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Joint Venture Team

July 9, 2020

Suzanne Suskind, PE
Acting Deputy Director and City Engineeer
San Francisco Department of Public Works
30 Van Ness Ave., 5th Floor
San Francisco, CA 94102

Subject: Response to Comments and Questions Memo, dated June 25, 2020, on the Mission Rock LCC TAP Review Report

Dear Suzanne:

The TAP Panel has reviewed the comments and questions, contained in your June 25, 2020 memo regarding the Mission Rock Lightweight Cellular Concrete Technical Advisory Panel Technical Review Report (Addendum) dated May 8, 2020. The panel is able to offer additional clarification and information based upon your comments and questions. We have also included several additional reference documents (please see Attachments 1-4).

We hope this resolves your remaining questions. Please let us know if we can be of further assistance.

Sincerely,

MISSION ROCK LCC TAP PANEL

Steven Bartlett, Ph.D, P.E.

Stan Peters, P.E.

Arul Arulmoli, Ph.D., G.E.

Attachments: Attachment 1-Permeability Testing Procedure with Modification for LCC
Attachment 2-Natural Saturation Density Testing Procedure with Modification
Attachment 3-LCC Mix Designs
Attachment 4-Materials Properties Model of Aging Concrete (DSO-05-05)

TAP Responses to Comments and Questions

TESTING METHODOLOGY, STANDARDS, SAMPLING AND PROJECT INFORMATION

Exclusive Reliance on Unmodified ASTM Procedures

With regard to the desire to rely exclusively on unmodified ASTM tests, there are two ASTM standards regarding cellular concrete that can be used without modifications. C495 covers compressive strength (originally created in 1962), and C796 (originally created in 1974) covers testing to qualify foaming agents, and includes wet-density (as-cast), dry density, absorption, air content, etc. Only wet density and compressive strength testing with these standards are relevant to the project. Regarding Public Works' desire to have testing performed by an independent certified (accredited) testing laboratory, it should realize none technically exist for cellular concrete, as shown by the following excerpt from the CCRL website (http://www.ccrl.us/). The actual ASTM testing standards for concrete do not include C495 or C796.

CCRL
Cement and Concrete Reference Laboratory

Concrete

The program is based on Standard Practice for Laboratories Testing Concrete and Concrete Aggregates for Use in Construction and Criteria for Laboratory Evaluation (ASTM C1077). This standard provides requirements for test methods, laboratory facilities, quality systems, personnel, and organizational structure. These requirements together with the apparatus and procedures prescribed in the referenced test methods provide the scope of the concrete laboratory inspection.

Facilities checked include the laboratory work area and curing facilities (moist room, tanks, or both). Apparatus includes compression testing machines, cylinder molds, capping plates, capping material, slump cones, unit weight measures, platform scales, volumetric air meters, and pressure air meters. General use items such as tamping rods, mallets, and strike-off plates and bars are also checked. Each item is evaluated to determine if it meets standards requirements and is in satisfactory operating condition.

Tests are demonstrated by a technician of the laboratory being inspected for the CCRL inspector to assure that the prescribed procedures are being followed. The tests are selected by the laboratory from the following required list of methods in ASTM C1077: Making and Curing Concrete Test Specimens in the Field (ASTM C31); Compressive Strength of Cylindrical Concrete Specimens (ASTM C39); Unit Weight, Yield, and Air Content (Gravimetric) of Concrete (ASTM C138); Slump of Hydraulic Cement Concrete (ASTM C143); Sampling Freshly Mixed Concrete (ASTM C172); Air Content of Freshly Mixed Concrete by the Volumetric Method (ASTM C173); and Air Content of Freshly Mixed Concrete by the Pressure Method (ASTM C231). The tests demonstrated are compared with the procedures detailed in the standards.

