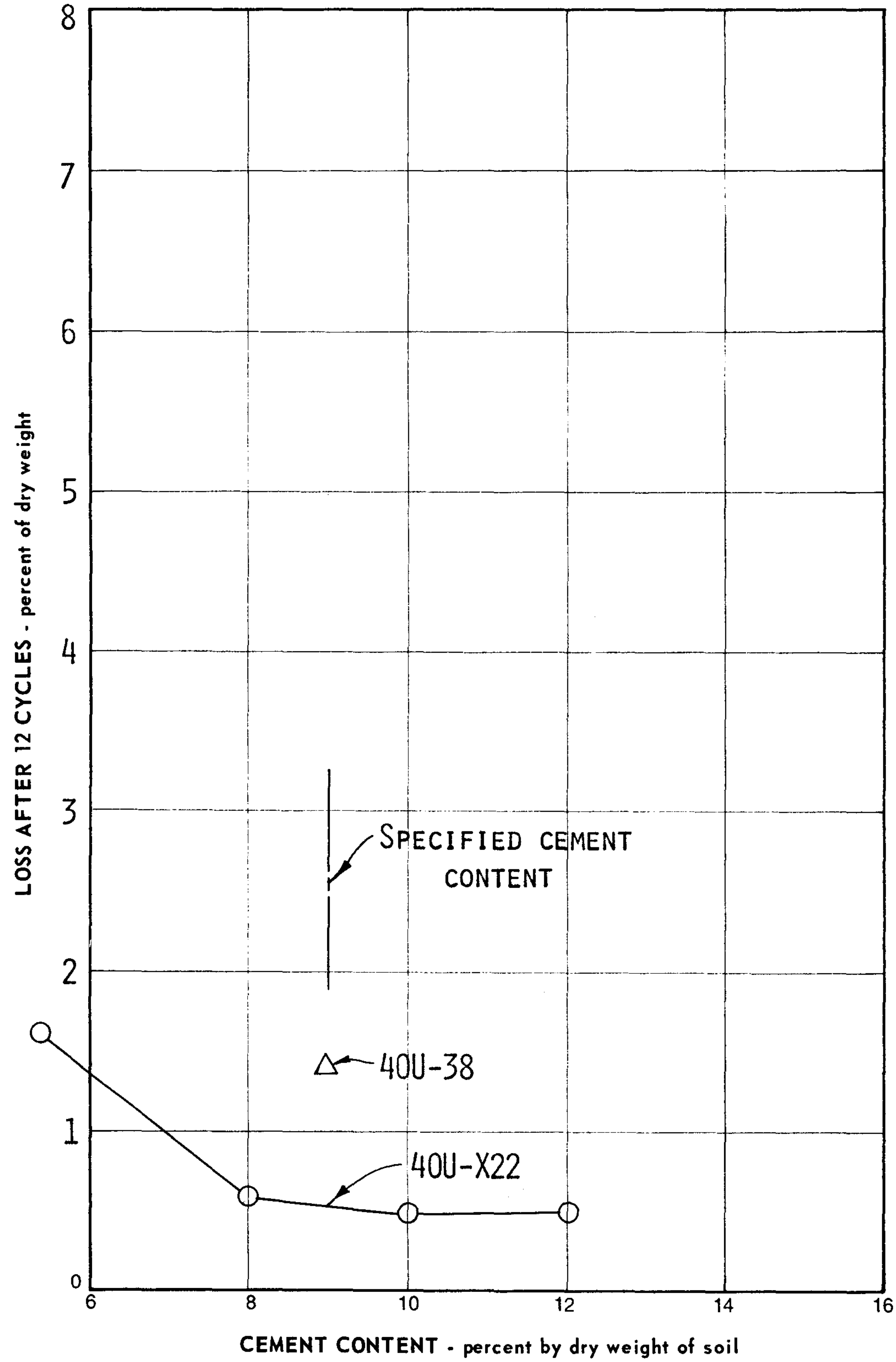
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW X WET-DRY ____

LITTLE PANOCHE CREEK DETENTION DAM

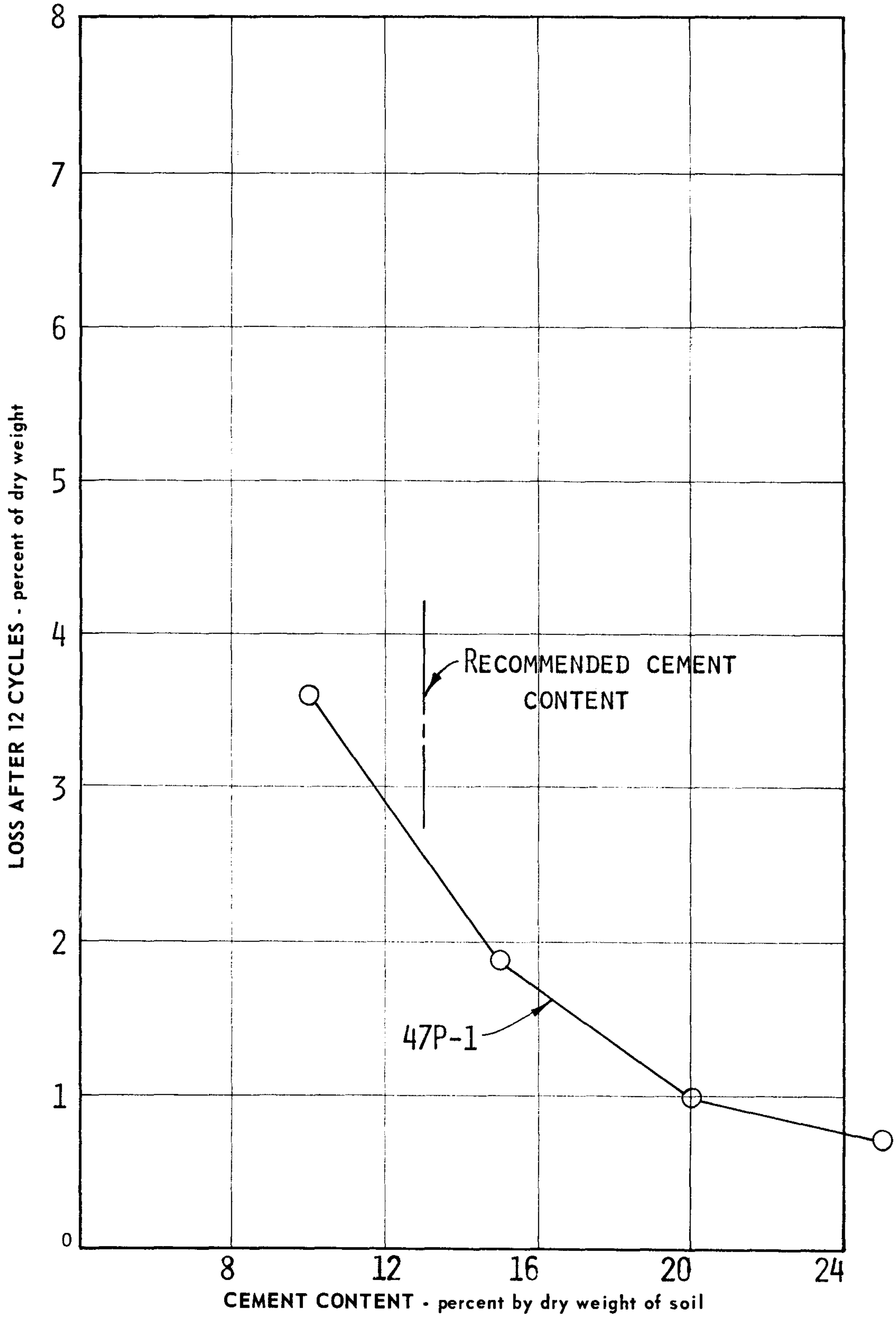
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW X WET-DRY____

RED BLUFF RESERVOIR

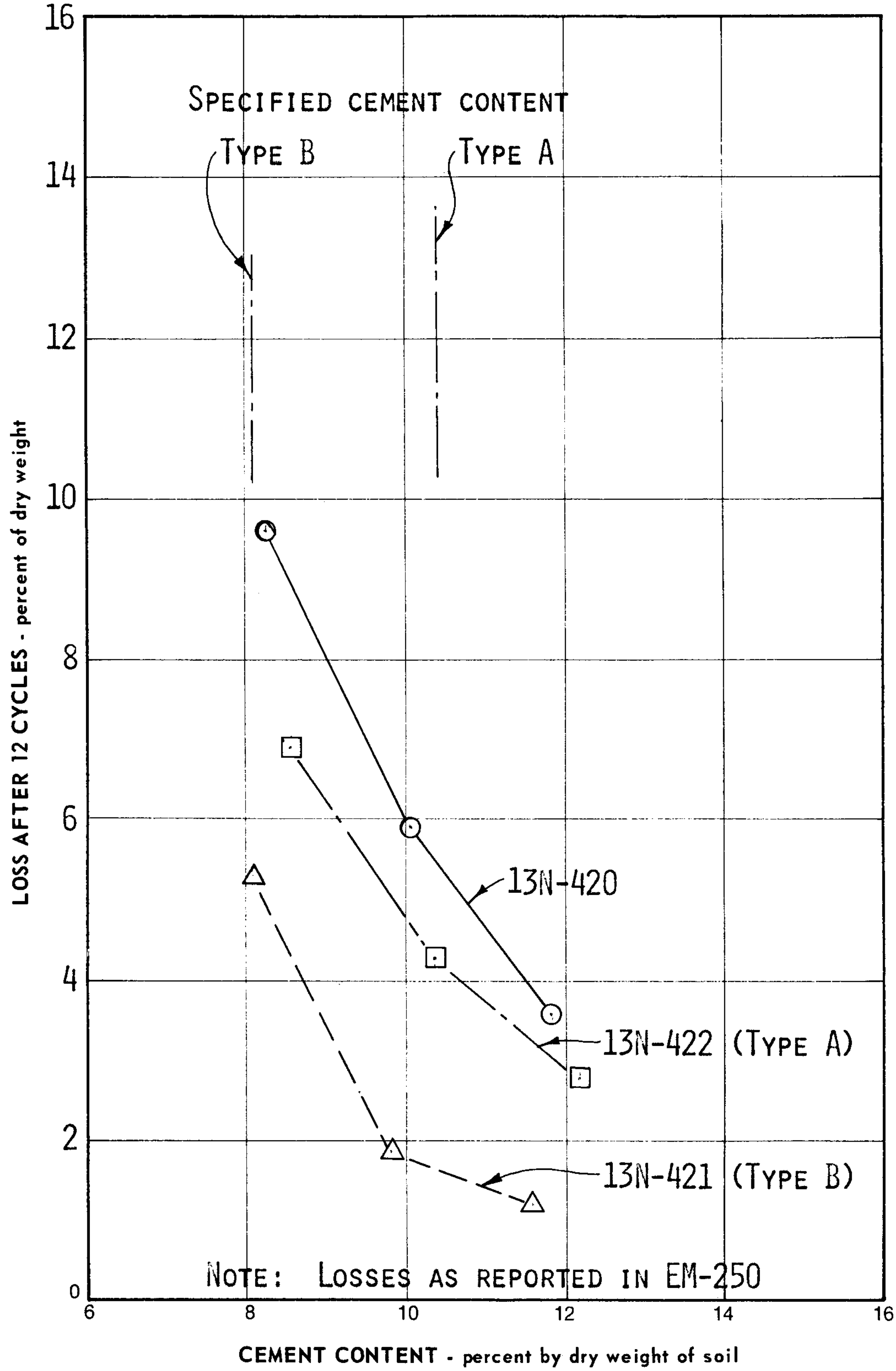
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW X WET-DRY ____

CONCONULLY DAM

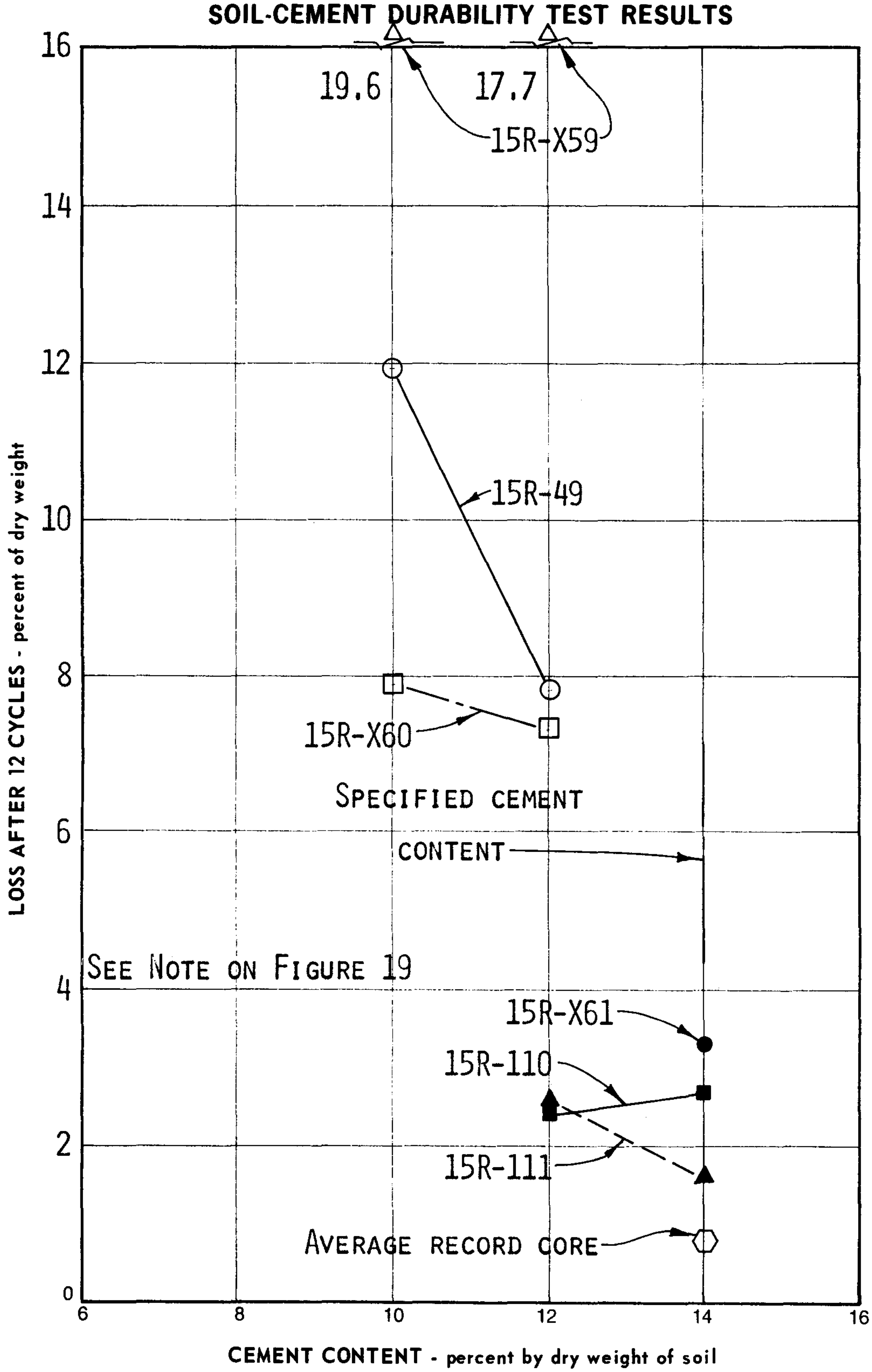
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW____ WET-DRY X

BONNY TEST SECTION

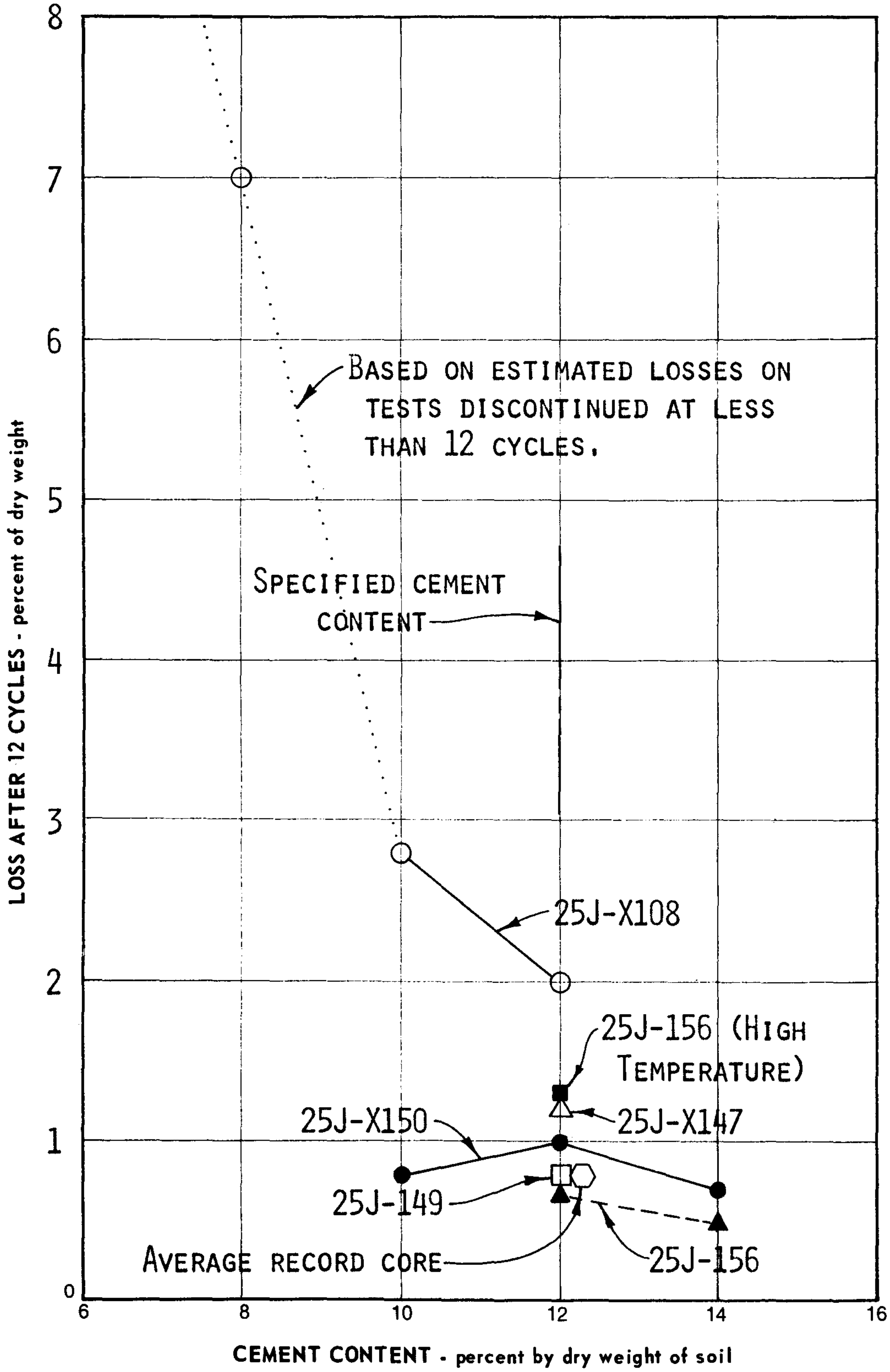
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW____ WET-DRY X

MERRITT DAM

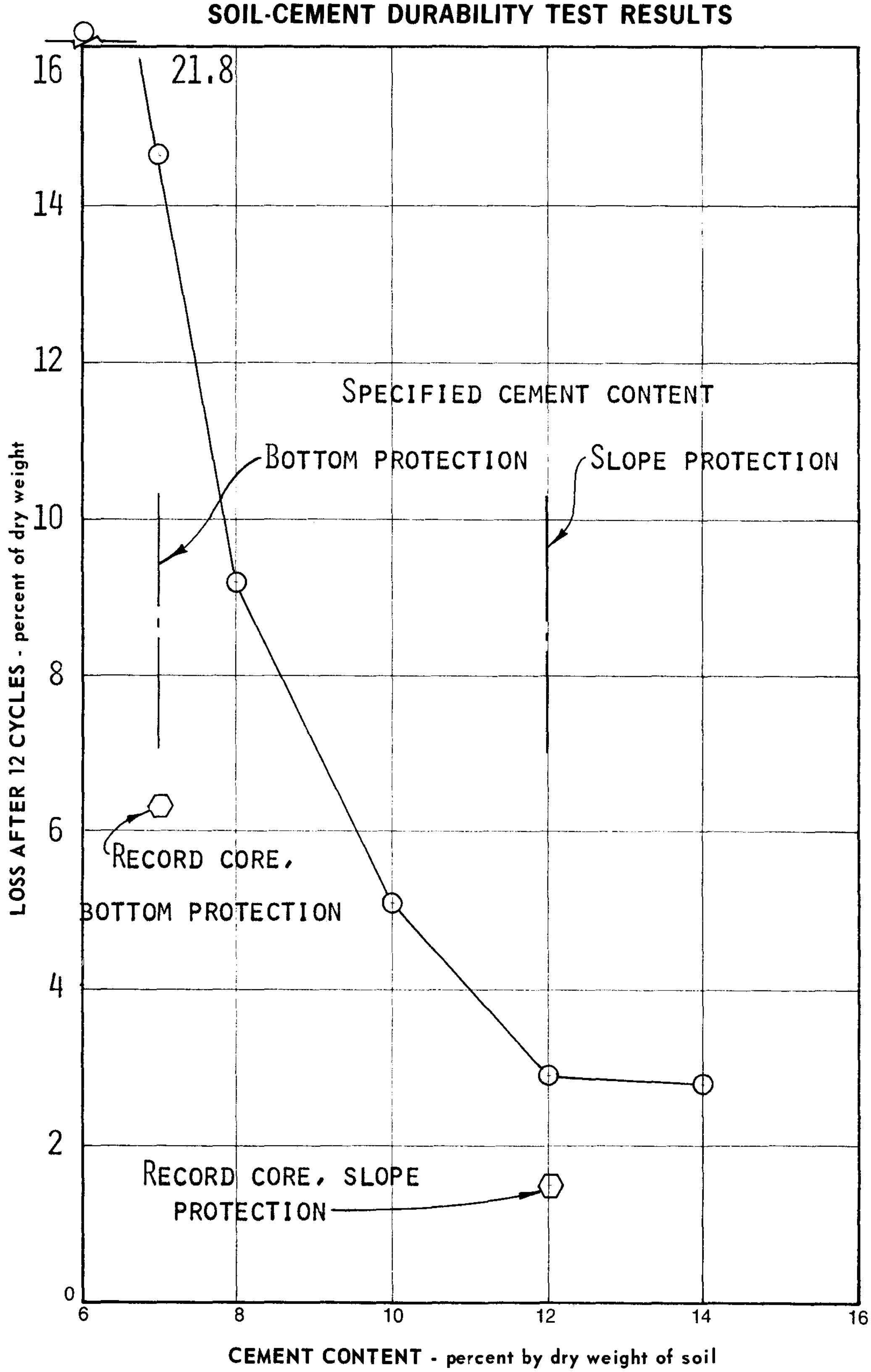
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW____ WET-DRY X