With regard to the technicians working for the accredited laboratory, there are no national certification programs for cellular concrete, as indicated by the following excerpts from ACI's website (www.concrete.org) for certification of testing technicians.

aci American Concrete Institute
Always advancing

Certification Programs

Testing Programs

Assess the knowledge and ability to perform, record, and report the results of concrete field/laboratory tests

AGGREGATE TESTING

CEMENT TESTING

FIELD CONCRETE TESTING

ACI-ICT EN Standards Concrete Field Testing Technician
Concrete Field Testing Technician - Grade I
CSA-Based Concrete Field Testing Technician - Grade I (Canada Only)
Self-Consolidating Concrete Testing Technician

LABORATORY CONCRETE TESTING

Concrete Laboratory Testing Technician - Level 1
Concrete Laboratory Testing Technician - Level 2
Concrete Strength Testing Technician

The closest an agency can come is to utilize a qualified CCRL accredited lab and ACI certified concrete technicians, that read and self-teach, or get training from individuals with years of experience in the testing of cellular concrete, such as those from a cellular foam manufacturer. Inspite of cellular concrete being used since the 1940s, testing certifications and accreditations similar to those for concrete have not been developed for LCC.

Regarding the LCC's permeability, no suitable ASTM procedures exist, without minor modification. While a test cylinder of hardened LCC can easily be inserted in an ASTM D5084 test vessel, the test procedure is not appropriate for testing permeabilities of 1E-3 cm/sec or higher. The letter below from Dr. John Kevern from 2013 concurs that ASTM D2434 is more applicable (regarding permeabilities tested and final flow rates involved), although he does not address how LCC would best be inserted in the D2434 testing

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12/11/13

To Whom It May Concern,

This response is regarding discussion of permeability testing methods for pervious cellular concrete with a permeability requirement of $5x10^{-5}$ m/s. Two ASTM standards (ASTM D2434 and ASTM D5084) are available to determine permeability in this range. ASTM D5084 specifically states that ASTM D2434 is the appropriate method when permeability is $1x10^{-5}$ m/s or greater. When permeability ranges from $1x10^{-6}$ m/s to $1x10^{-11}$ m/s, ASTM D5084 is appropriate. In order to achieve permeability in the range of $1x10^{-5}$ m/s to $1x10^{-6}$ m/s using the ASTM D5084 equipment, modifications must be made to the testing setup. In my experience testing using ASTM D2434 is much simpler and would be more appropriate for this application. Pervious concrete is commonly tested using falling head or constant head setups.

Based on the specification requirement of $5x10^{-5}$ m/s, the modified ASTM D5084 would only be appropriate if the sample fell between $1x10^{-5}$ m/s and $5x10^{-5}$ m/s. If the permeability is greater than $1x10^{-5}$ m/s, ASTM D2434 must be used. Since many cellular pervious concretes have permeability values much higher than $1x10^{-5}$ m/s, I would recommend first testing using the simpler ASTM D2434 method and only testing using the modified ASTM D5084 if the first result was outside the allowable test range.

Sincerely,

John Kevern

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equipment. The standard permeability cell is designed for sands and other cohesionless materials to be compacted within the cell, using porous stones and fine wire-mesh screens, which would plug and foul if fluid LCC was placed in the cell. Therefore a similar cell was created to allow a wet-cast LCC sample to be prepared for testing, without those problems; please see the attached testing procedure for sample modification (Attachment 1).

Natural Saturation Density Testing

With regard to natural saturation density testing, this testing evolved from questions during the initial TAP review regarding buoyancy calculations; the naturally saturated density of the LCC in service had never been tested formally before. Aerix Industries funded a testing study with hydraulic heads of one to twelve feet, with fairly consistent results. Afterwards, testing for natural saturation density was incorporated with the LCC testing of D2434 permeability as a routine test procedure; please see the attached document for the "laboratory" test procedure for natural saturation density (Attachment 2).

Field Testing of Natural Saturation Density

With regard to field testing of natural saturation density, during the Pilot Project it became apparent that the LCC's permeability was more than adequate to handle slow rises in groundwater from attenuated tidal action. But for de-watering termination and buoyancy issues, natural saturation density was more important. During the course of the project review, Stan Peters was asked to develop a faster field test for saturated density, which CRC did; this procedure was submitted to the Montez Group for review and modification, and reviewed by Langan.