CHENEY DAM

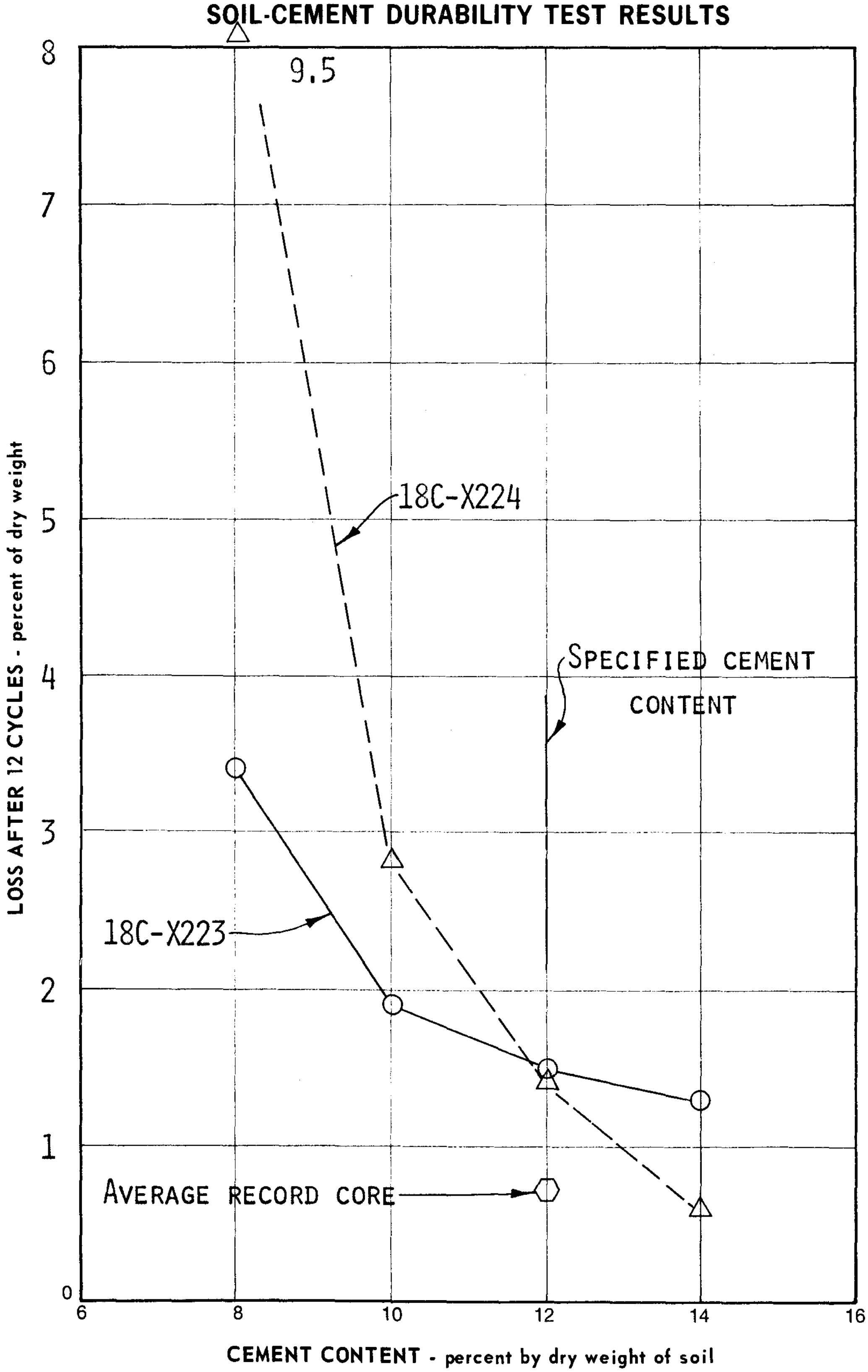
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW____ WET-DRY X

LUBBOCK REGULATING RESERVOIR

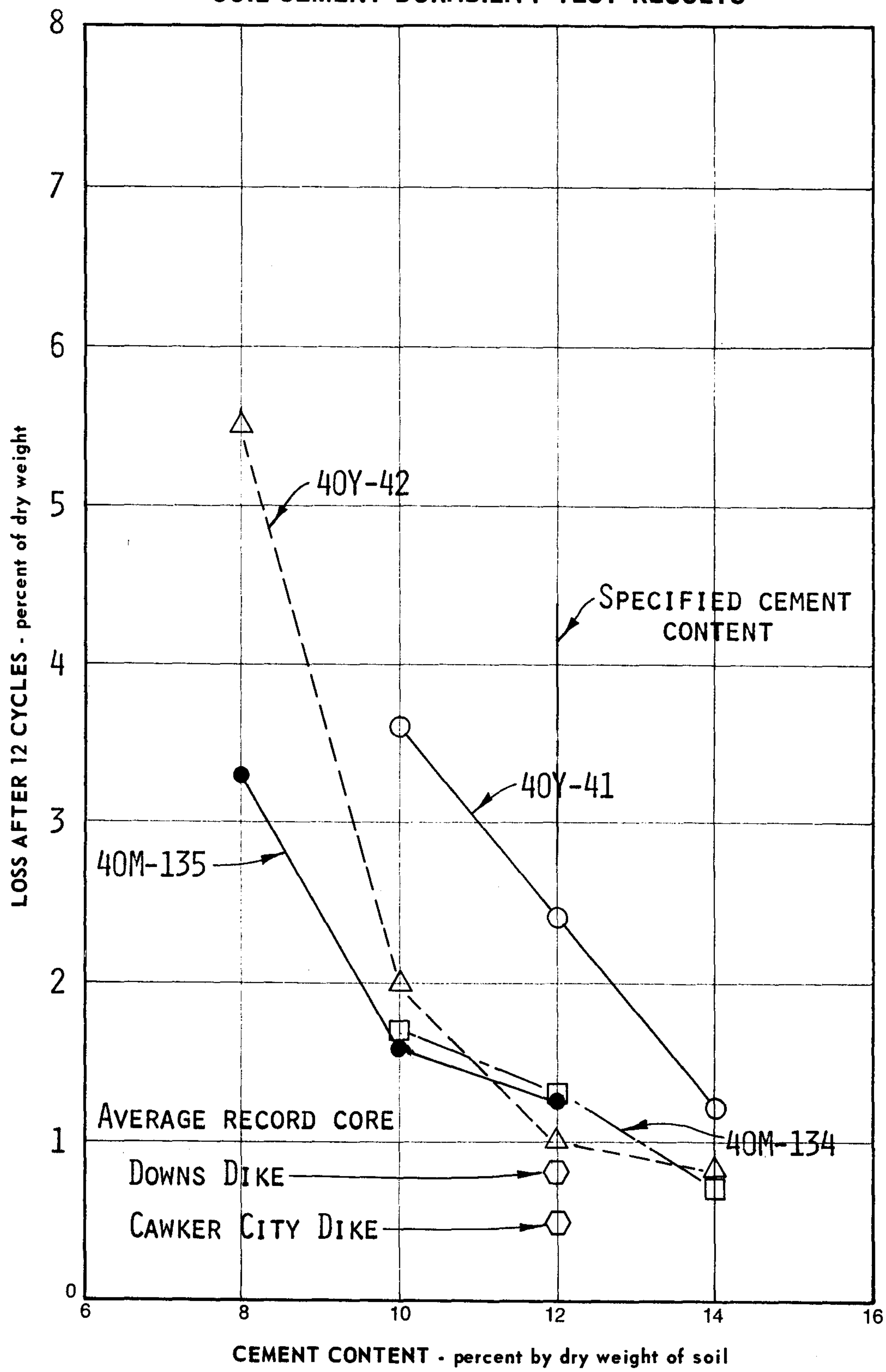
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW____ WET-DRY X

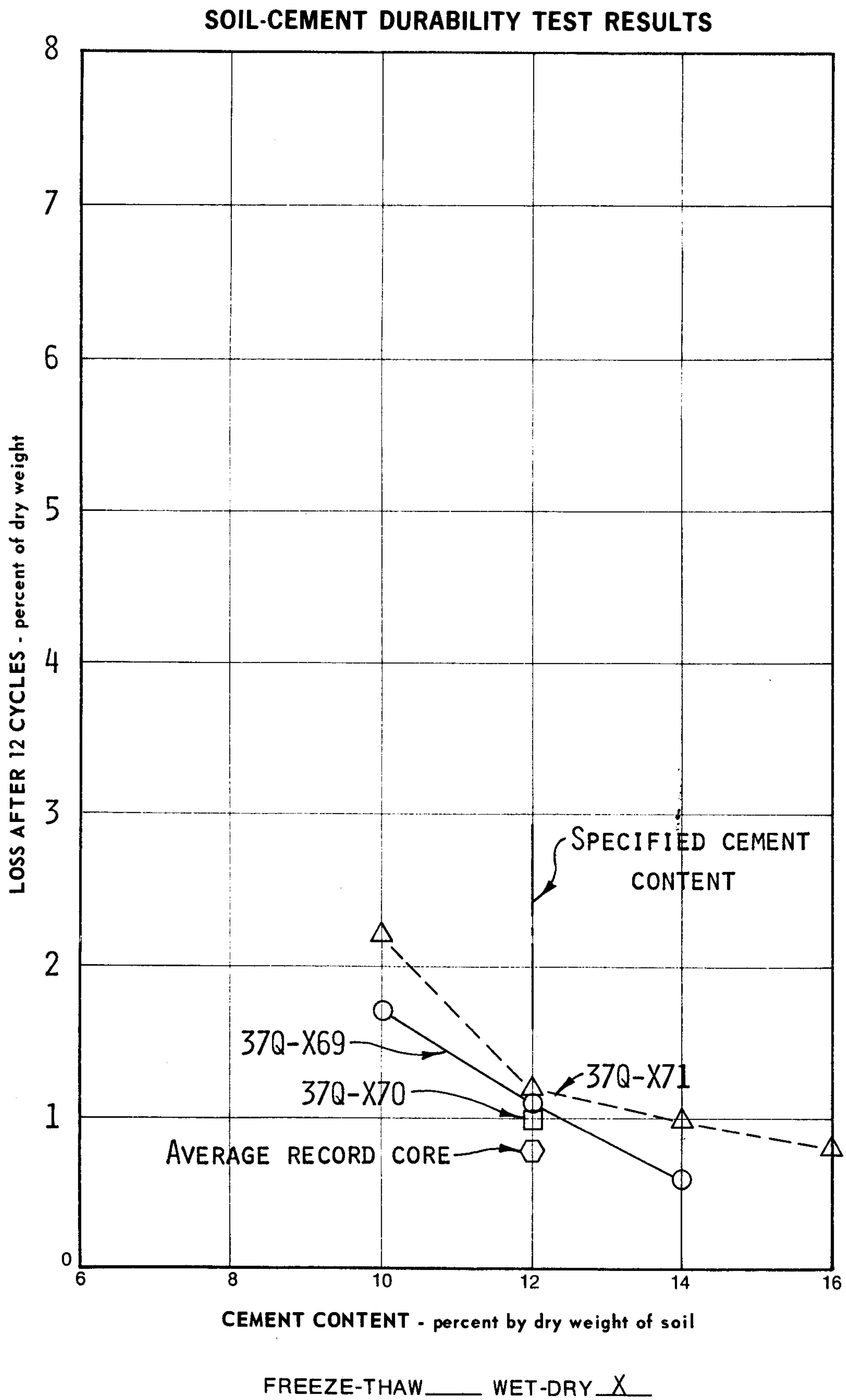
GLEN ELDER DAM

SOIL-CEMENT DURABILITY TEST RESULTS



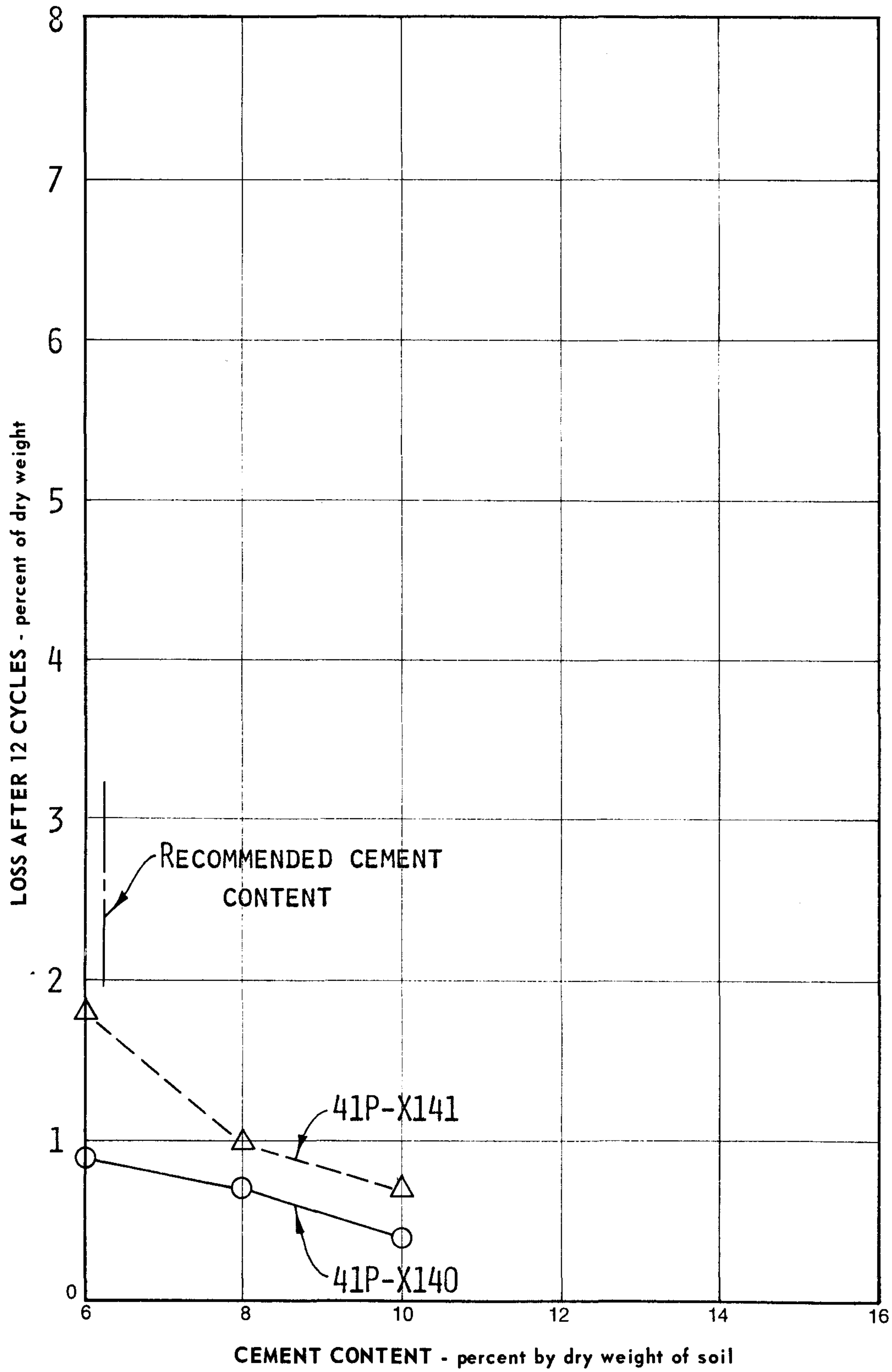
FREEZE-THAW____ WET-DRY X

DOWNS DIKE 40Y
CAWKER CITY DIKE 40M



STARVATION DAM

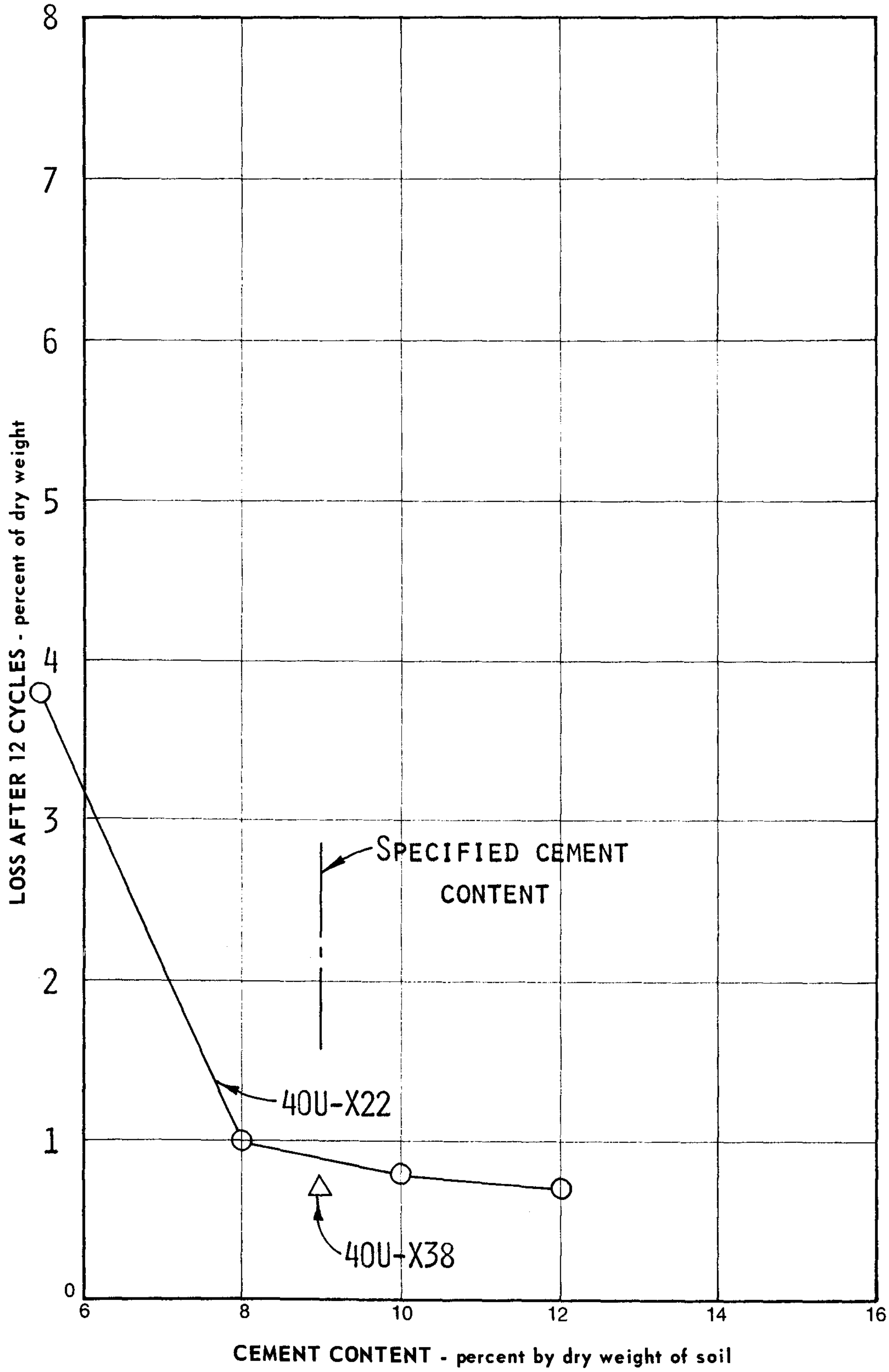
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW_____ WET-DRY _X_

LITTLE PANOCHE CREEK DETENTION DAM

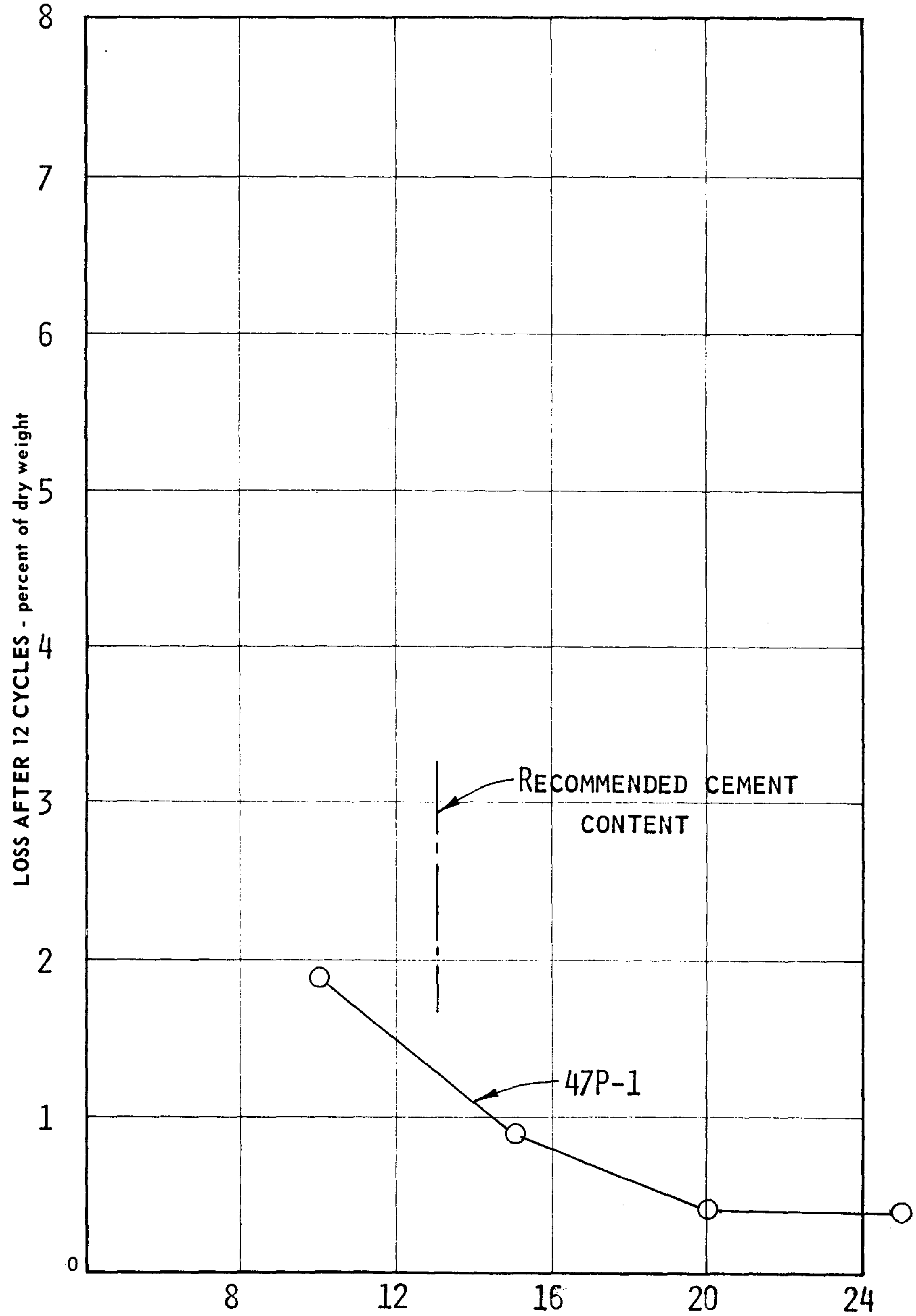
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW____ WET-DRY X