Field Permeability Testing

Similarly, a field permeability test was desired for QA/QC testing on-site. The Port asked that a field percolation test developed by CRC in the past be modified into a Falling Head Permeability test, using a half filled 6"x12" open-ended cylinder mold, after LCC was cast inside it. This test procedure was submitted to the Montez Group as well for review and modification.

Since several of these modified ASTM tests were developed for this project, the TAP Panel suggests that Public Works, once it concludes its review and approval of them with Langan as the EOR, have them included with the relevant project specifications.

If SFDPW would like additional expert opinions on the validity of the modified ASTM tests discussed above for the Mission Rock project, the TAP suggests the following experts/entities who have relevant experience with permeable LCC and could be engaged to review the test procedures.

- Madrid Engineering Group, Bartow, FL – Jason McSwain, Laboratory Manager Jason.McSwain@madridcpwg.com, 863-533-9007
- Twining Laboratories, Long Beach CA – Boris Stein, www.twininginc.com
- Knight-Piesold, Denver CO – Jani Bruce, 720-354-3411
- Dr. John Kevern, University of Missouri – Kansas City
- Cell-Fill, Grove, OK (an LCC producer laboratory) – James Diver, Engineer/Owner www.cellfill.com, 918-787-2355

Projects Utilizing Foaming Agent produced by Aerix Industries

With regard to information on projects utilizing the concrete foaming agent produced by Aerix industries, all informationthat is available for public review on those projects is listed on the Aerix website (https://aerixindustries.com/).

CRC Colorado Client CLSM Mixes

With regard to the mix designs referenced in the RE Table on pages 75 - 76 of the TAP report, attached (Attachment 3) are copies of five of the six mix designs that were included in the "CRC's Colorado Client CLSM Mixes" portion of the RE Table in the TAP report. The original sixth mix design is not readily available, however, a similar mix design has been included in place of the Client #3 mix design. Testing was performed by CRC staff or by the following producer technicians; all of whom meet the professional requirements of the Colorado DOT: Flashfill Services, Ready Mixed Concrete, and On-Demand Concrete.

Additional Products

With regard to identifying additional products that meet the project specification for LCC, most of the materials that make up LCC, such as water, cement, aggregates, fly ash, etc. are available from multiple sources. SFDPW has expressed concerns about the specified permeable cellular foaming agent, which is available from only one manufacturer. The TAP panel is not aware of any other manufacturer that produces a similar foaming agent that meets all of the design requirements of this project, including load reduction, buoyancy, and strength.

SECTION 1.12 – EXCAVATABILITY

- The reference to ACI536.1R-06 was a typo. Both ACI 229R-13 and ACI 523.1R-06 are applicable to the proposed LCC.
- The reference to ACI536.1R-06 was a typo. The correct reference is ACI 523.1R-06, Guide for Cast-in-Place Low-Density Cellular Concrete.

- Flashfill is a rapid-setting flowable fill material, originally consisting of water, Class C cementitious flyash, and an inert filler such as sand, non-cementitious fly ashes, bottom ash, etc. It was originally covered by the 1992 Bennett patent #5,106,422. Flashfill made with the '422 had a fatal flaw of patch heave caused by ice-lens development in the below surface fracture cracks from premature asphalt patch compaction. This flaw was corrected with the inclusion of cellular foam, lowering the modulus of elasticity, and preventing fracture cracking; it is covered in the 2014 Patent # 8,747,547; it currently is made with 100% Class C fly ash, water and cellular foam.
- "High-strength, flashfill mixture" was developed as rapid-backfilling material for pipeline repair work that could withstand forces of water hammer. Denver Water's requirement was a minimum of 100psi in 4hours, yet still be excavatable with a maximum RE of 1.5 at 28days. Adjustments in the proportions give it additional strength, over "normal" flashfill. Denver Water accepts the material, as tested by CRC (using the Flashfill letterhead for submittal purposes); independent testing by certified laboratories was not needed, nor performed to our knowledge.
- The technical basis of the "Hypothetical table" on page 75-76 was using the sum concrete batch weights provided, dividing by 27cf/CY to determine the approximate unit weight of the mixture. This value, with the reported 28day strength, was used in the RE equation to determine the Removability Modulus.
- A qualitative technical basis for the assumption that cement hydration mechanism in LCC is similar to aggregate-based concrete is : Normal concrete fundamentally consists of Portland cement and water to form the hardening binder that gains strength; fine and coarse aggregates are used to occupy most of the volume in in concrete and add economy, but do not contribute to strength gains by themselves. In LCC mixtures, the paste is also created by Portland cement and water; pre-formed cellular foam provides much of the volume for economy as well as the desired light unit weight/density. In both cases, the cement paste subject to continued hydration controls strength gain. Neither air bubbles nor aggregates have inherent strength by themselves without the paste.