RED BLUFF RESERVOIR

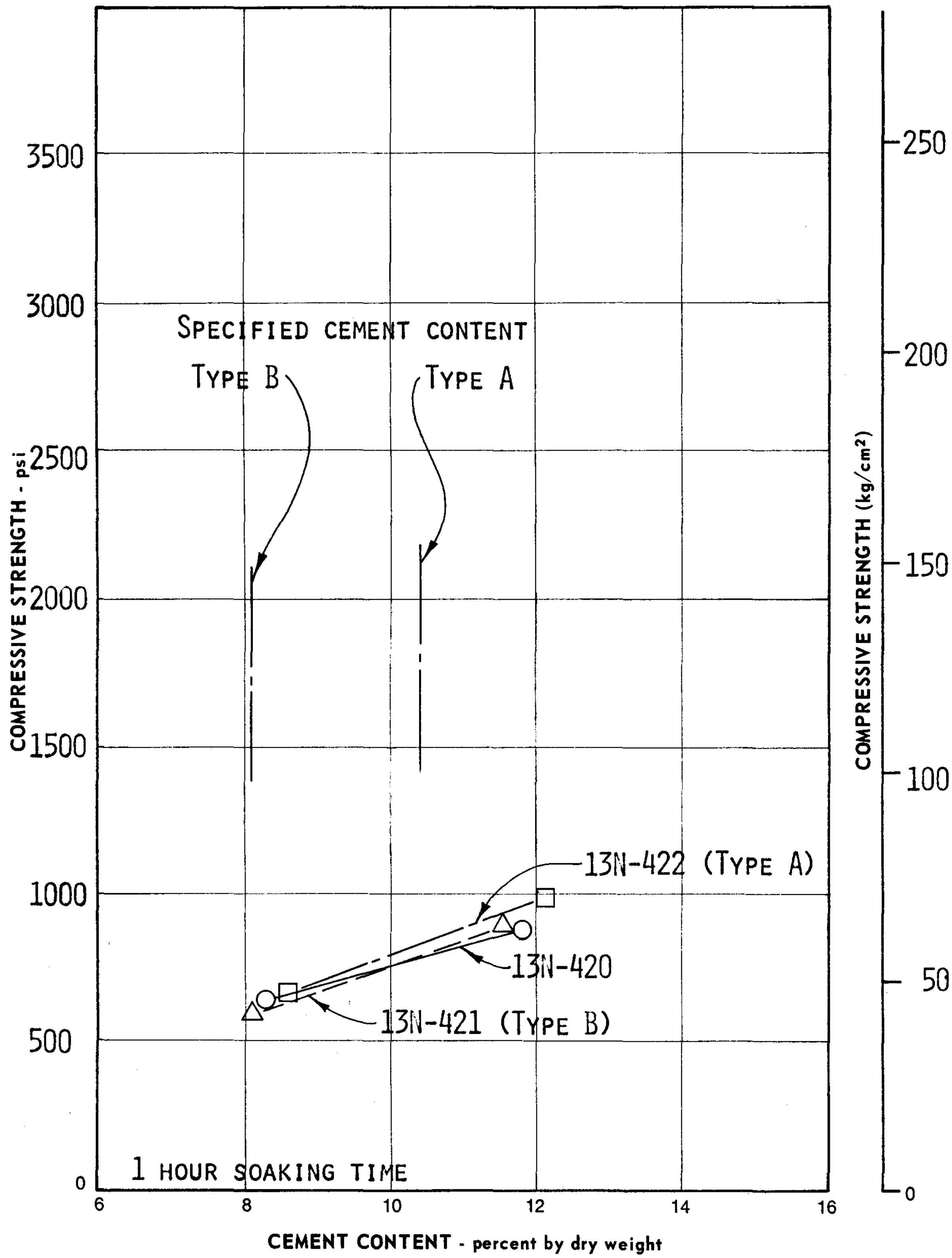
SOIL-CEMENT DURABILITY TEST RESULTS



FREEZE-THAW____ WET-DRY X

CONCONULLY DAM

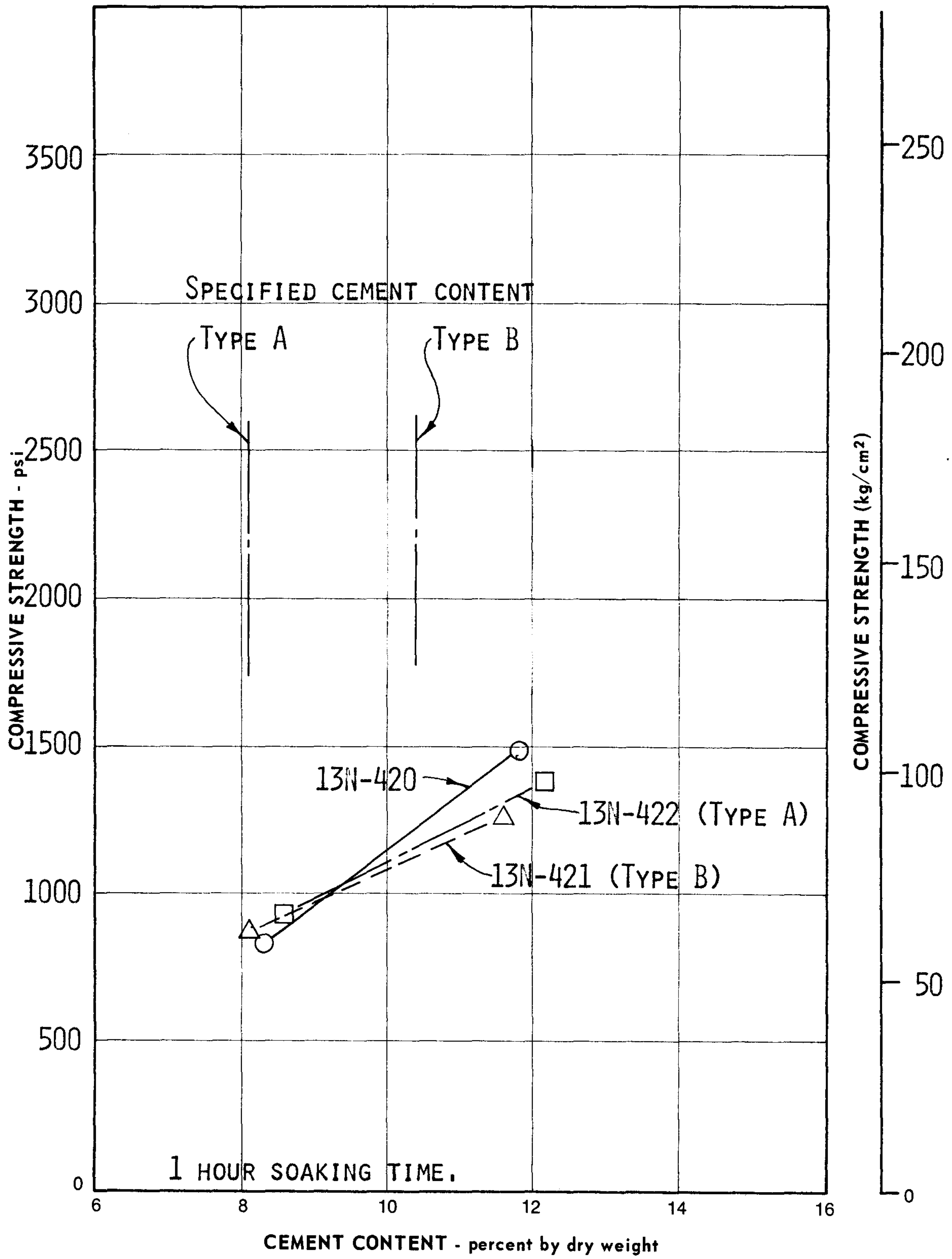
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2 x 2 in. (5 x 5 cm) CURE 7 days

BONNY TEST SECTION

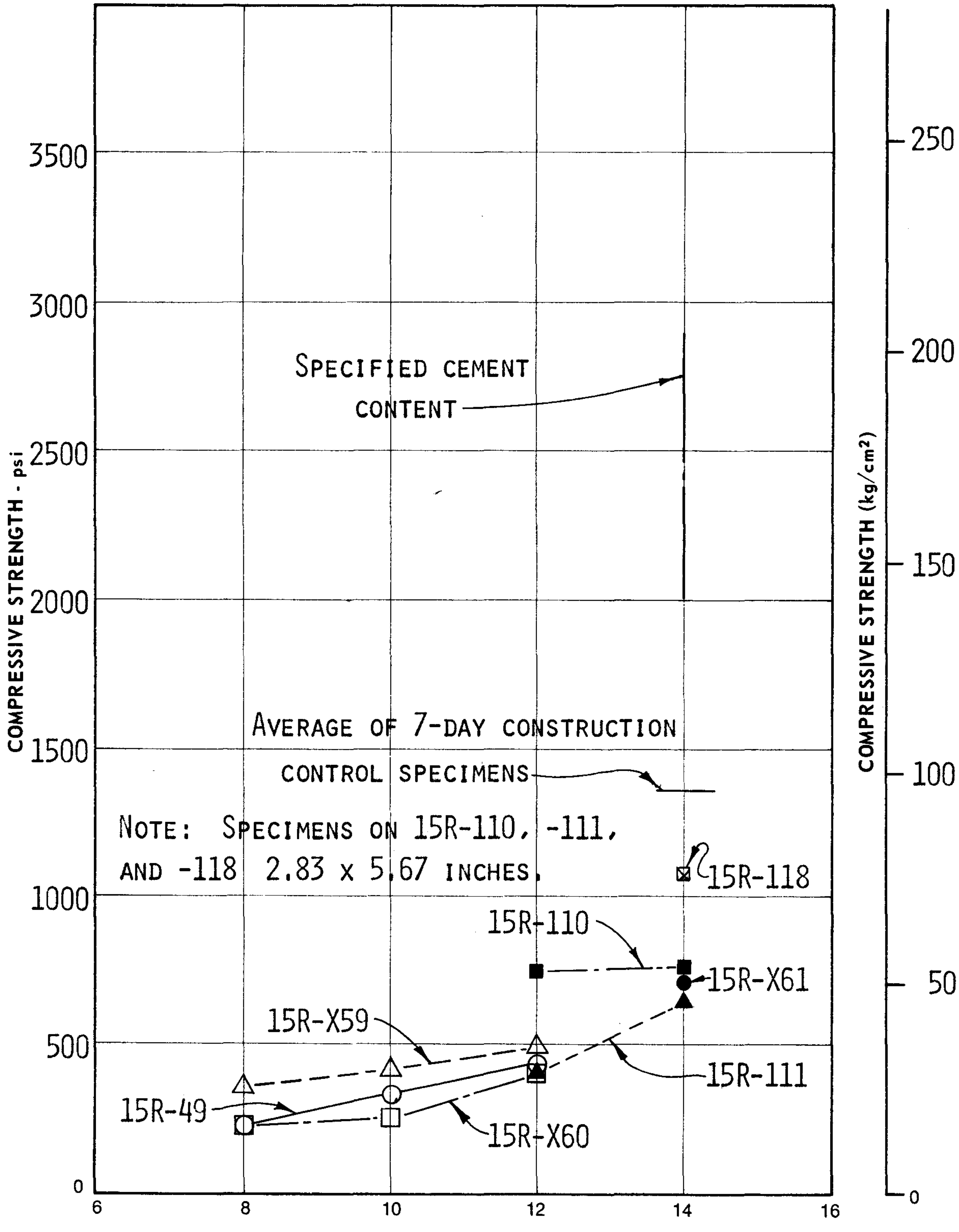
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2 x 2 in. (5 x 5 cm) CURE 28 days

BONNY TEST SECTION

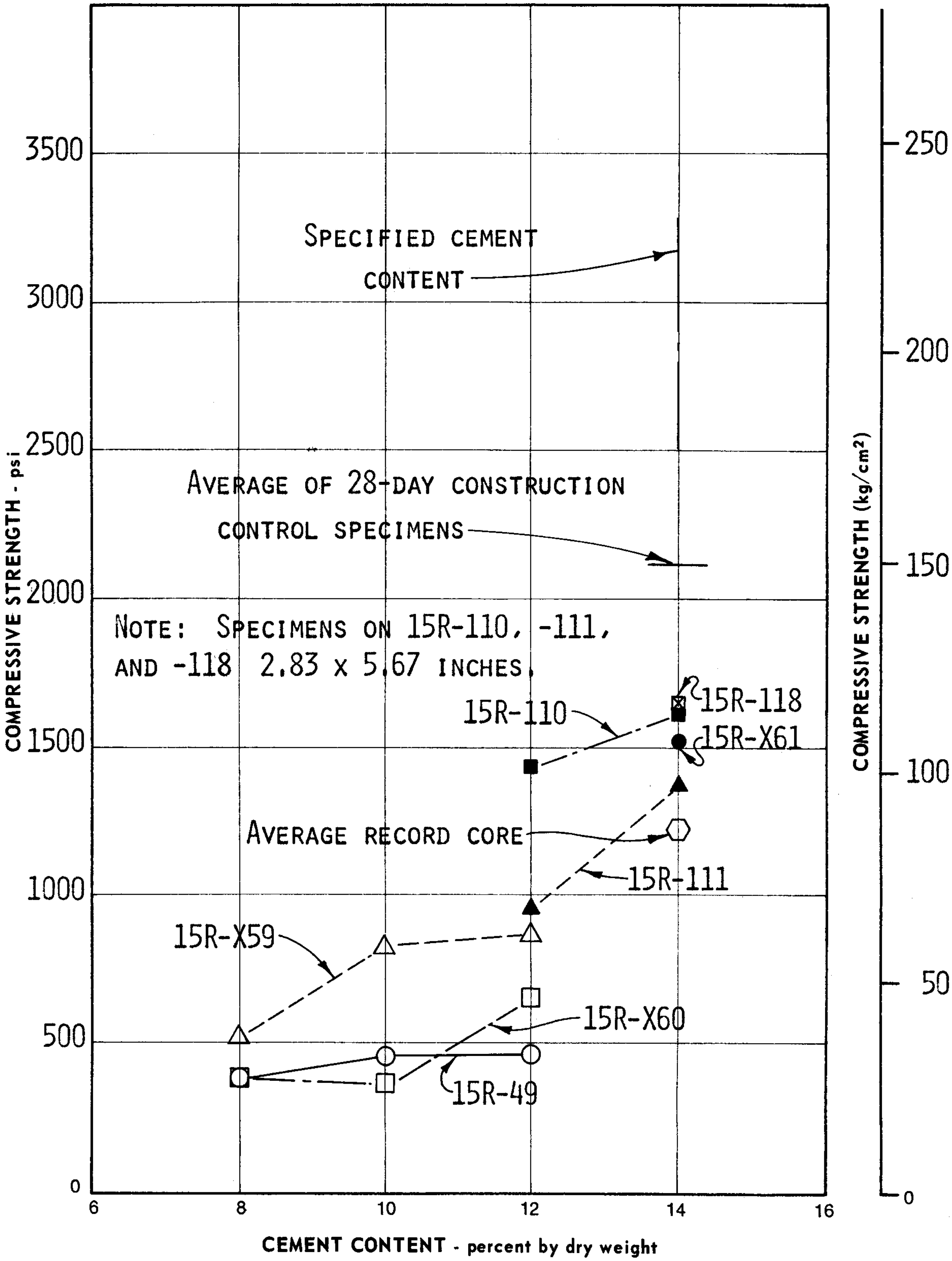
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 2.83 in. (7.2 x 7.2 cm) CURE 7 days

MERRITT DAM

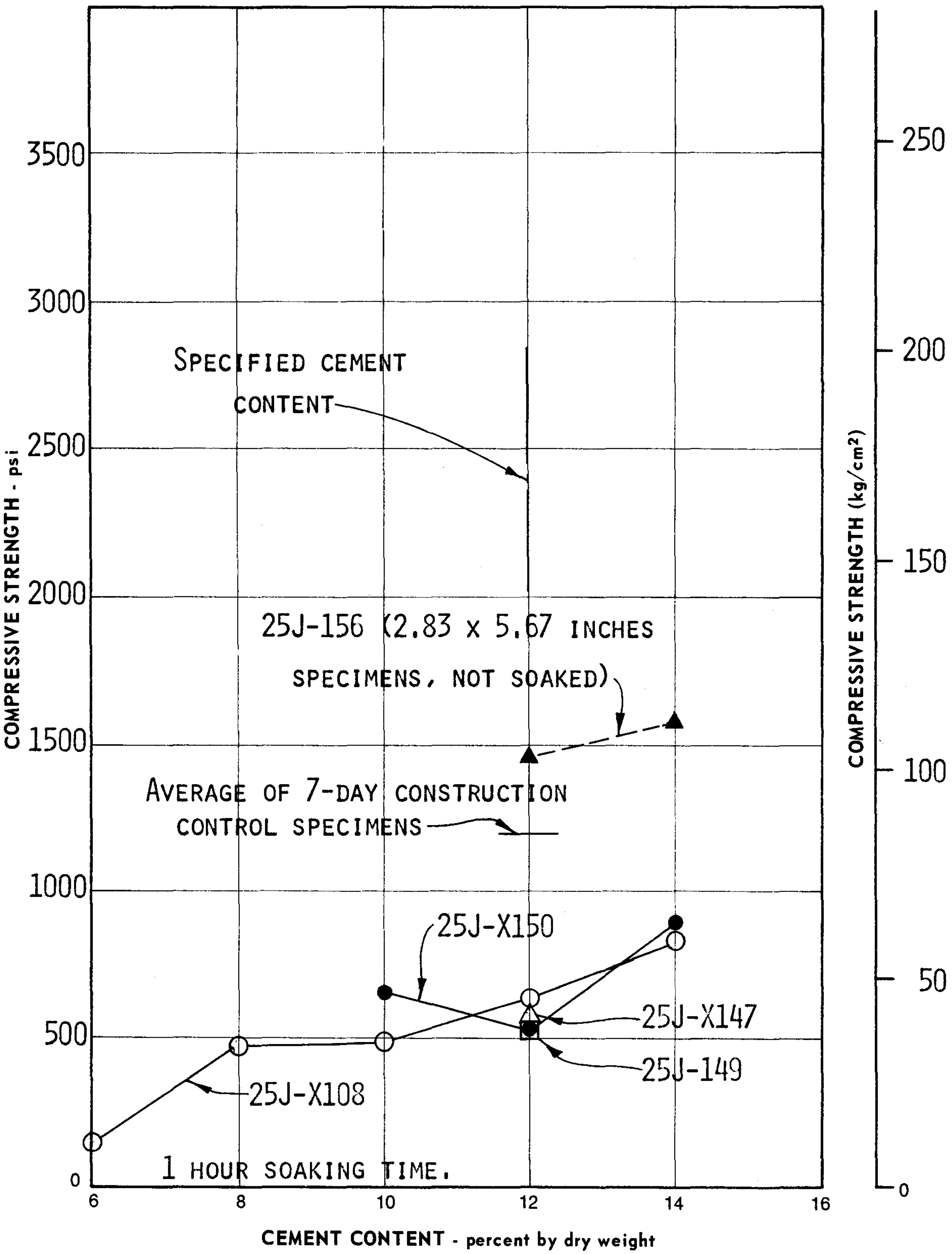
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 2.83 in. (7.2 x 7.2 cm) CURE 28 days

MERRITT DAM

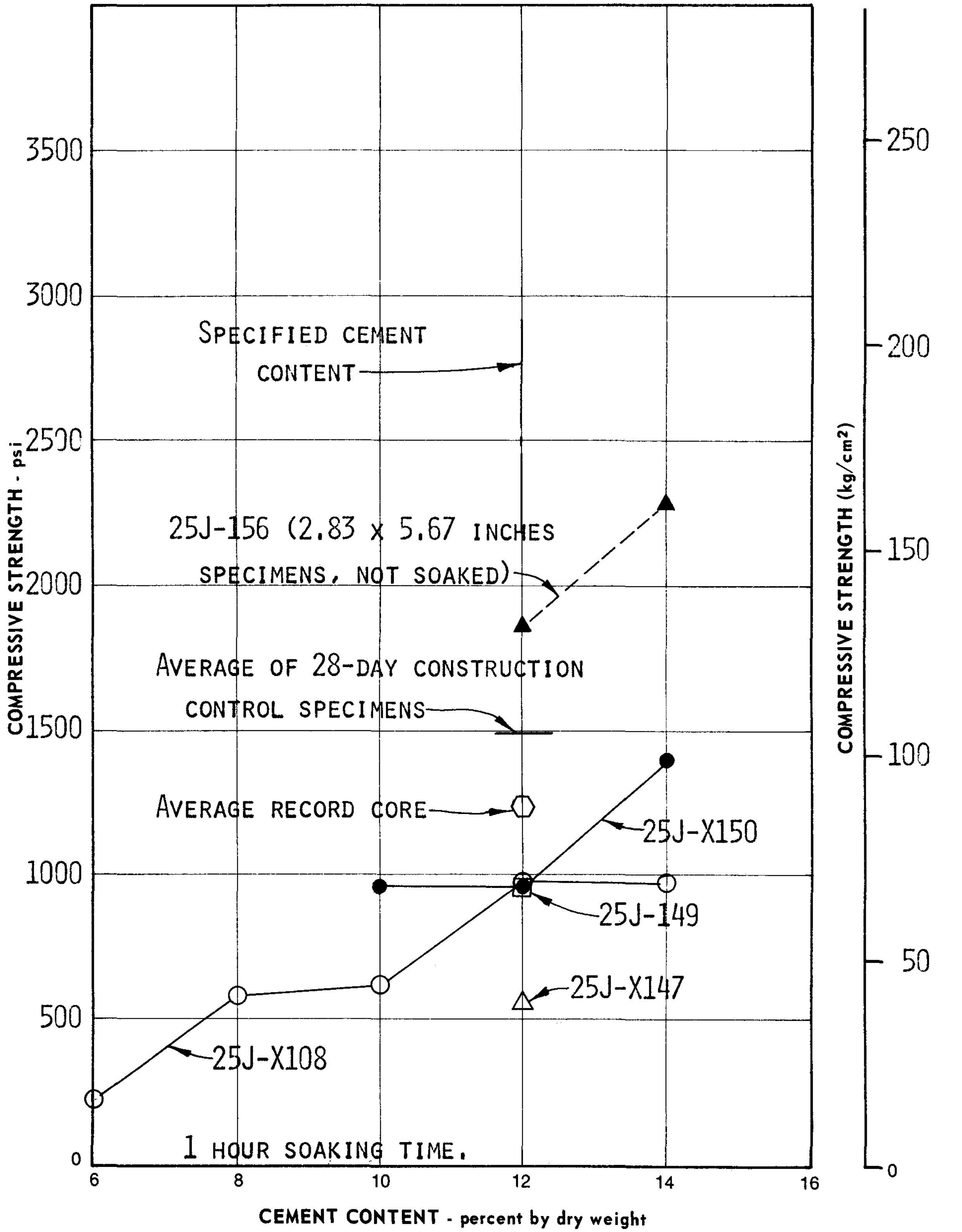
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2 x 2 in. (5 x 5 cm) CURE 7 days

CHENEY DAM

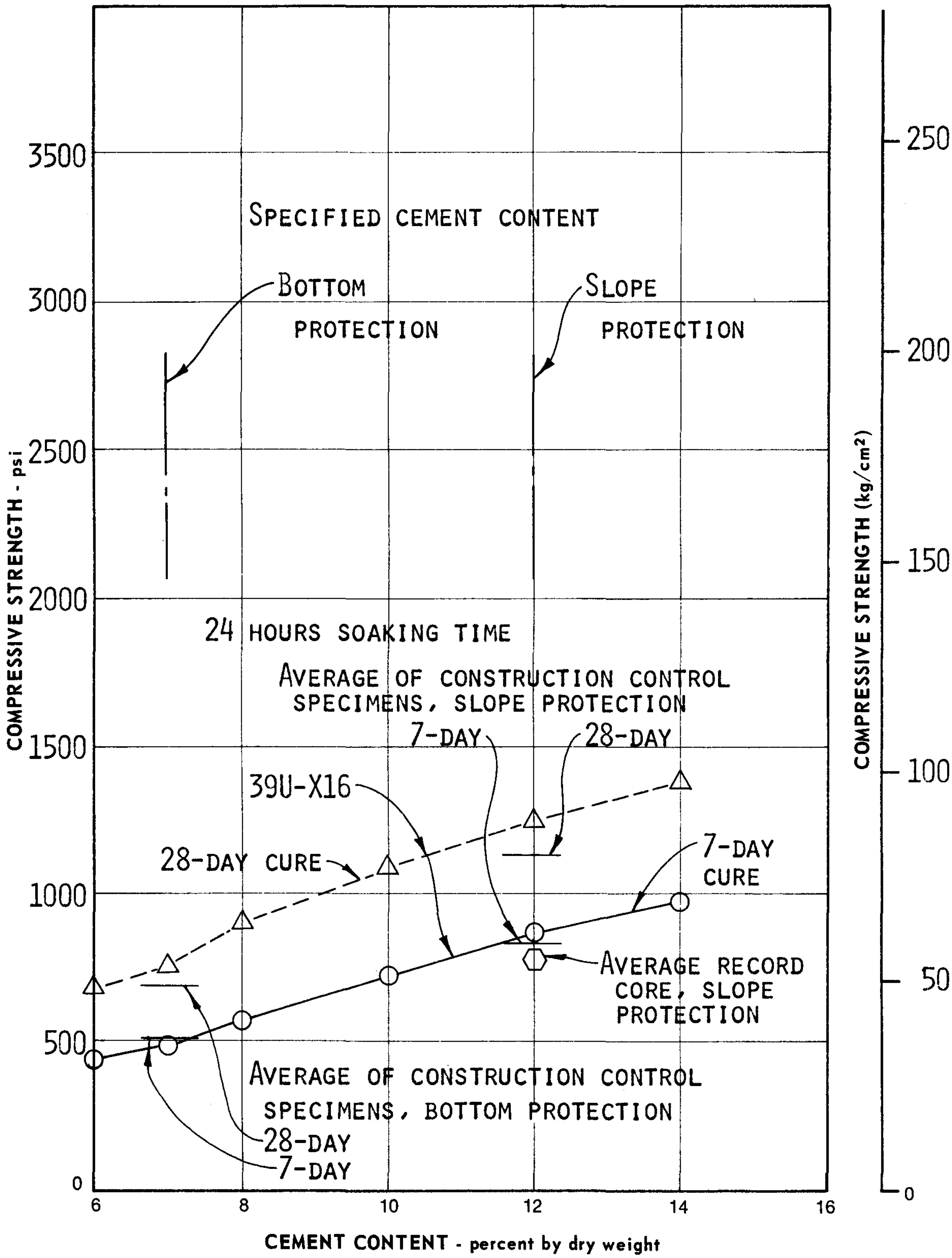
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2 x 2 in. (5 x 5 cm) CURE 28 days

CHENEY DAM

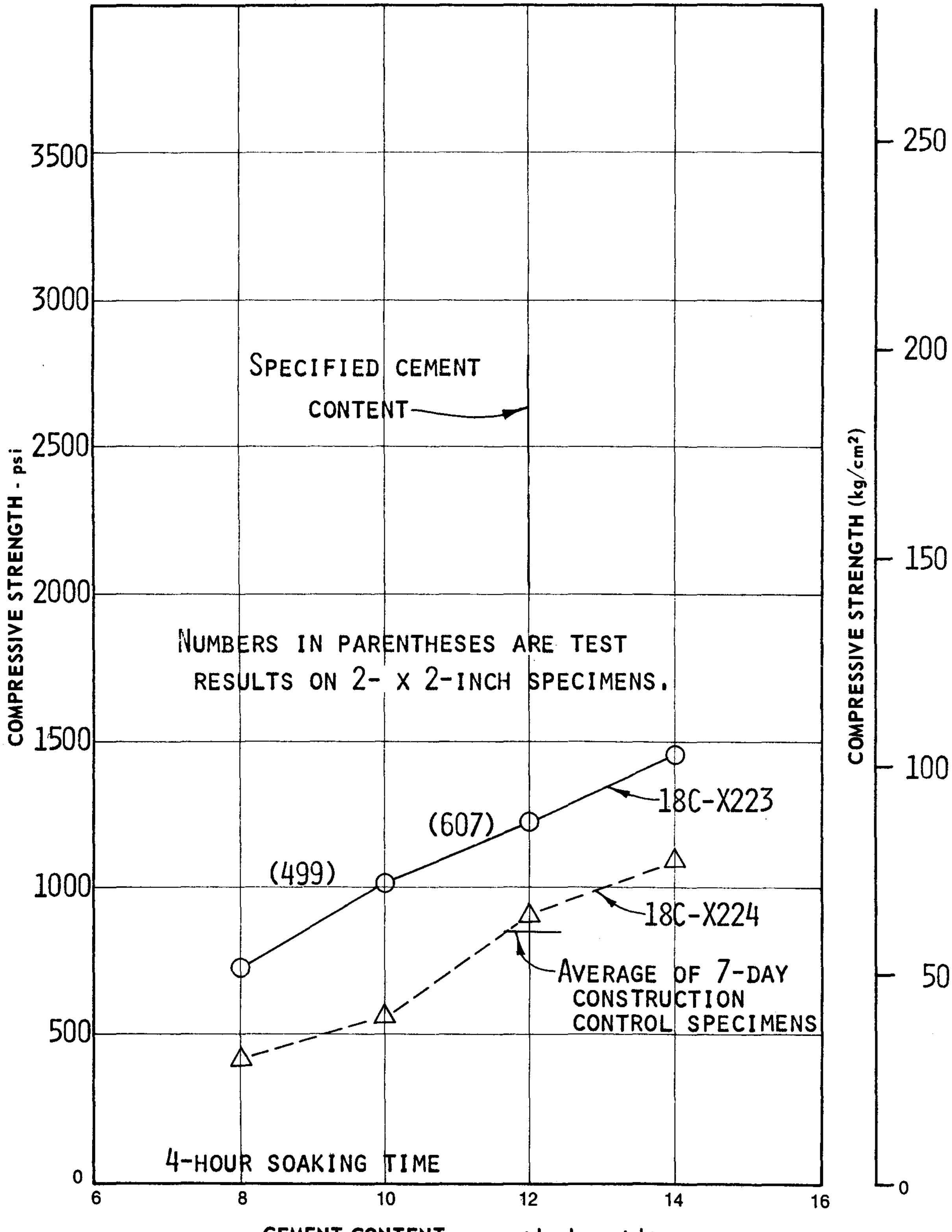
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE ________ days

LUBBOCK REGULATING RESERVOIR

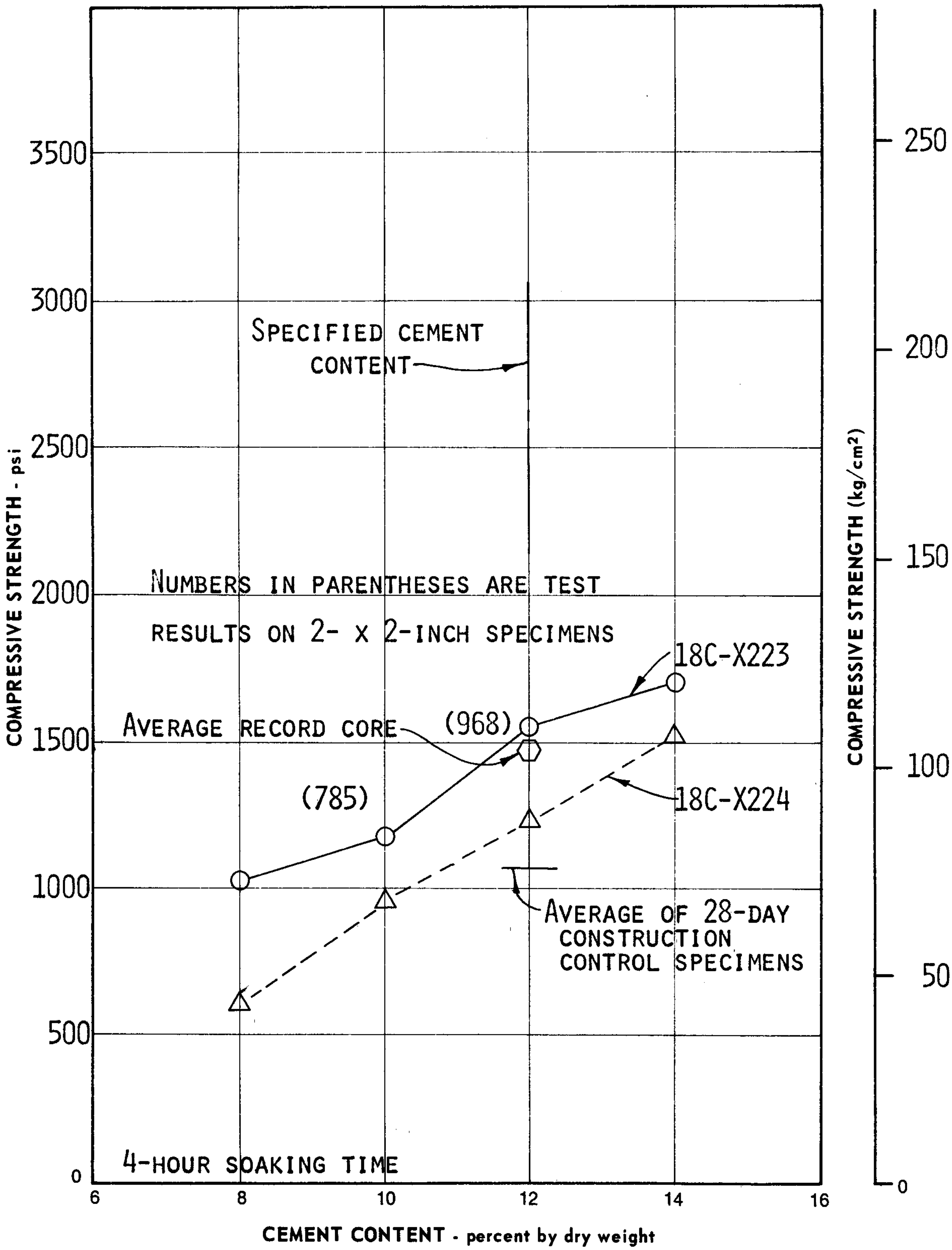
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x14.4 cm) CURE 7 days

GLEN ELDER DAM

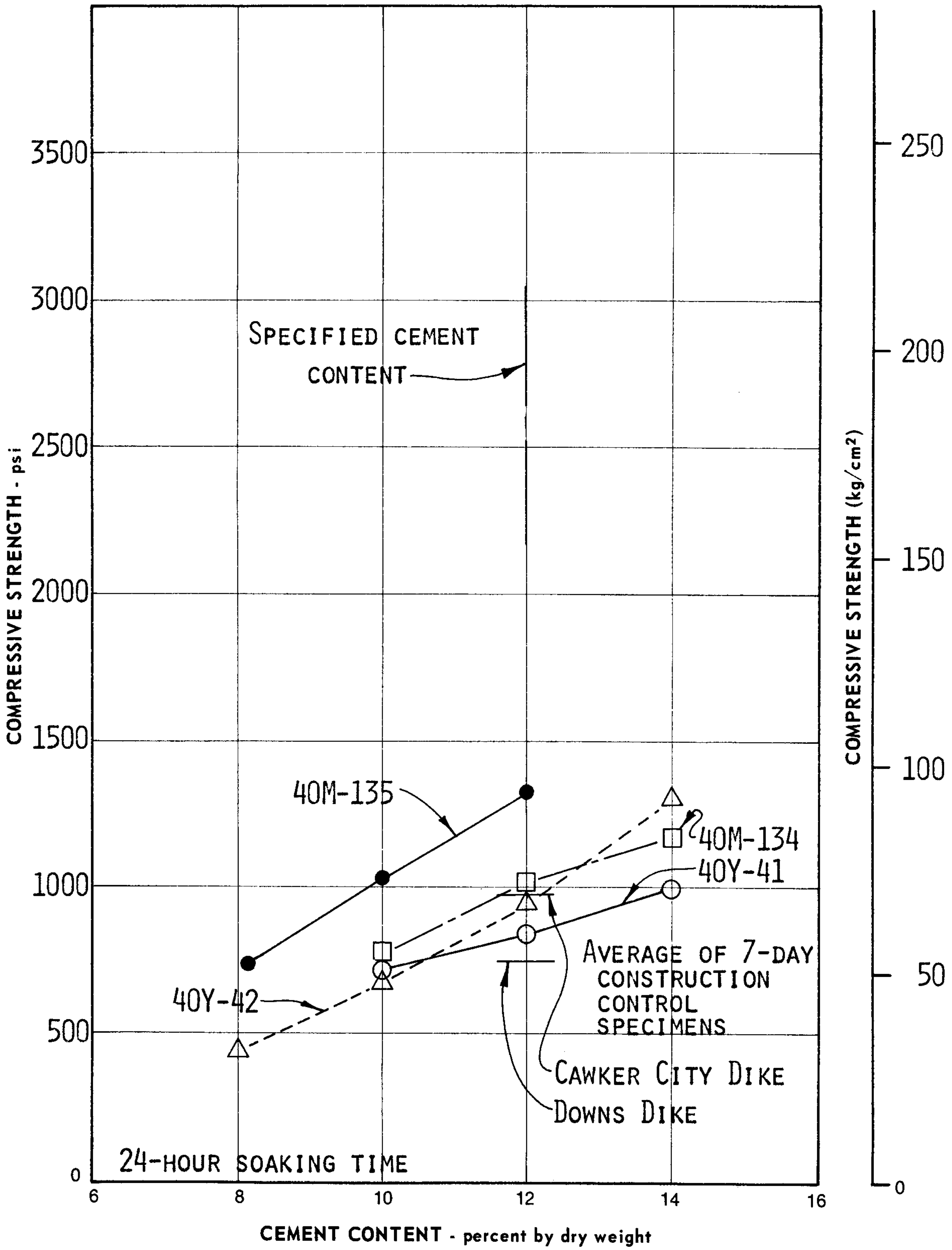
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2x14.4 cm) CURE 28 days

GLEN ELDER DAM

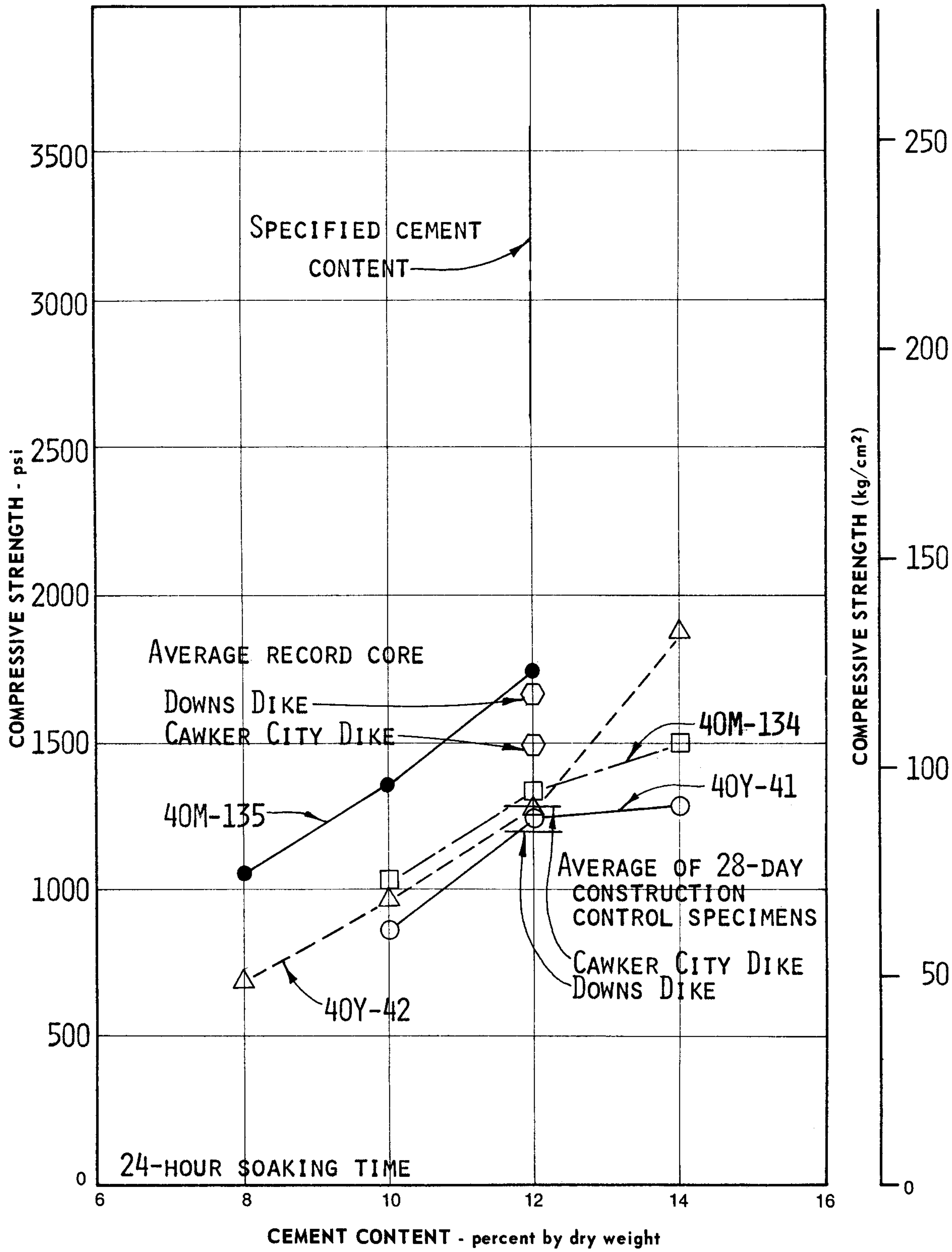
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2x 14.4 cm) CURE 7 days

DOWNS DIKE 40Y
CAWKER CITY DIKE 40M

SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS

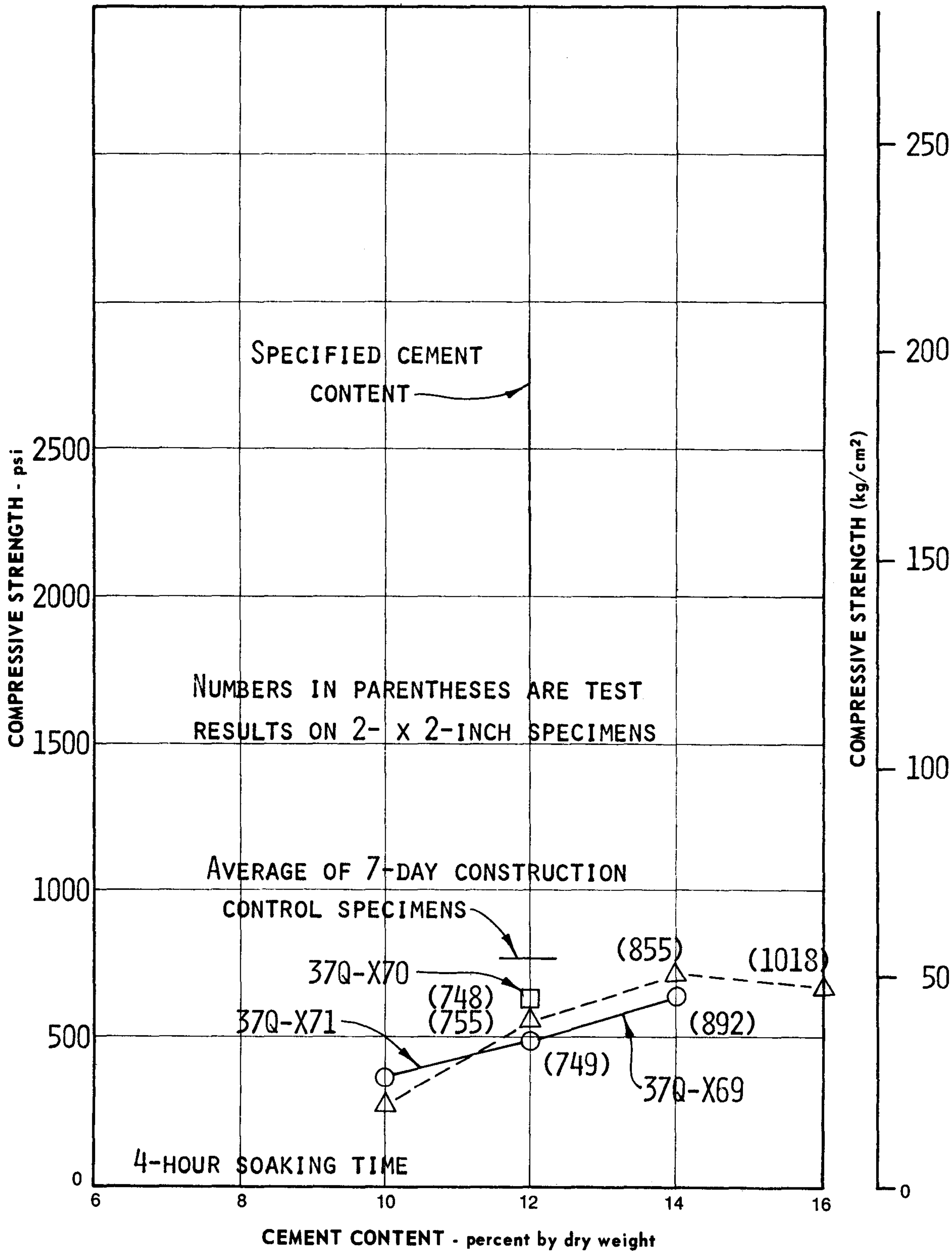


Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE 28 days

DOWNS DIKE 40M

CAWKER CITY DIKE 40M

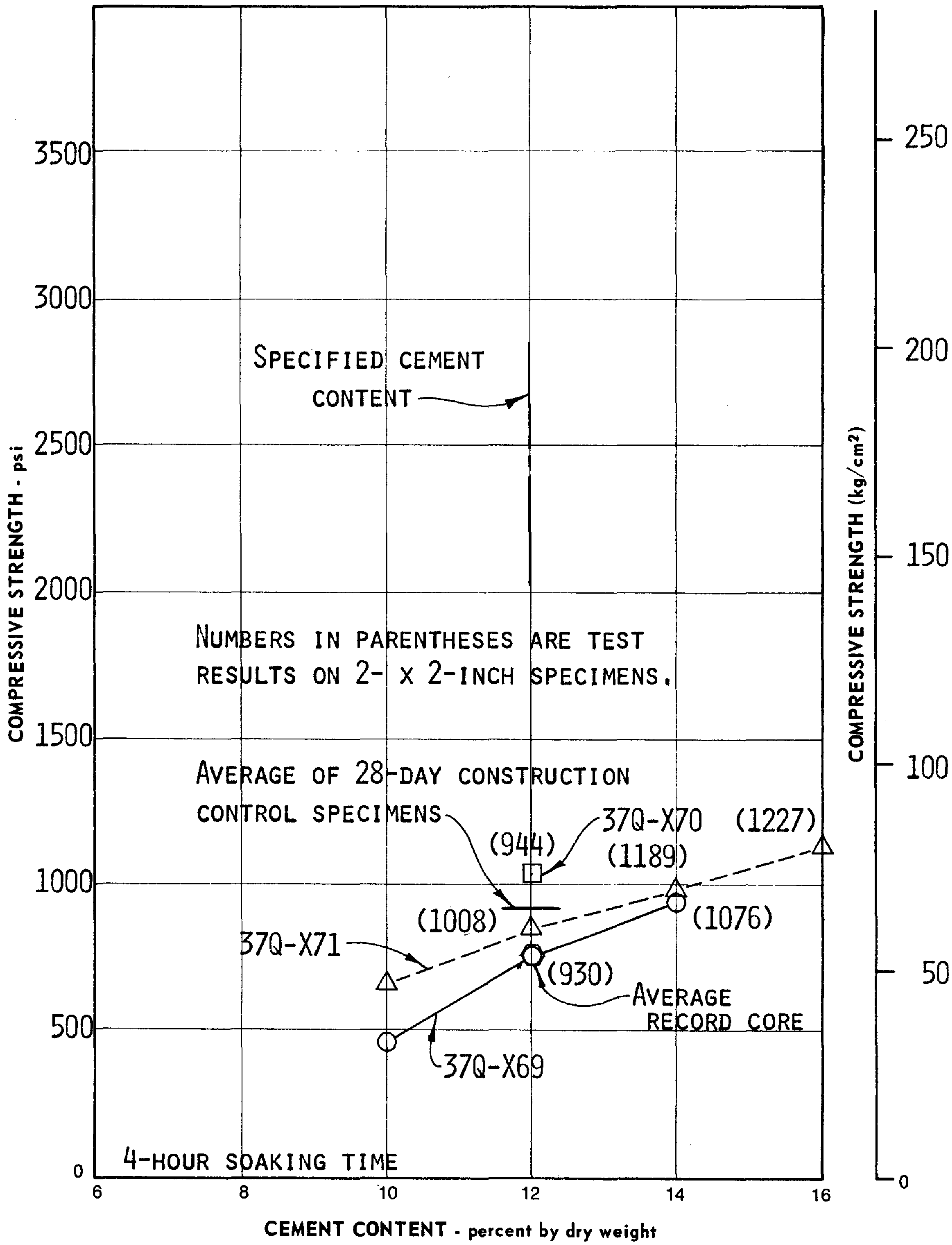
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE 7 days