  Quantitavely, a comparison of strength gains from 7days to 28days with Portland cement is shown in the following table. The first set of values are from a Colorado client making normal concrete (3000 to 4500psi mixes, with sand and gravel) with Cemex cement (Type I-II, Lyons, CO Plan). The second set of data are for cellular concrete made with the same cement.

Normal Concrete

Design, psi	7Day, psi	28Day, psi	% Gain
3000	4280	5170	121%
3000	3600	5100	142%
3500	4250	5220	123%
3500	4720	6020	128%
4000	4510	5610	124%
4000	4510	5610	124%
4500	4860	6070	125%
4500	4970	5950	120%
		Average	**126%**

LCC

Design, pcf	w/c	7Day, psi	28Day, psi	% Gain
40	0.60	160	250	156%
40	0.70	190	270	142%
40	0.80	125	180	144%
50	0.60	270	530	196%
50	0.70	220	370	168%
55	0.60	520	610	117%
55	0.70	335	410	122%
70	0.70	540	890	165%
			Average	**151.4%**

- The complete documents from which the tables and graphs on pages 77, 78, and 79 are taken are referenced on page 77. A link to the ASCE 2010 article is as follows: https://ascelibrary.org/doi/pdf/10.1061/41141%28390%296 Please see Attachment 4 for a copy of USBR Report DSO-05-05.
- The results shown on Page 80 are the results of an internal, self-funded CRC study that Mr. Peters shared with the TAP. It first shows the effects of using higher water contents than the w/c of 0.55 as recommended; the permeability results were quite close. It also tested non-pervious foam vs. pervious.