STARVATION DAM

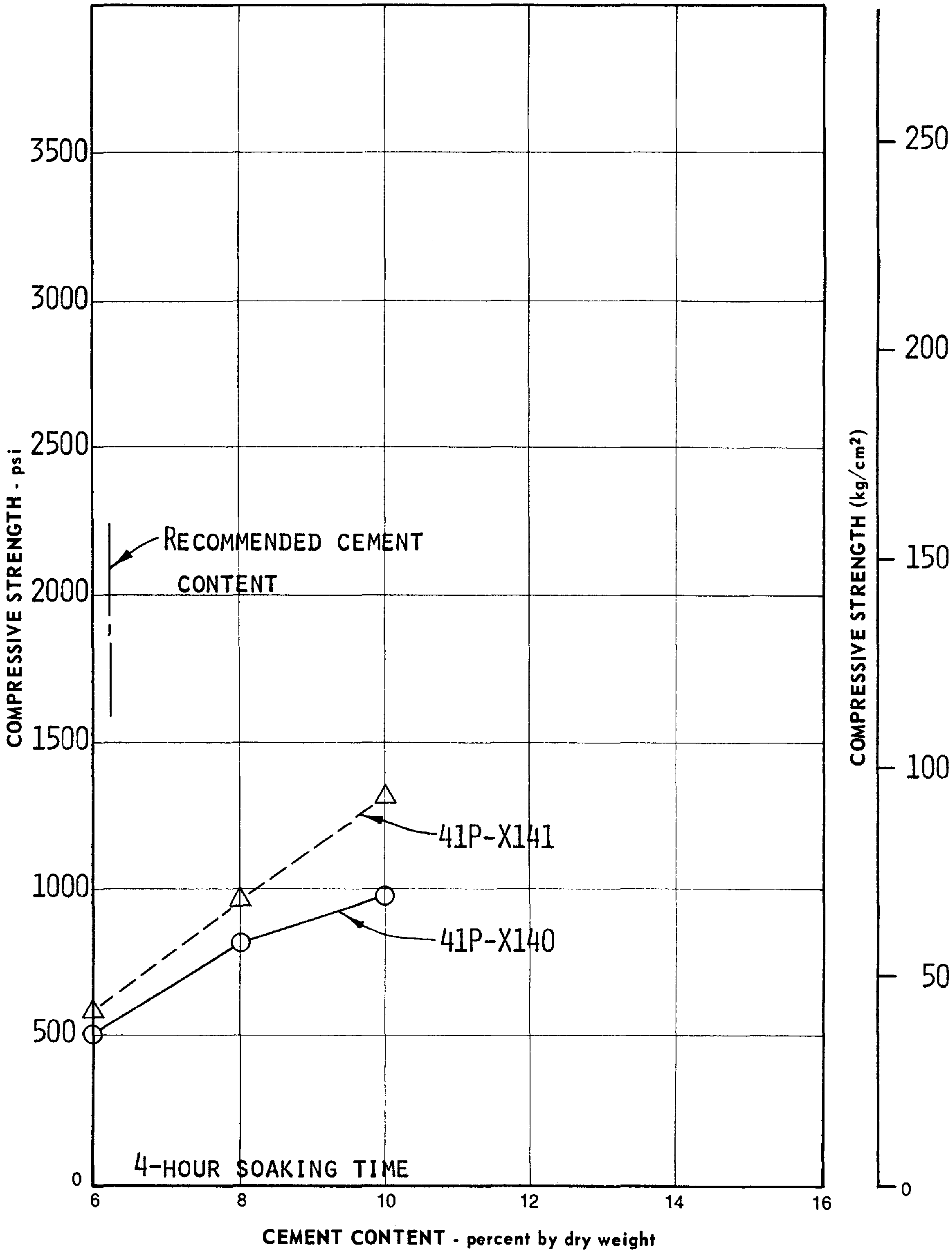
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE 28 days

STARVATION DAM

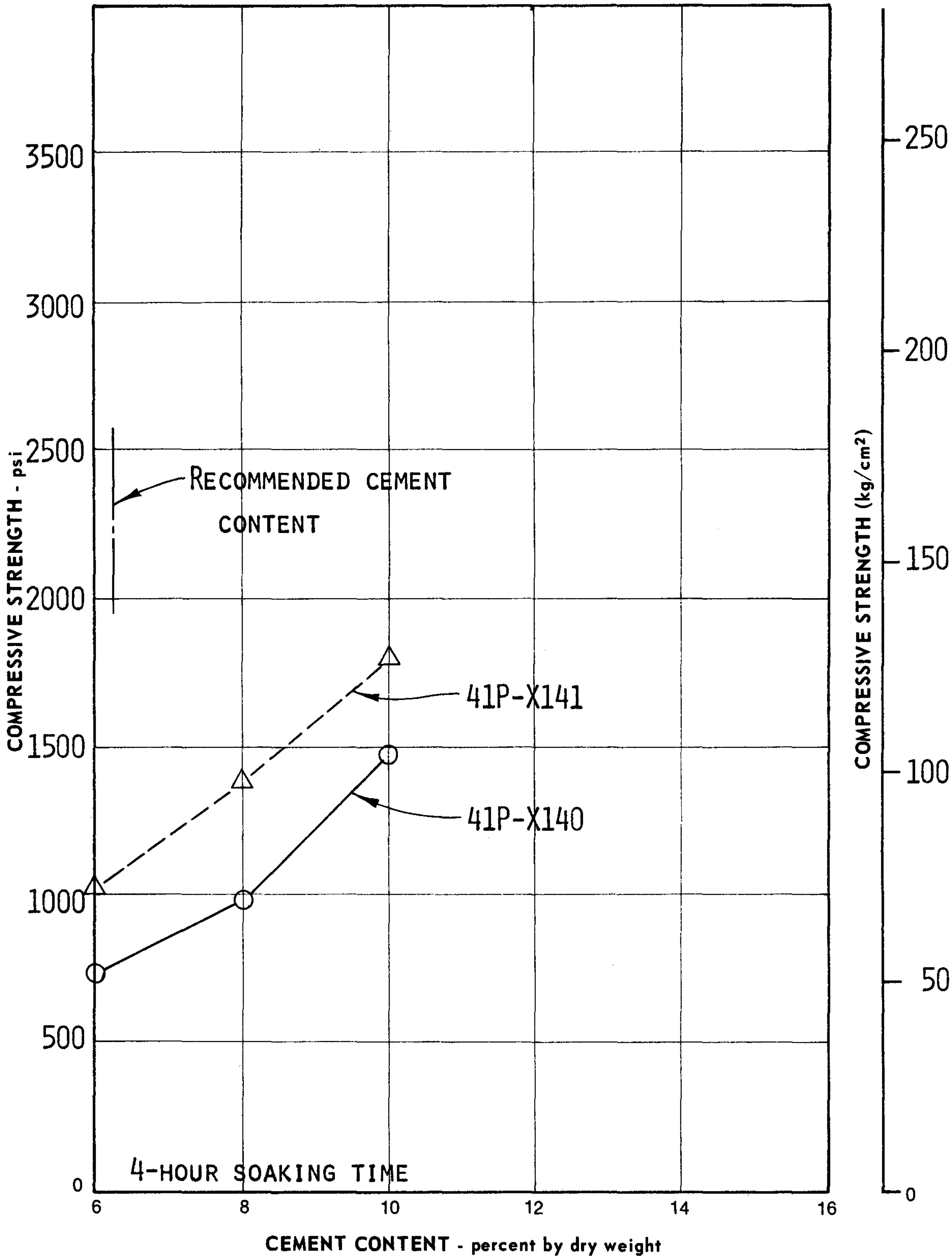
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE 7 days

LITTLE PANOCHE CREEK DETENTION DAM

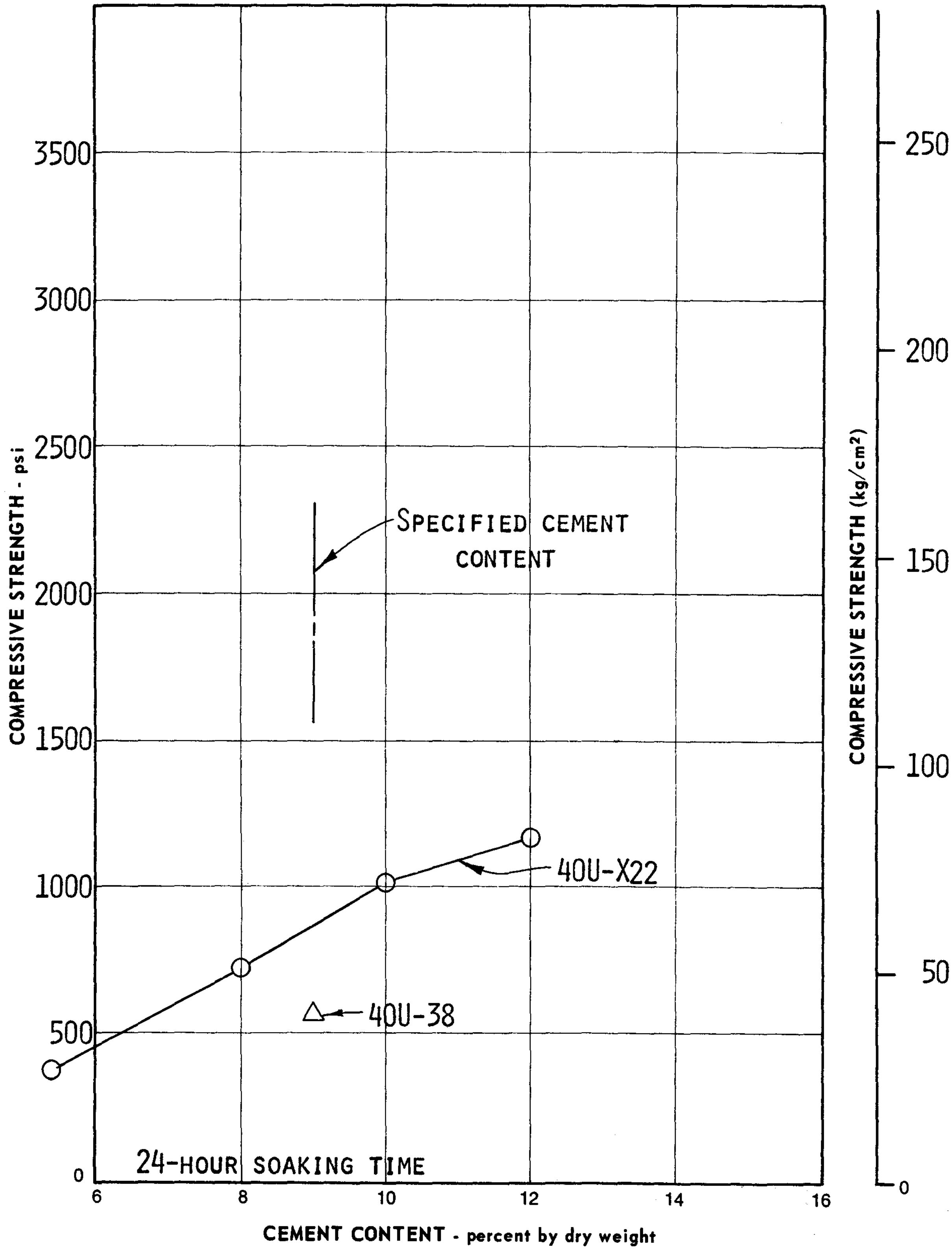
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm)   CURE 28 days

LITTLE PANOCHE CREEK DETENTION DAM

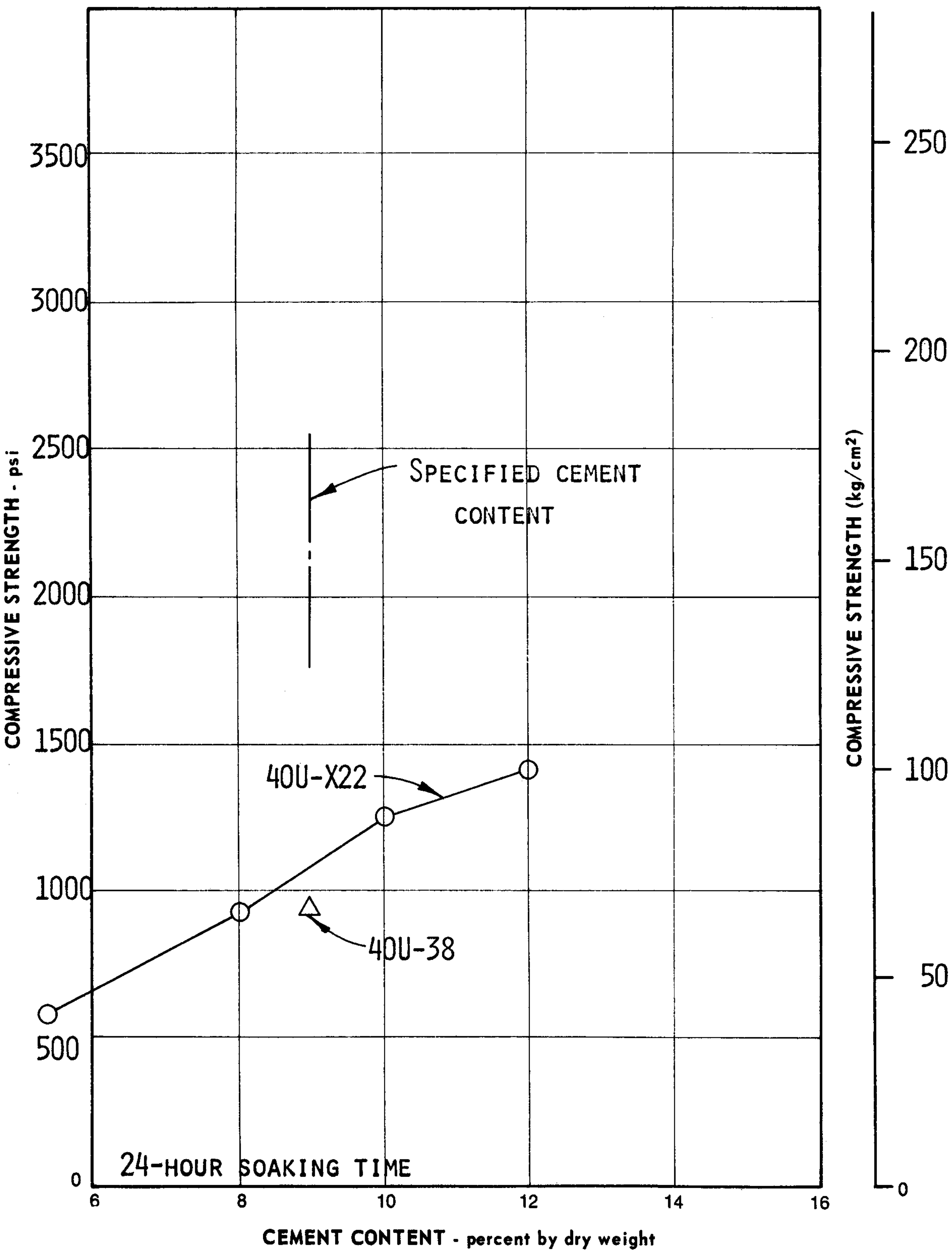
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE 7 days

RED BLUFF RESERVOIR

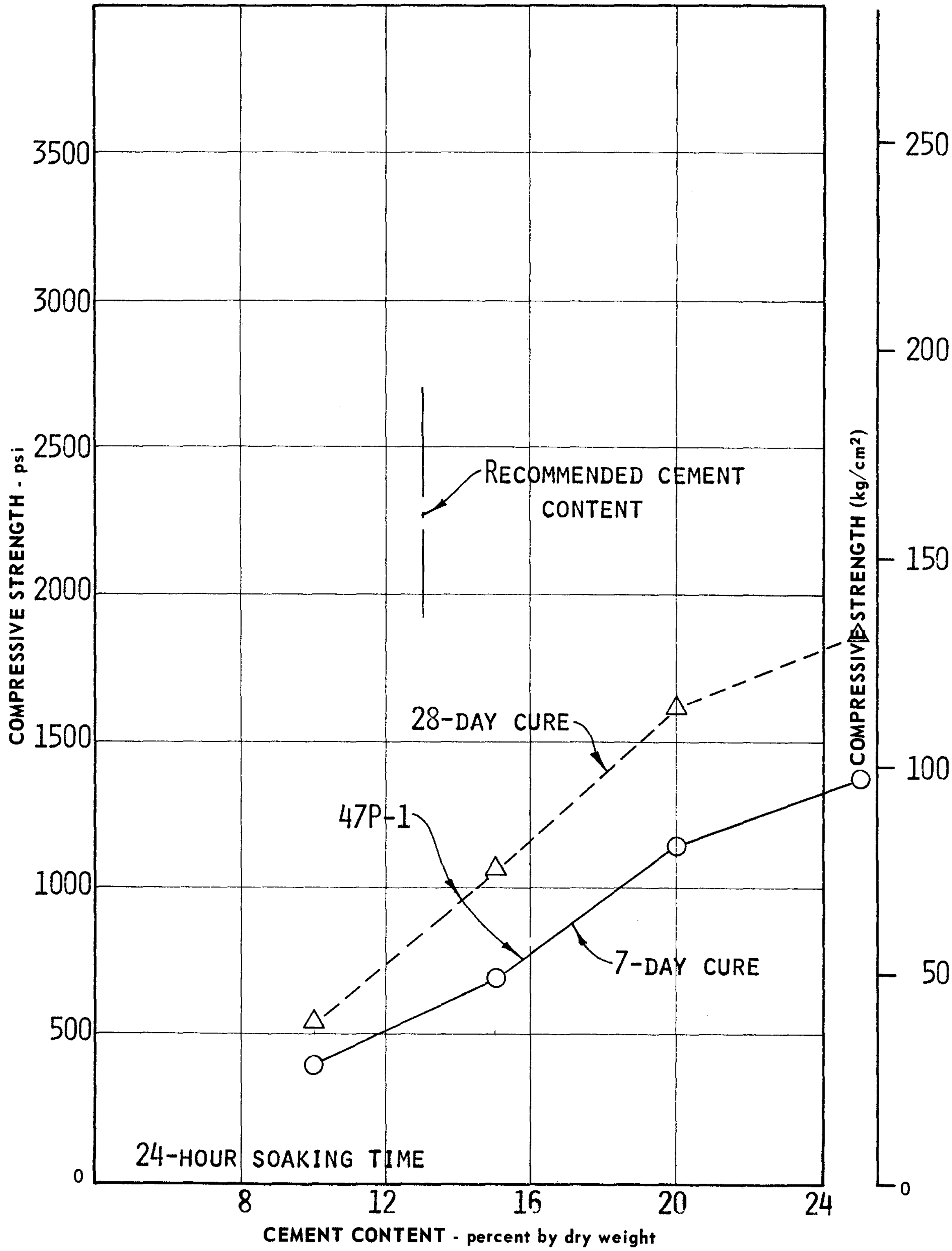
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2x 14.4 cm) CURE 28 days

RED BLUFF RESERVOIR

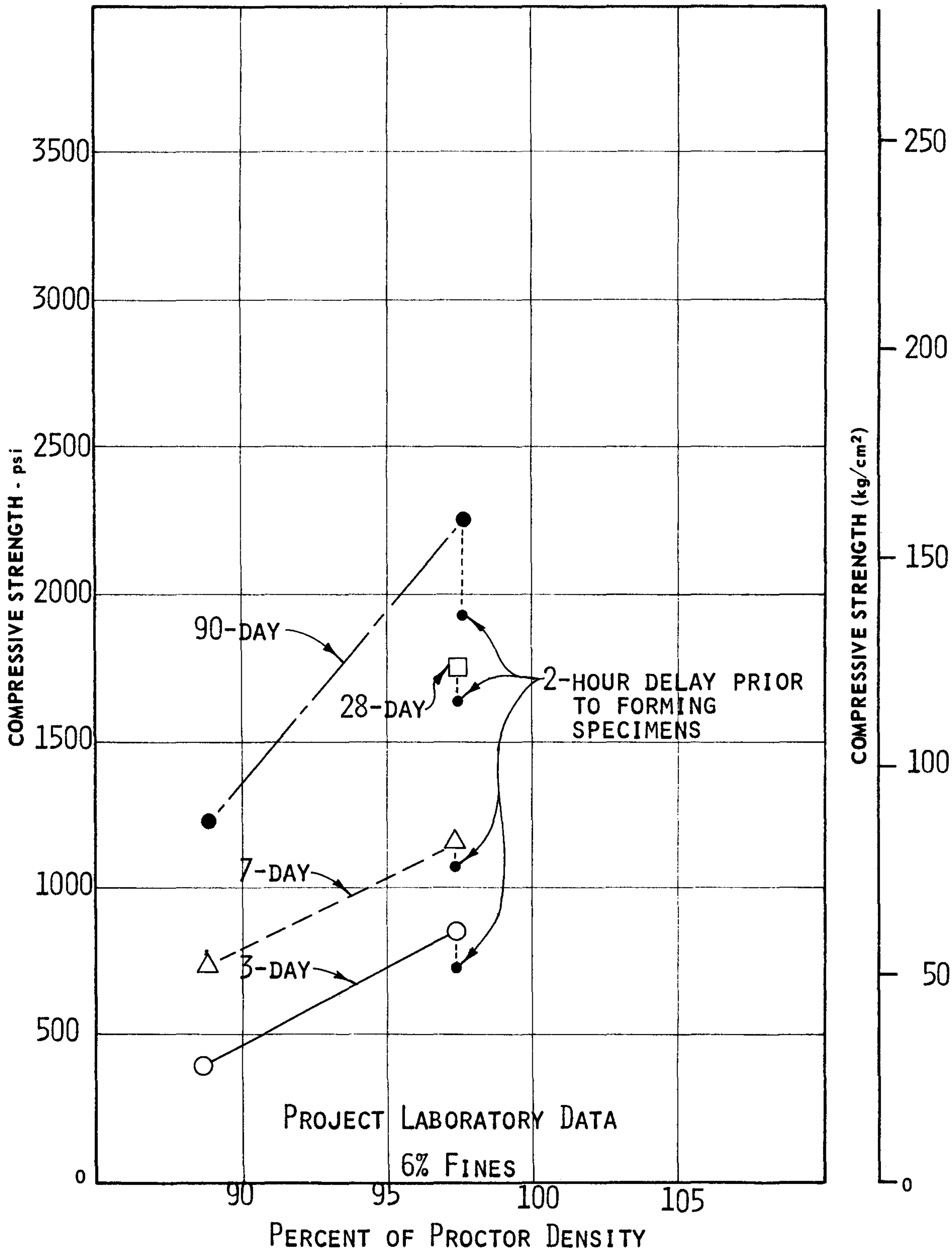
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE ____ days

CONCONULLY DAM

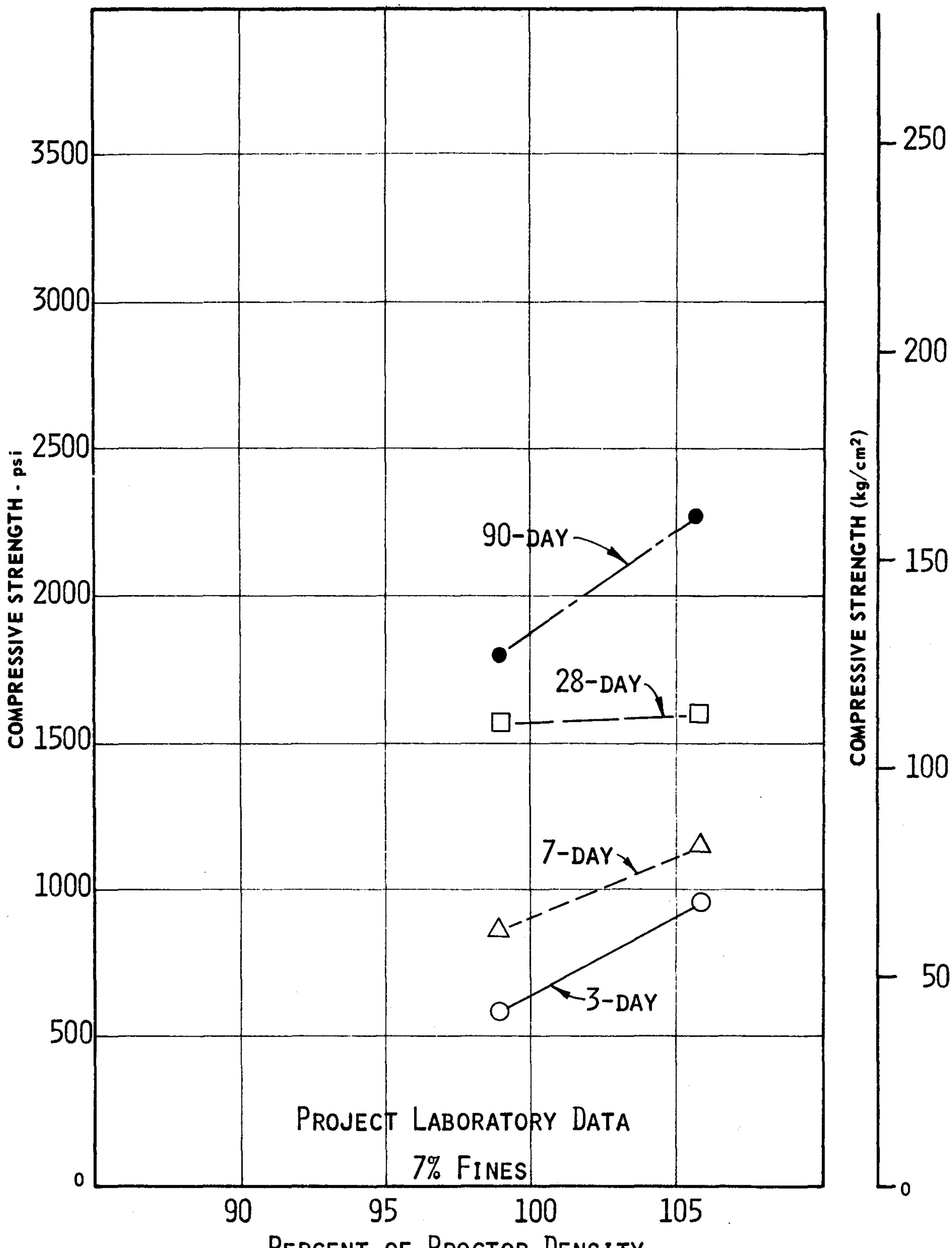
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE ________ days

CHENEY DAM

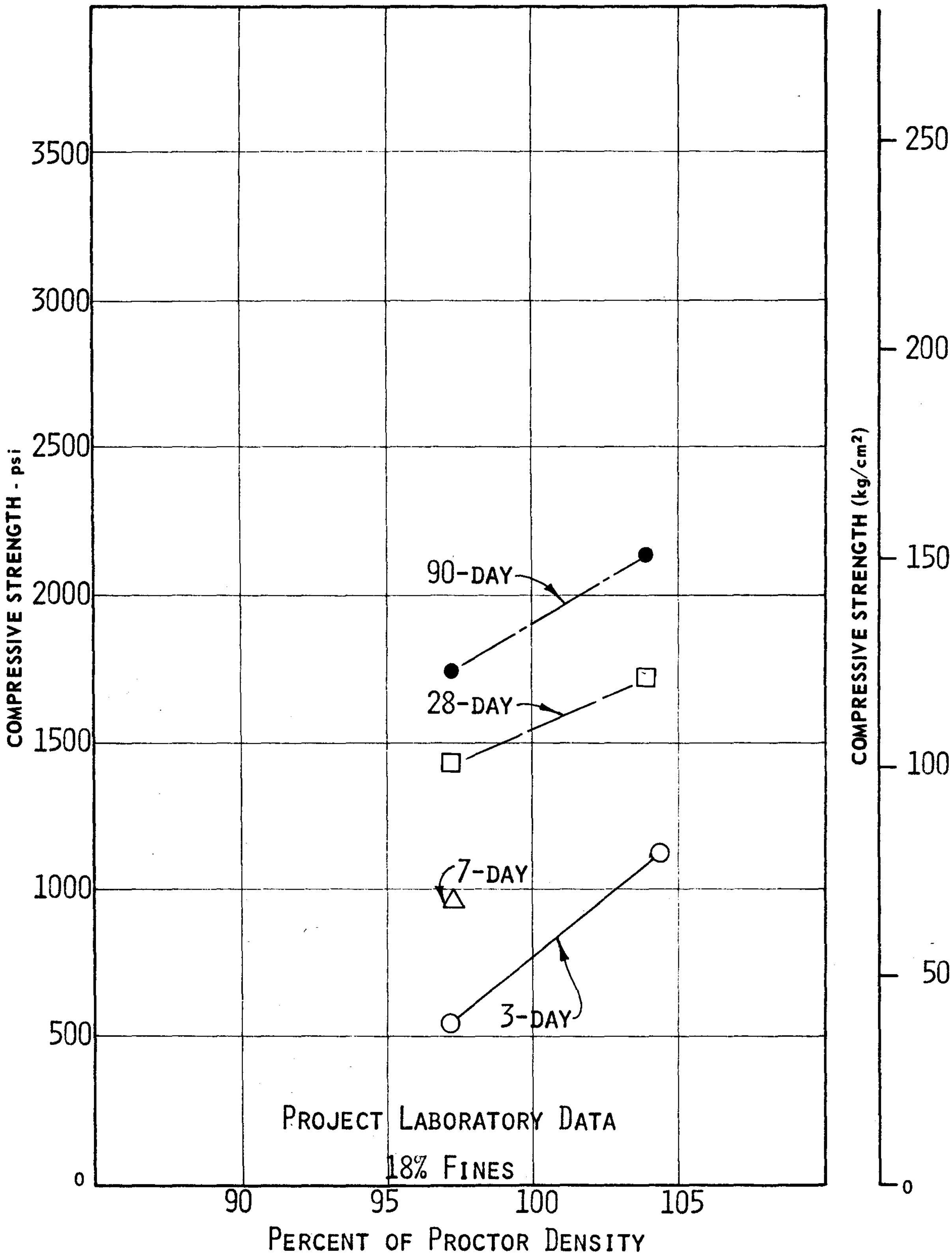
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE ____________ days

CHENEY DAM

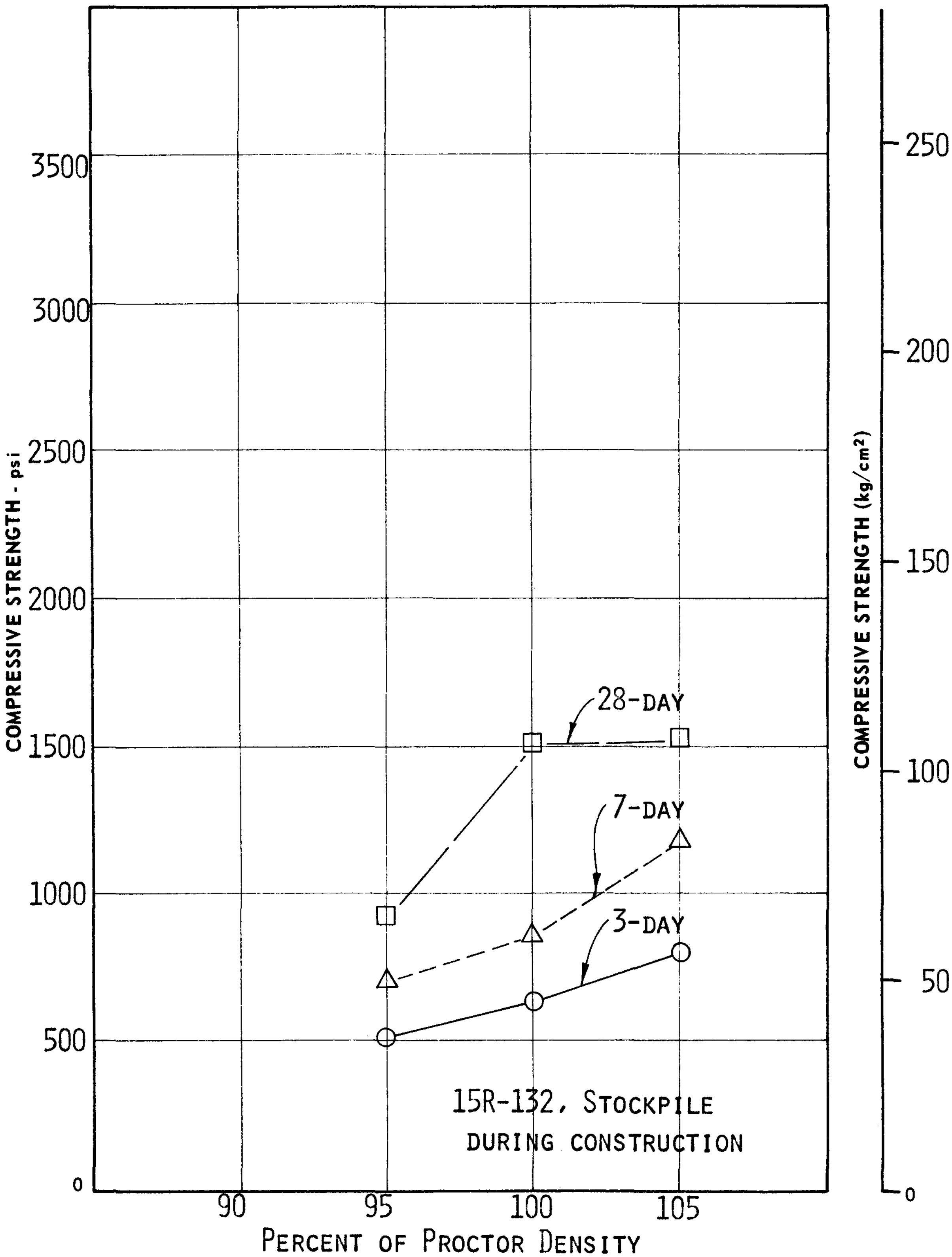
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE ________ days

CHENEY DAM

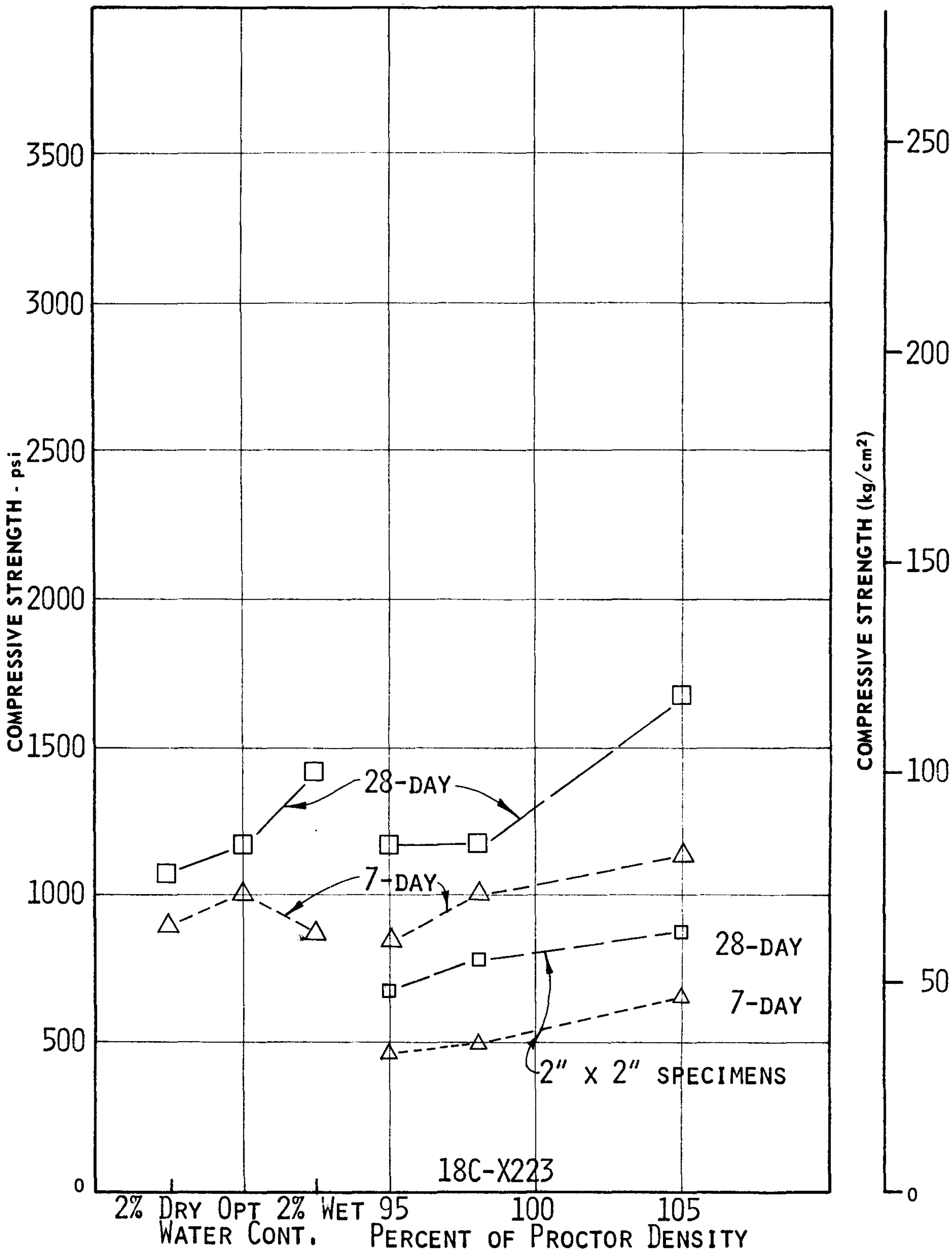
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm)   CURE ______ days

MERRITT DAM

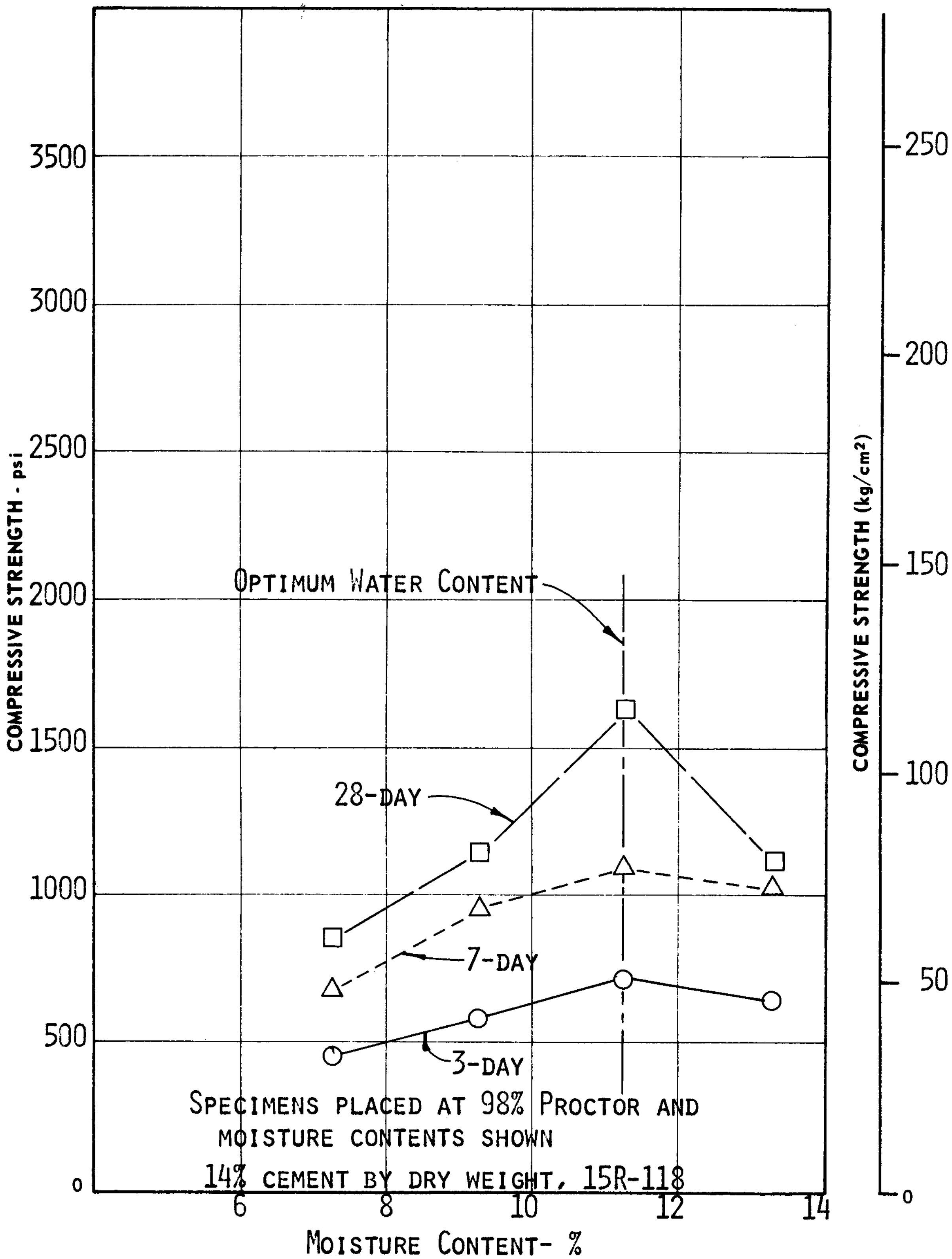
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.2 cm) CURE ______ days

GLEN ELDER DAM

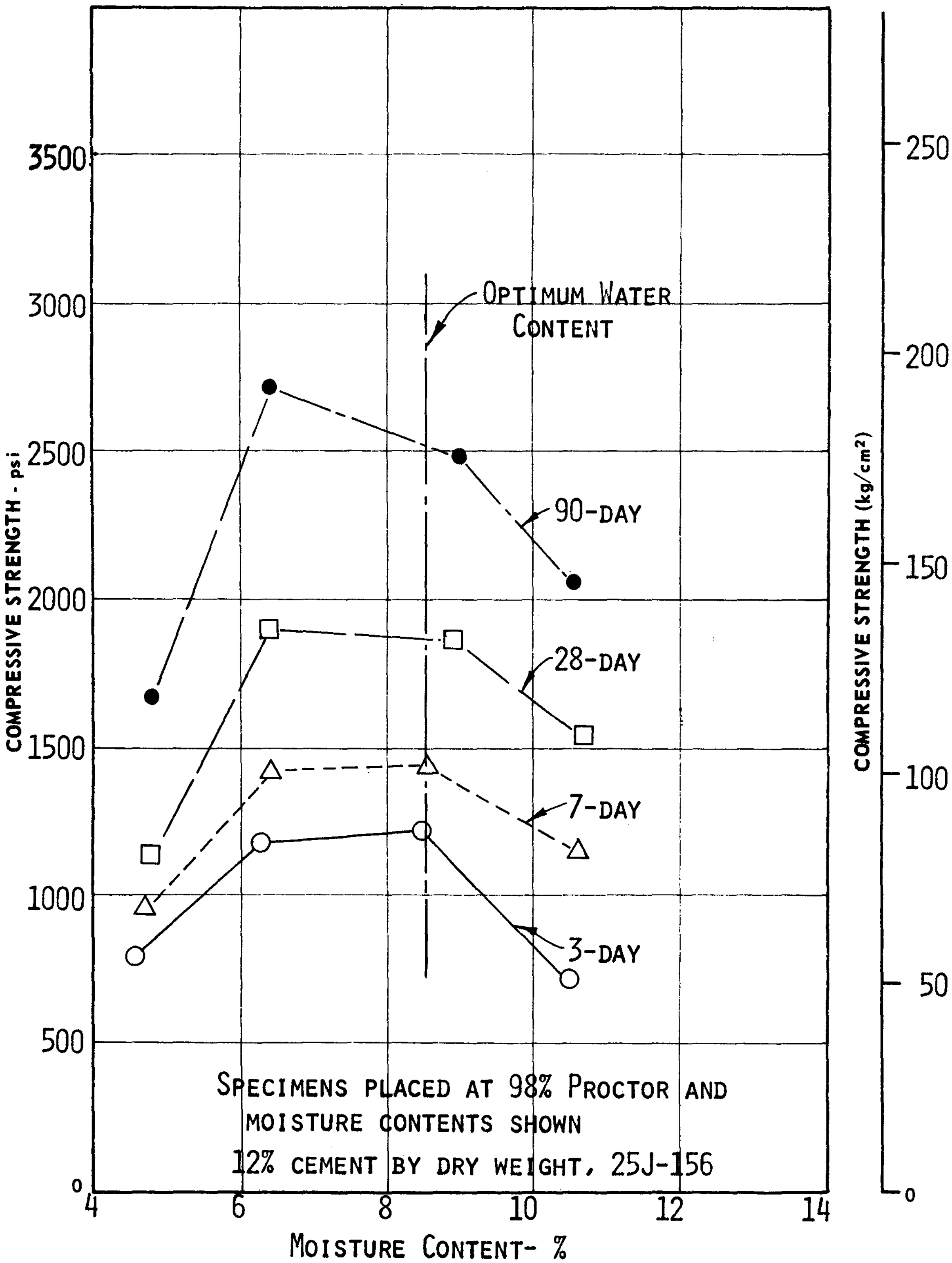
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE ________ days

MERRITT DAM

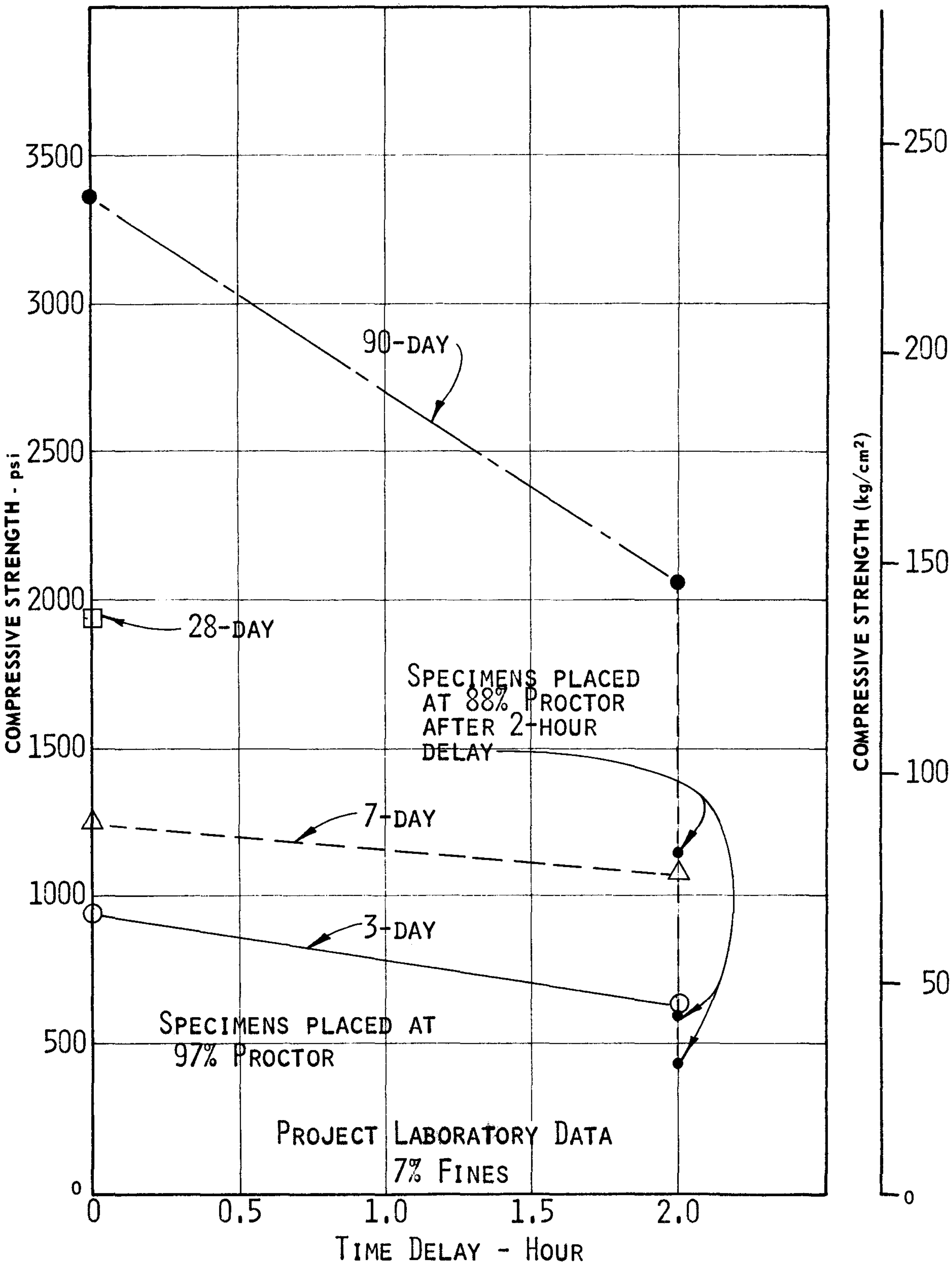
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE ______ days

CHENEY DAM

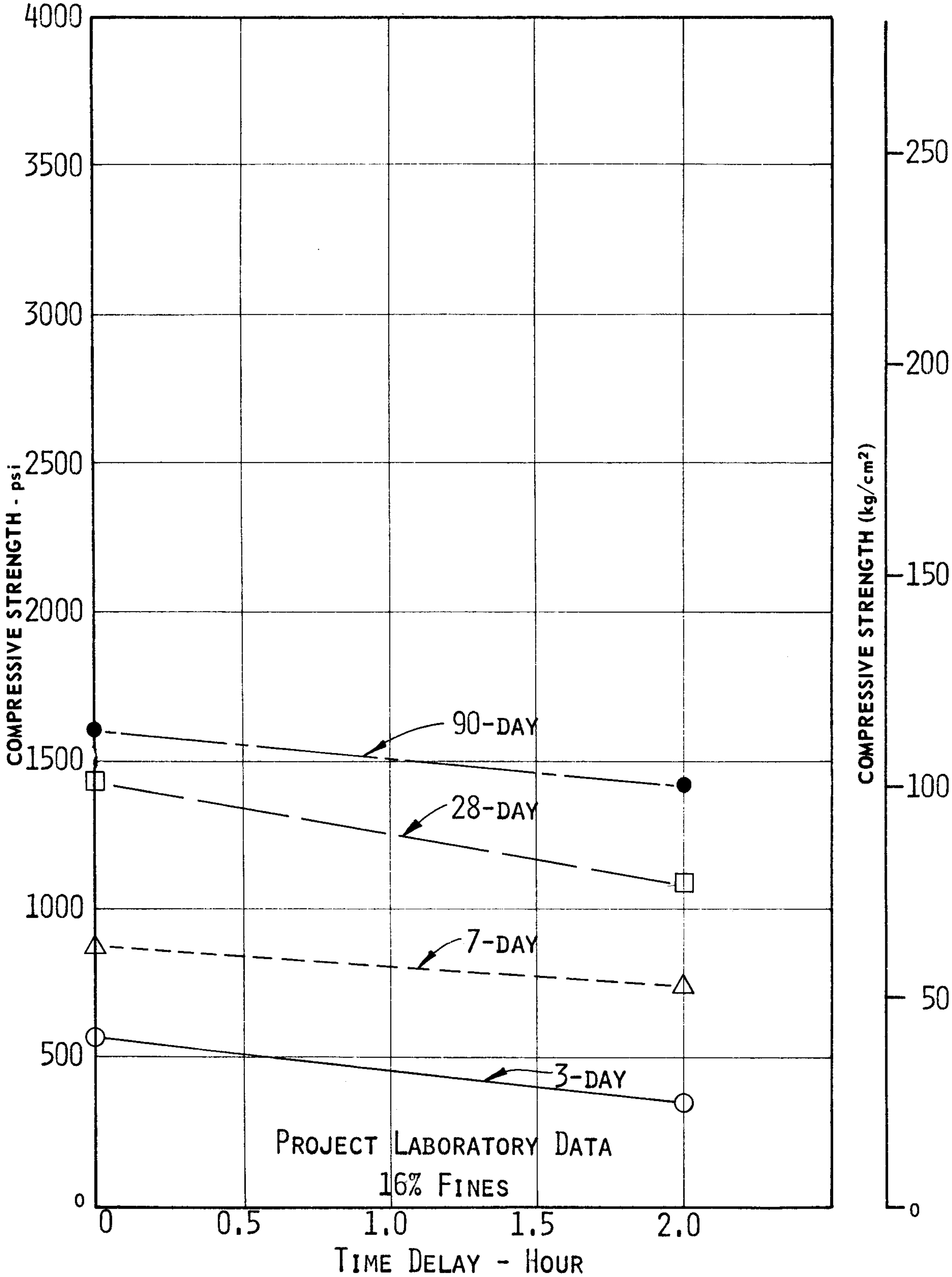
SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.4 cm) CURE ______ days

CHENEY DAM

SOIL-CEMENT COMPRESSIVE STRENGTH TEST RESULTS



Specimen Size 2.83 x 5.67 in. (7.2 x 14.2 cm) CURE ______ days

CHENEY DAM

CONSTRUCTION CONTROL RESULTS

Reporting period ending	Yardage placed (cu yd)	No. of accepted tests	Percent Proc dens (av)	Variation from opt (av-percent)	Av 7-day comp str (psi)	Av 28-day comp str (psi)
			Merritt Dam—DC-5462			
11-26-63	30,000	83	102.0	0.2 dry	1,410	1,884
12-26-63	16,000	34	102.5	0.5 dry	1,239	1,646
	Weighted averages		102.1	0.3 dry	1,360	1,815
			Cheney Dam—DC-5744			
4-30-64	14,200	45	99.8	0.2 dry	1,010	1,330
5-31-64	14,900	40	98.2	0.3 dry	1,090	1,340
6-30-64	23,400	57	98.6	0.3 dry	1,085	1,415
7-31-64	30,800	88	98.5	0.2 dry	1,175	1,395
8-31-64	33,900	84	98.7	0.4 dry	1,205	1,530
9-30-64	43,300	99	99.2	0.5 dry	1,360	1,705
10-31-64	18,000	23	98.6	0.4 dry	1,425	1,680
	Weighted averages		98.8	0.3 dry	1,199	1,497
			Lubbock Regulating Reservoir—DC-6000			
10-30-66	5,100	17	98.9	0.3 dry	954	1,249
11-25-66	21,000	53	100.2	0.3 dry	830	1,146
12-27-66	*21,000	45	100.2	0.7 dry	508	692
1-13-67	* 500	1	102.8	0.6 dry	669	589
1-13-67	5,400	11	101.1	0.2 dry	671	901
	Weighted averages		100.0	0.3 dry	834	1,134
			Downs Dike—DC-6405			
9-26-67	11,330	26	98.3	0.5 dry	827	1,368
10-26-67	38,405	85	99.0	0.1 dry	675	1,062
11-26-67	19,076	46	100.6	0.3 dry	836	1,355
	Weighted averages		99.4	0.2 dry	748	1,201

*Bottom lining with 7.0 percent cement, not included in the averages.

CONSTRUCTION CONTROL RESULTS—Continued

Reporting period ending	Yardage placed (cu yd)	No. of accepted tests	Percent Proc dens (av)	Variation from opt (av-percent)	Av 7-day comp str (psi)	Av 28-day comp str (psi)
		Glen Elder Dam—DC-6147				
9-26-67	16,779	57	100.6	0.6 dry	918	1,191
10-26-67	33,070	88	101.5	1.0 dry	975	1,245
11-26-67	21,276	33	102.7	1.3 dry	949	1,387
4-26-68	21,091	47	100.7	0.8 dry	1,029	1,165
5-26-68	31,702	71	100.8	0.4 dry	847	986
6-26-68	31,196	78	100.4	0.8 dry	648	878
7-26-68	28,142	72	100.1	0.4 dry	849	1,026
8-26-68	9,337	21	98.6	0.9 dry	650	874
	Weighted averages		100.8	0.7 dry	854	1,071
		Cawker City Dike—DC-6548				
8-26-68	9,886	22	100.4	0.5 dry	788	1,009
9-26-68	27,043	58	99.7	0.7 dry	897	1,200
10-26-68	30,512	66	100.2	0.3 dry	896	1,178
11-25-68	6,599	17	100.4	0.3 dry	987	1,367
4-26-69	7,803	22	99.6	0.2 dry	1,085	1,468
5-26-69	20,072	40	98.9	0.3 dry	1,113	1,531
6-11-69	8,678	21	98.8	0.2 dry	1,097	1,447
	Weighted averages		99.7	0.4 dry	962	1,287
		Merritt Dam Modifications—DC-6654				
11-14-68	13,440	38	100.0	1.0 dry	1,006	1,556
		Starvation Dam—DC-6488				
8-26-69	9,394	24	99.7	1.3 dry	895	1,060
9-26-69	70,272	143	97.8	1.7 dry	640	880
	Weighted averages		98.1	1.6 dry	769	905

MERRITT DAM
Soil-cement Record Cores

Core No.	Station	Elevation (top of hole)	Percent loss	Compressive strength (psi)
		Wet-dry Tests		
3	21+70.2	2884.9	0.3	
4	22+02.1	2876.1	0.2	
5	27+73.6	2919.0	0.2	
7	22+76.0	2931.7	0.7	
9	3+77.9	2880.5	0.3	
10	4+27.5	2902.0	0.5	
12	15+00.0	2950.0	1.7	
13	20+00.0	2950.0	0.7	
15	20+69.0	2899.3	0.6	
16	25+00.0	2926.5	1.6	
17	30+00.0	2951.6	1.4	
			Average = 0.7 percent	
		Freeze-thaw Tests		
3	21+702	2884.9	0.5	
4	22+02.1	2876.1	0.5	
6	26+14.2	2943.7	0.5	
11	5+89.1	2912.8	1.0	
9	3+77.9	2880.5	0.4	
13	20+00.0	2950.0	0.8	
15	20+69.0	2899.3	1.8	
16	25+00.0	2926.5	0.9	
			Average = 0.8 percent	
		Unconfined Compression Tests		
2-1	26+00.0	2927.8		1,333
2-2	26+00.0	2927.8		903
3	21+70.2	2884.9		967
5	27+73.6	2919.0		609
7	22+76.0	2931.7		1,236
10-1	4+27.5	2902.0		1,077
10-2	4+27.5	2902.0		805
12	15+00.0	2950.0		860
14-1	23+50.0	2948.0		1,046
14-2	23+50.0	2948.0		1,158
16-1	25+00.0	2926.5		1,006
16-2	25+00.0	2926.5		903
17	30+00.0	2951.6		1,057
				Average = 930 psi
Average of 28-day construction control strength specimens				1,815 psi

CHENEY DAM
Soil-cement Record Cores

Laboratory Sample No. 25J-	Hole No.	Station	Elevation (top of hole)	Percent loss
		Wet-dry Tests		
178B	3	109+70.1	1400	0.7
178C	3	109+70.1	1400	0.5
181B	6	101+62.9	1424.1	0.9
183F	8H	99+66.4	1425.5	0.8
184D	9H	94+88.2	1417.9	1.4
185C	10	88+31.5	1400	0.7
187D	12	88+52.5	1444.6	0.8
188B	13	80+94.6	1400	0.6
191C	16H	68+62.9	1415.3	0.7
192A	17	65+84.4	1400	0.5
198B	23	49+88.4	1400	1.4
200B	25	49+75.7	1444.5	0.5
209B	34H	116+00	1423	0.7
				Average = 0.8 percent
		Freeze-thaw Tests		
176B	1	119+89.9	1425.9	1.5
177C	2	119+90.1	1445.8	0.5
182B	7	101+67.7	1445.1	0.8
184E	9H	94+88.2	1417.9	0.9
185D	10	88+31.5	1400	0.8
187A	12	88+52.5	1444.6	1.2
187B	12	88+52.5	1444.6	0.8
188C	13	80+94.6	1400	0.6
188D	13	80+94.6	1400	0.8
191B	16H	68+62.9	1415.3	1.5
194B	19	65+86.8	1444.5	0.8
198D	23	49+88.4	1400	1.5
200D	25	49+75.7	1444.5	0.6
				Average = 0.9 percent

Note: H indicates area placed during high temperature (approximately 100° F).

CHENEY DAM
Soil-cement Record Cores

Laboratory Sample No. 25J-	Hole No.	Station	Elevation (top of hole)	Compressive strength (psi)
176D	1	119+89.9	1425.9	883
177D	2	119+90.1	1445.8	888
179C	4	110+01.1	1445.0	1,668
180B	5	101+61.1	1400	1,018
181B	6	101+62.9	1424.1	1,393
182C	7	101+67.7	1445.1	1,245
183C	8H	99+66.4	1425.5	733
183G	8H	99+66.4	1425.5	1,481
186C	11	88+53.1	1424.8	866
186E	11	88+53.1	1424.8	1,215
187E	12	88+52.5	1444.6	1,358
189B	14	80+92.1	1422.2	828
190C	15	80+90.6	1438.7	1,589
192C	17	65+84.4	1400	1,142
193D	18	65+86.1	1424.4	1,364
194C	19	65+86.8	1444.5	1,256
198C	23	49+88.4	1400	888
199D	24	49+81.0	1423.8	839
200C	25	49+75.7	1444.5	1,373
204D	29	41+04.4	1444.1	2,033
204E	29	41+04.4	1444.1	1,808
209C	34H	116+00	1423	1,599
209D	34H	116+00	1423	1,086
				Average = 1,241 psi
Average of 28-day construction control strength specimens				1,497 psi

Note: H indicates area placed during high temperature (approximately 100° F).

LUBBOCK REGULATING RESERVOIR
Soil-cement Record Cores

Laboratory Sample No. 39U-	Hole No.	Station	Elevation (top of hole)	Percent loss	Compressive strength (psi)
		Wet-dry Tests			
21	A-3	6+50	3270.6	1.5	
26	A-1-A	6+50	3278.6	1.4	
30	C-2	26+00	3275.6	0.8	
34	D-1	37+25	3281.6	2.8	
35	E-3	43+00	3272.6	0.8	
42	#2	25+50 (288 ft Rt)	3264.3	* 6.3	
			Average (slope) = 1.5		
		Freeze-thaw Tests			
24	A-3-A	6+50	3270.6	2.3	
28	B-2	14+30	3274.2	4.2	
30	C-2	26+00	3275.6	4.7	
33	D-2	37+25	3276.6	6.8	
38	E-3-A	42+90	3272.8	4.3	
39	E-2-A	42+90	3280.9	3.4	
41	#1	39+90 (152 ft Rt)	3264.3	*14.0	
			Average (slope) = 4.3		
		Unconfined Compression Tests			
24	A-3-A	6+50	3270.6		700
26	A-1-A	6+50	3278.6		736
29	C-3	26+00	3270.6		689
31	C-1	26+00	3280.6		670
33	D-2	37+25	3276.6		881
38	E-3-A	42+90	3272.8		864
39	E-2-A	42+90	3280.9		936
					Average = 782
Average of 52– 28-day construction control strength specimens					1,134

*Bottom lining

GLEN ELDER DAM
Soil-cement Record Cores

Laboratory Sample No. 18C-	Hole No.	Station	Elevation (top of hole)	Depth of core (feet)	Percent loss	Compressive strength (psi)
			Wet-dry Tests			
424	5	119+00	1438.9	0 to 0.6	0.9	
426	12	72+00	1454.7	2.0 to 2.6	0.5	
429	23	52+00	1484.6	1.3 to 1.7	0.5	
431	32	65+00	1500.2	0.9 to 1.5	0.6	
432	33	72+00	1470.1	1.6 to 2.2	0.8	
433	37	81+00	1485.1	2.2 to 2.8	0.9	
434	39	91+00	1470.1	0.0 to 0.7	0.4	
436	46	105+50	1447.8	0.5 to 1.2	0.3	
439	53	111+00	1500.8	0.5 to 1.1	0.8	
441	57	119+00	1485.3	0.0 to 0.6	1.3	
442	61	131+00	1448.0	1.4 to 1.9	0.4	
					Average = 0.7	
			Freeze-thaw Tests			
426	12	72+00	1454.7	0.0 to 0.6	1.8	
426	12	72+00	1454.7	2.8 to 3.4	1.2	
429	23	52+00	1484.6	0.6 to 1.3	0.9	
430	25	58+00	1455.2	1.1 to 1.8	0.5	
432	33	72+00	1470.1	1.1 to 1.6	0.8	
433	37	81+00	1485.1	1.5 to 2.2	0.5	
434	39	91+00	1470.1	0.7 to 1.2	0.6	
437	47	105+50	1470.6	1.8 to 2.4	0.7	
438	50	111+00	1448.6	0.0 to 0.6	0.6	
439	53	111+00	1500.8	1.1 to 1.8	0.9	
441	57	119+00	1485.3	1.1 to 1.7	0.5	
					Average = 0.8	
			Unconfined Compression Tests			
425	9	100+50	1441.0	1.1 to 1.6		1,471
Average of 176 tests performed on record cores by project laboratory						1,478
Average of 427— 28-day construction control strength specimens						1,071

Direct Shear Tests on Bonded Layers

DH-5, Station 119+00, Elevation 1438.9 (top of hole)

Depth of bonded layer (feet)	Shear strength (psi) At bonded layer	About 1-1/2 inches above bond	About 1-1/2 inches below bond
1.3	205	254	231
1.9	115	226	116

DOWNS DIKE
Soil-cement Record Cores

Laboratory Sample No. 40Y-	Hole No. DH-	Station	Elevation (top of hole)	Depth of core (feet)	Percent loss	Compressive strength (psi)
			Wet-dry Tests			
53	4	100+00	1472.3	1.2 to 1.7	1.2	
55	13	75+35	1498.0	1.1 to 1.6	0.6	
56	15	60+50	1482.7	1.9 to 2.4	0.6	
					Average = 0.8	
			Freeze-thaw Tests			
53	4	100+00	1472.3	1.7 to 2.2	1.9	
54	12	75+35	1483.3	1.7 to 2.2	1.5	
57	20	43+00	1498.0	1.2 to 1.7	1.3	
					Average = 1.6	
			Unconfined Compression Tests			
55	13	75+35	1498.0	2.3 to 2.8		1,752
56	15	60+50	1482.7	1.3 to 1.9		1,391
57	20	43+00	1498.0	0.1 to 0.6		1,049
						Average = 1,397
Average of 86 tests performed on record cores by project laboratory						1,665
Average of 110— 28-day construction control strength specimens						1,201

CAWKER CITY DIKE
Soil-cement Record Cores

Laboratory Sample No. 40M-	Hole No. DH-	Station	Elevation (top of hole)	Depth of core (feet)	Percent loss	Compressive strength (psi)
			Wet-dry Tests			
150	5	55+25	1471.1	1.2 to 1.7	0.6	
151	9	46+35	1486.0	1.1 to 1.6	0.5	
153	16	17+06	1486.0	0.6 to 1.1	0.6	
154	21	36+00	1500.1	0.8 to 1.4	0.3	
155	25	68+00	1470.0	1.9 to 2.4	0.4	
157	36	100+00	1485.0	1.9 to 2.5	0.3	
					Average = 0.5	
			Freeze-thaw Tests			
150	5	55+25	1471.1	0.6 to 1.2	0.9	
151	9	46+35	1486.0	0.5 to 1.1	0.8	
152	13	26+08	1464.0	1.7 to 2.2	0.6	
153	16	17+06	1486.0	1.1 to 1.6	0.7	
155	25	68+00	1470.0	0.4 to 0.9	0.6	
157	36	100+00	1485.0	0.5 to 1.2	0.8	
					Average = 0.7	
Average of 125 tests performed on record cores by project laboratory						1,493
Average of 244— 28-day construction control strength specimens						1,287

STARVATION DAM
Soil-cement Record Cores

Laboratory Sample No. 370-	Hole No. DH-	Station	Elevation (top of hole)	Depth of core (feet)	Percent loss	Compressive strength (psi)
			Wet-dry Tests			
115	1	15+00	5625	1.8 to 2.3	0.6	
119	5	15+00	5705	1.4 to 2.0	0.6	
121	7	20+00	5645	2.1 to 2.6	0.5	
122	8	20+00	5665	1.8 to 2.6	1.1	
123	9	20+00	5685	1.9 to 2.5	1.0	
126	12	25+00	5645	0.5 to 1.0	0.8	
127	13	25+00	5665	1.6 to 2.2	0.8	
129	15	25+00	5706	0.6 to 1.2	0.7	
					Average = 0.8	
			Freeze-thaw Tests			
117	3	15+00	5665	1.9 to 2.4	1.3	
118	4	15+00	5685	0.9 to 1.5	2.5	
119	5	15+00	5705	2.0 to 2.5	2.9	
120	6	20+00	5625	2.4 to 3.0	1.1	
121	7	20+00	5645	0.5 to 1.0	2.5	
126	12	25+00	5645	2.2 to 2.8	1.3	
127	13	25+00	5665	0.2 to 0.7	3.7	
129	15	25+00	5706	1.2 to 1.7	3.0	
					Average = 2.3	
			Unconfined Compression Tests			
116	2	15+00	5641	1.1 to 1.7		817
116	2	15+00	5641	1.7 to 2.3		774
117	3	15+00	5665	2.4 to 3.0		780
118	4	15+00	5685	2.1 to 2.7		777
121	7	20+00	5645	2.6 to 3.2		821
122	8	20+00	5665	1.3 to 1.8		761
123	9	20+00	5685	1.4 to 1.9		733
124	10	20+00	5705	1.5 to 2.1		814
124	10	20+00	5705	2.1 to 2.7		752
125	11	25+00	5625	0.9 to 1.5		782
125	11	25+00	5625	1.5 to 2.2		780
126	12	25+00	5645	1.0 to 1.6		726
127	13	25+00	5665	2.2 to 2.9		753
128	14	25+00	5685	1.4 to 1.9		590
128	14	25+00	5685	1.9 to 2.5		682
129	15	25+00	5706	2.2 to 2.9		741
						Average = 755
Average of 28-day construction control strength specimens						905

Direct Shear Tests on Bonded Layers

Laboratory Sample No. 370-	Hole No.	Station	Elevation (top of hole)	Depth of core (feet)	Shear strength (psi)		
					At bond	Above bond	Below bond
118	4	15+00	5685	1.5	94	226	201
120	6	20+00	5625	1.2	139	130	181
120	6	20+00	5625	1.8	137	181	242
122	8	20+00	5665	1.0	52	174	184
125	11	25+00	5625	3.4	*	264	211
127	13	25+00	5665	1.5	54	215	216

*Specimen broke on bonded layer before test.

7-1737
(5-63)
BUREAU OF RECLAMATION

SUMMARY OF FIELD AND LABORATORY TESTS OF COMPACTED SOIL-CEMENT

TOTAL CU. YDS. SOIL-CEMENT PLACED THIS PERIOD:

FACING 24,761 (Pay Estimate 10-26 to 11-26)

OTHER ---

PERIOD OF REPORT: FROM 10-31 TO 11-25

PROJECT ______

FEATURE ______

SPECIFICATIONS NO. ______

DATE OF REPORT: MONTH ______ YEAR ______

TEST NO. (NOTE 1) (1)	LOCATION OF TEST: STATION OFFSET COORDINATES ELEV. ETC. (2)	YARDAGE REPRESENTED BY FIELD DENSITY TEST (3)	SOURCE OF MATERIAL (BORROW AREA, REQUIRED EXCAVATION ETC.) (4)	SOIL CLASSIFICATION SYMBOL (UNIFIED CLASSIFICATION SYSTEM) DES E-3 △ (5)	MINUS NO. 200 % DES. E-6B △ (6)	SPECIFIC GRAVITY SOIL ONLY (-#4) (7)	METHOD OF COMPACTION SH-SHEEPS FOOT RT-RUBBER TIRED NO OF PASSES ROLLER NOS. (8)	TIME: PLANT MIXING COMPLETED * (9)	TIME: ** LAB COMPACTION COMPLETED (10)	TIME: FIELD COMPACTION COMPLETED ** (11)	TIME: FIELD DENSITY TEST STARTED** (12)	DRY DENSITY AND WATER CONTENT Controlled by Proctor Test (Designations E-11, E-24, E-25) △ — FILL DRY DENSITY: TOTAL MATERIAL (p.c.f.) (13)	FILL DRY DENSITY: MINUS NO.4 (p.c.f.) (14)	LABORATORY STANDARD MAXIMUM DENSITY (p.c.f.) (15)	RATIO FILL DRY DEN. TO LAB. MAX. DRY DEN. (D) % (16)	FILL WATER CONTENT: TOTAL MATERIAL % (17)	FILL WATER CONTENT: MINUS NO. 4 (w_f) % (18)	LABORATORY OPTIMUM (w_o) % (19)	VARIATION FROM OPTIMUM (w_o-w_f) % (20)	CEMENT CONTENT (%) — CALIBRATION FROM PLANT: BY WEIGHT FROM BATCH PLANT (21)	CALIBRATION FROM PLANT: BY VOLUME FROM BATCH PLANT (22)	BY WEIGHT FROM CHEM. TESTS (23)	COMPRESSIVE STRENGTH (lb per sq. in.) 3-DAY (24)	7-DAY (25)	28-DAY (26)	90-DAY (27)
11-16-A-1	30+79 40'R¢	320	3277.2	SM			6SH-4RT	10:30	40	30	35	116.0	116.0	118.8	97.6	12.2	12.3	12.3	opt.	12.0			-	-	-	-
11-16-A-2	13+36 40'R¢	310	3276.7	"			6SH-4RT	12:50	37	30	35	119.6	119.6	116.6	102.6	11.3	12.1	12.3	0.2D	12.0		11.8	690	958	1218	-
11-16-A-3	13+36 43'R¢	310	3276.7	"			6SH-4RT	12:50	37	30	36	116.9	116.9	116.6	100.3	11.3	11.8	12.3	0.5D	12.0			-	-	-	-
11-17-A-1	23+50 38'R¢	480	3279.0	"	30.6		6SH-4RT	10:50	55	45	50	117.7	117.7	115.0	102.3	13.7	13.7	12.7	1.0W	12.0		12.1	619	828	974	-
11-17-A-2	46+00 40'R¢	490	3277.6	"			6SH-4RT	13:00	30	25	30	115.8	115.8	115.8	100.0	13.1	13.0	13.0	opt.	12.0			631	797	-	1477
11-18-A-1	7+70 38'R¢	500	3278.7	"			6SH-4RT	15:00	45	30	40	115.6	115.6	116.5	99.2	13.3	13.5	13.0	0.5W	12.0		14.5	774	1096	1461	-
11-19-A-1	43+00 37'R¢	360	3278.6	"			6SH-4RT	10:45				119.5	119.5	116.9	102.2	12.7	12.6	12.5	0.1W	11.8			828	1015	1445	-
11-19-A-2	39+50 52'R¢	260	3271.6	"			6SH-4RT					122.4	122.4	116.9	104.7	12.0	10.4	11.8	1.4D	11.8		13.3	893	1136	1494	-
11-19-A-3	12+10 35'R¢	260	3279.2	"			6SH-4RT	14:30	15	25	25	115.7	115.7	117.7	98.3	10.6	9.9	12.2	2.3D	11.8			-	-	-	-
11-20-A-1	34+70 43'R¢	520	3275.6	"			6SH-4RT	10:50	65	45	45	117.4	117.4	116.3	100.9	13.5	12.8	13.4	0.6D	12.0		12.4	630	745	1055	-
11-20-A-2	34+00 39'R¢	520	3277.3	"			6SH-4RT	14:35	36	35	40	117.2	117.2	118.1	99.2	12.3	12.6	12.5	0.1W	12.0		12.3	651	893	-	1607
11-21-A-1	36+30 39'R¢	360	3277.8	"			6SH-4RT					116.3	116.3	116.7	99.7	13.2	13.0	13.0	opt.	12.0		12.7	732	776	1185	-
11-21-A-2	26+00 34'R¢	360	3279.7	"			6SH-4RT	11:53	97	57	62	112.6	112.6	111.4	101.1	15.6	15.4	14.6	0.8W	12.0			424	610	860	-
11-21-A-3	24+25 30'R¢	360	3281.3	"			6SH-4RT	14:00	35	30	32	117.9	117.9	114.8	102.7	13.2	12.9	13.8	0.9D	12.0			623	739	1055	-
11-22-A-1	3+50 45'R¢	400	3276.1	"			6SH-4RT	08:40	23	23	25	117.6	117.6	116.2	101.2	13.6	13.2	12.8	0.4W	12.0			594	724	-	-
11-22-A-2	35+40 28'R¢	400	3281.8	"			6SH-4RT	11:40	50	35	38	114.8	114.8	114.9	90.9	12.1	11.9	13.0	1.1D	12.0			498	644	812	-
11-22-A-3	38+75 28'R¢	400	3282.2	"			6SH-4RT	14:45	55	30	33	114.2	114.2	115.5	98.9	13.2	12.3	12.9	0.6D	12.0		13.1	643	763	-	-
11-23-A-1	0+50 33'R¢	580	3279.6	"	29.1	2.66	6SH-4RT	12:10	50	30	35	117.2	117.2	116.4	100.7	11.9	11.9	12.6	0.7D	12.0		12.6	713	797	1153	-
11-23-A-2	4+00 31'R¢	580	3280.6	"			6SH-4RT	14:30	40	30	35	118.4	118.4	115.7	102.3	11.7	11.8	12.4	0.6D	12.0			705	800	1177	-
11-25-A-1	25+00 25'R¢	650	3282.3	"			6SH-4RT	09:00	50	40	60	114.5	114.5	116.2	98.5	12.8	13.4	13.1	0.3W	12.0			562	623	795	-
11-25-A-2	27+00 24'R¢	650	3282.8	"			6SH-4RT	14:15	70	55	60	115.0	115.0	116.2	99.0	13.4	13.0	13.5	0.5D	12.0			636	808	974	-
				Totals for 53 tests								6209.6	6209.6	6197.3	5308.6	653.5	648.3	663.8	15.5			346.9	28,830			9885
				Average for 53 tests								117.2	117.2	116.9	100.2	12.3	12.2	12.5	0.3D			13.3	655	830	1146	1412

* Re-worked
** Check after re-working

NOTE 1: MINIMUM NUMBER OF FIELD DENSITY TESTS REQUIRED 1 Test for every 500 cu. yds or 2 tests per shift, whichever is greater.

NOTE 2: * Column 9 to denote time of day completed.
** Columns 10, 11 and 12 to denote total elapsed time from plant-mix completion time.
△ Refers to Designations - Earth Manual - 1st Edition

Note 3: Offset refers to distance from centerline of compacted embankment.

Note 4: All soil from Caprock Sand and Gravel Company pit.

(17) Water Content as spread
(18) Water Content after compaction
(20) Variation from optimum after compaction

Sheet ___ of ___

GPO 834383

7-1750 (3-71)
Bureau of Reclamation

CONVERSION FACTORS–BRITISH TO METRIC UNITS OF MEASUREMENT

The following conversion factors adopted by the Bureau of Reclamation are those published by the American Society for Testing and Materials (ASTM Metric Practice Guide, E 380-68) except that additional factors (*) commonly used in the Bureau have been added. Further discussion of definitions of quantities and units is given in the ASTM Metric Practice Guide.

The metric units and conversion factors adopted by the ASTM are based on the "International System of Units" (designated SI for Systeme International d'Unites), fixed by the International Committee for Weights and Measures; this system is also known as the Giorgi or MKSA (meter-kilogram (mass)-second-ampere) system. This system has been adopted by the International Organization for Standardization in ISO Recommendation R-31.

The metric technical unit of force is the kilogram-force; this is the force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 9.80665 m/sec/sec, the standard acceleration of free fall toward the earth's center for sea level at 45 deg latitude. The metric unit of force in SI units is the newton (N), which is defined as that force which, when applied to a body having a mass of 1 kg, gives it an acceleration of 1 m/sec/sec. These units must be distinguished from the (inconstant) local weight of a body having a mass of 1 kg, that is, the weight of a body is that force with which a body is attracted to the earth and is equal to the mass of a body multiplied by the acceleration due to gravity. However, because it is general practice to use "pound" rather than the technically correct term "pound-force," the term "kilogram" (or derived mass unit) has been used in this guide instead of "kilogram-force" in expressing the conversion factors for forces. The newton unit of force will find increasing use, and is essential in SI units.

Where approximate or nominal English units are used to express a value or range of values, the converted metric units in parentheses are also approximate or nominal. Where precise English units are used, the converted metric units are expressed as equally significant values.

Table I

QUANTITIES AND UNITS OF SPACE

Multiply	By	To obtain
LENGTH		
Mil	25.4 (exactly)	Micron
Inches	25.4 (exactly)	Millimeters
Inches	2.54 (exactly)*	Centimeters
Feet	30.48 (exactly)	Centimeters
Feet	0.3048 (exactly)*	Meters
Feet	0.0003048 (exactly)*	Kilometers
Yards	0.9144 (exactly)	Meters
Miles (statute)	1,609.344 (exactly)*	Meters
Miles	1.609344 (exactly)	Kilometers
AREA		
Square inches	6.4516 (exactly)	Square centimeters
Square feet	*929.03	Square centimeters
Square feet	0.092903	Square meters
Square yards	0.836127	Square meters
Acres	*0.40469	Hectares
Acres	*4,046.9	Square meters
Acres	*0.0040469	Square kilometers
Square miles	2.58999	Square kilometers
VOLUME		
Cubic inches	16.3871	Cubic centimeters
Cubic feet	0.0283168	Cubic meters
Cubic yards	0.764555	Cubic meters
CAPACITY		
Fluid ounces (U.S.)	29.5737	Cubic centimeters
Fluid ounces (U.S.)	29.5729	Milliliters
Liquid pints (U.S.)	0.473179	Cubic decimeters
Liquid pints (U.S.)	0.473166	Liters
Quarts (U.S.)	*946.358	Cubic centimeters
Quarts (U.S.)	*0.946331	Liters
Gallons (U.S.)	*3,785.43	Cubic centimeters
Gallons (U.S.)	3.78543	Cubic decimeters
Gallons (U.S.)	3.78533	Liters
Gallons (U.S.)	*0.00378543	Cubic meters
Gallons (U.K.)	4.54609	Cubic decimeters
Gallons (U.K.)	4.54596	Liters
Cubic feet	28.3160	Liters
Cubic yards	*764.55	Liters
Acre-feet	*1,233.5	Cubic meters
Acre-feet	*1,233,500	Liters

Table II

QUANTITIES AND UNITS OF MECHANICS

Multiply	By	To obtain
MASS		
Grains (1/7,000 lb)	64.79891 (exactly)	Milligrams
Troy ounces (480 grains)	31.1035	Grams
Ounces (avdp)	28.3495	Grams
Pounds (avdp)	0.45359237 (exactly)	Kilograms
Short tons (2,000 lb)	907.185	Kilograms
Short tons (2,000 lb)	0.907185	Metric tons
Long tons (2,240 lb)	1,016.05	Kilograms
FORCE/AREA		
Pounds per square inch	0.070307	Kilograms per square centimeter
Pounds per square inch	0.689476	Newtons per square centimeter
Pounds per square foot	4.88243	Kilograms per square meter
Pounds per square foot	47.8803	Newtons per square meter
MASS/VOLUME (DENSITY)		
Ounces per cubic inch	1.72999	Grams per cubic centimeter
Pounds per cubic foot	16.0185	Kilograms per cubic meter
Pounds per cubic foot	0.0160185	Grams per cubic centimeter
Tons (long) per cubic yard	1.32894	Grams per cubic centimeter
MASS/CAPACITY		
Ounces per gallon (U.S.)	7.4893	Grams per liter
Ounces per gallon (U.K.)	6.2362	Grams per liter
Pounds per gallon (U.S.)	119.829	Grams per liter
Pounds per gallon (U.K.)	99.779	Grams per liter
BENDING MOMENT OR TORQUE		
Inch-pounds	0.011521	Meter-kilograms
Inch-pounds	1.12985×10^6	Centimeter-dynes
Foot-pounds	0.138255	Meter-kilograms
Foot-pounds	1.35582×10^7	Centimeter-dynes
Foot-pounds per inch	5.4431	Centimeter-kilograms per centimeter
Ounce-inches	72.008	Gram-centimeters
VELOCITY		
Feet per second	30.48 (exactly)	Centimeters per second
Feet per second	0.3048 (exactly)*	Meters per second
Feet per year	*0.965873×10^{-6}	Centimeters per second
Miles per hour	1.609344 (exactly)	Kilometers per hour
Miles per hour	0.44704 (exactly)	Meters per second
ACCELERATION*		
Feet per second^2	*0.3048	Meters per second^2
FLOW		
Cubic feet per second (second-feet)	*0.028317	Cubic meters per second
Cubic feet per minute	0.4719	Liters per second
Gallons (U.S.) per minute	0.06309	Liters per second
FORCE*		
Pounds	*0.453592	Kilograms
Pounds	*4.4482	Newtons
Pounds	*4.4482×10^5	Dynes

Table II–Continued

Multiply	By	To obtain
WORK AND ENERGY*		
British thermal units (Btu)	*0.252	Kilogram calories
British thermal units (Btu)	1,055.06	Joules
Btu per pound	2.326 (exactly)	Joules per gram
Foot-pounds	*1.35582	Joules
POWER		
Horsepower	745.700	Watts
Btu per hour	0.293071	Watts
Foot-pounds per second	1.35582	Watts
HEAT TRANSFER		
Btu in./hr ft^2 degree F (k, thermal conductivity)	1.442	Milliwatts/cm degree C
Btu in./hr ft^2 degree F (k, thermal conductivity)	0.1240	Kg cal/hr m degree C
Btu ft/hr ft^2 degree F	*1.4880	Kg cal m/hr m^2 degree C
Btu/hr ft^2 degree F (C, thermal conductance)	0.568	Milliwatts/cm^2 degree C
Btu/hr ft^2 degree F (C, thermal conductance)	4.882	Kg cal/hr m^2 degree C
Degree F hr ft^2/Btu (R, thermal resistance)	1.761	Degree C cm^2/milliwatt
Btu/lb degree F (c, heat capacity)	4.1868	J/g degree C
Btu/lb degree F	*1.000	Cal/gram degree C
Ft^2/hr (thermal diffusivity)	0.2581	Cm^2/sec
Ft^2/hr (thermal diffusivity)	*0.09290	M^2/hr
WATER VAPOR TRANSMISSION		
Grains/hr ft^2 (water vapor) transmission)	16.7	Grams/24 hr m^2
Perms (permeance)	0.659	Metric perms
Perm-inches (permeability)	1.67	Metric perm-centimeters

Table III

OTHER QUANTITIES AND UNITS

Multiply	By	To obtain
Cubic feet per square foot per day (seepage)	*304.8	Liters per square meter per day
Pound-seconds per square foot (viscosity)	*4.8824	Kilogram second per square meter
Square feet per second (viscosity)	*0.092903	Square meters per second
Fahrenheit degrees (change)*	5/9 exactly	Celsius or Kelvin degrees (change)*
Volts per mil	0.03937	Kilovolts per millimeter
Lumens per square foot (foot-candles)	10.764	Lumens per square meter
Ohm-circular mils per foot	0.001662	Ohm-square millimeters per meter
Millicuries per cubic foot	*35.3147	Millicuries per cubic meter
Milliamps per square foot	*10.7639	Milliamps per square meter
Gallons per square yard	*4.527219	Liters per square meter
Pounds per inch	*0.17858	Kilograms per centimeter

GPO 839-978

ABSTRACT

A summary of Bureau of Reclamation experience with soil-cement slope protection is presented. Compacted soil-cement has been used as a riprap substitute on 7 major Bureau structures. Preconstruction testing, construction equipment and procedures, construction control testing for soil-cement, and performance of soil-cement facings are discussed. Successful performance of a soil-cement test section at Bonny Reservoir in eastern Colorado was used as the basis for the design of the facings, and durability and compressive strength test results limits established by the test section have been generally followed. Most soils used by the Bureau have been fine, silty sands; a summary of test results is presented. The soil-cement is: (1) mixed in a continuous flow mixing system, (2) placed, and (3) compacted in nearly horizontal lifts with a combination of sheepsfoot and pneumatic rolling. Erosion of uncompacted material at the edge of the lifts results in a stairstep pattern of the slope. Durability tests on record cores taken at most features show low weight losses. Performance of soil-cement facings in service has been generally satisfactory. More than normal breakage has occurred at a few locations on Cheney Dam in Kansas. Has 17 references.

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REC-ERC-71-20
DeGroot, G
SOIL—CEMENT SLOPE PROTECTION ON BUREAU OF RECLAMATION FEATURES
Bur Reclam Rep REC-ERC-71-20, Div Gen Res, May 1971. Bureau of Reclamation, Denver, 104 p, 51 fig, 6 tab, 17 ref, append

DESCRIPTORS—/ *soil cement/ *slope protection/ *erosion control/ sands/ gradation/ soil compaction/ freeze-thaw tests/ wetting and drying tests/ mixing/ compaction equipment/ performance tests/ records/ construction/ embankments/ bibliographies/ soil investigations/ construction control/ construction equipment/ tests/ *earth dams/ dam construction/ durability
IDENTIFIERS—/ Merritt Dam, Nebr/ Cheney Dam, Kans/ Lubbock Regulating Reservoir, Tex/ Glen Elder Dam, Kans/ Starvation Dam, Utah

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