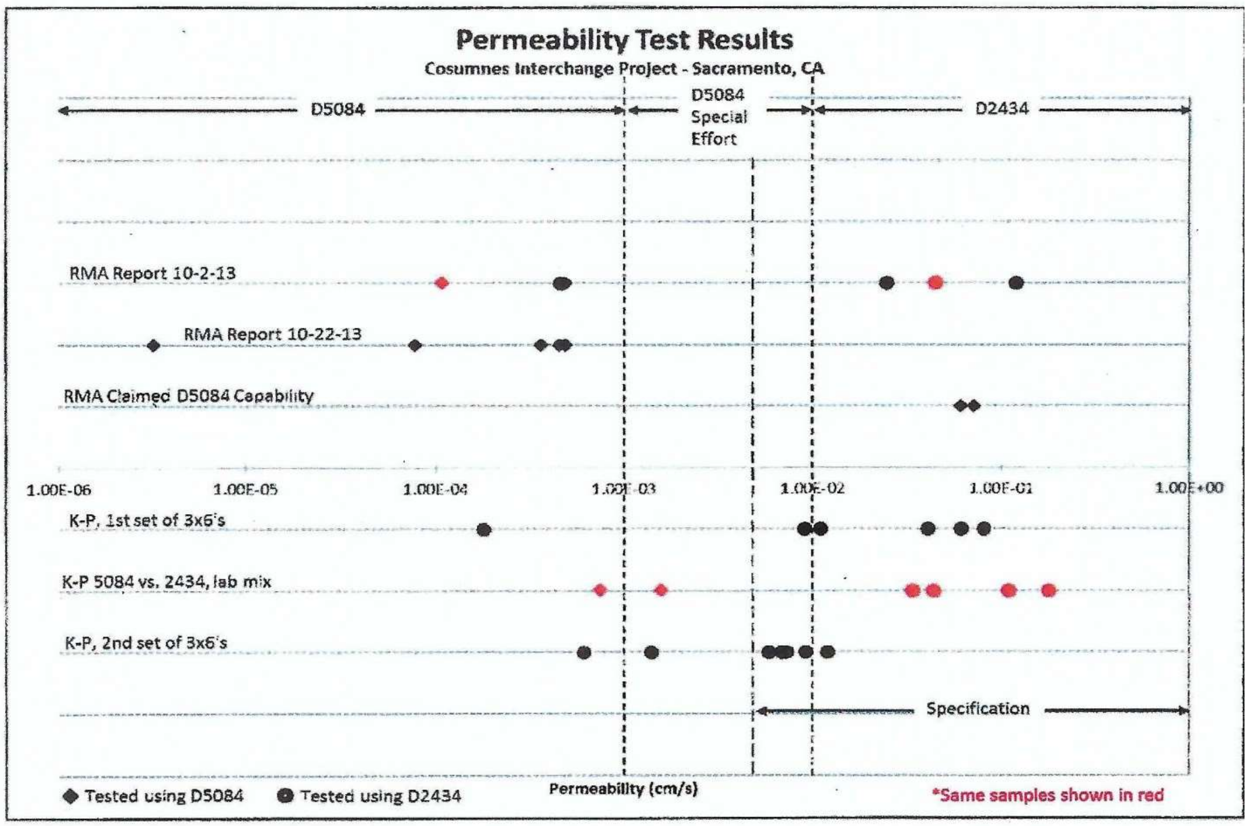
  It is known in the LCC industry if non-pervious LCC is made light enough, it starts becoming pervious. However, the non-pervious foam is less consistent, predictable and not as stable for long-distance pumping as needed (final product quality) as the pervious foam solution. The higher permeability tested in this case is likely due to more cell walls between bubbles breaking down (instability).

  While the Aerix pervious material is readily available to various LCC producers serving the San Francisco area, for small utility trench backfill for utility repairs in the future, non-pervious foam could be considered as a backup foam; for these applications, the LCC is "tailgated" into the trench, with no or only short-distance pumping required.

SECTION 1.13 – LCC DURABILITY

- Mr. Peters presented reasonable evidence that permeability should be tested with D2434, not D5084. It was his understanding that the engineers of record accepted the evidence, since they "accepted" the majority of the LCC, based on his explanation of the most appropriate permeability test to perform. RMA's assertion that they could test materials up to 7E-7 cm/sec conflicts with D5084's recommendations as well as their own test data. It appears that afterwards their engineers chose to judge the D5084 test data as failing, but performed other calculations that indicated the design was acceptable with regard to buoyancy. Mr. Peters' involvement ended when his client's material was accepted to remain in place.
- It is rather difficult to disclose what one does not know about, such as the Dr. Kevern letter. For whatever reasons, the engineers chose to ignore the recommendations of D5084, CRC's analysis of RMA & CRC's data, and Dr. Kevern's recommendations of using ASTM D2434 to test the permeability of LCC. The 2013 permeability testing in Sacramento was the first time the patented product had ever been tested in the laboratory before and testing procedures for LCC have evolved and improved since then.

- Their ill-advised choice of D5084 testing; however, should not become a precedent for this project. Engineering logic guides one to testing LCC with ASTM D2434, even if a slight modification to house the LCC sample in the permeability cell is required.



SECTION 1.14 – NATURAL SATURATION DENSITY TESTING

- As noted above, for the tests under discussion there are no appropriate ASTM tests available that can be utilized for LCC without minor modifications. If SFDPW would like additional expert opinions on the validity of the modified ASTM tests discussed above for the Mission Rock project, the TAP suggests that the experts/entities listed above, who have relevant experience with permeable LCC, could be engaged to review the test procedures and provide additional opinions of reasonableness.

AFFILIATION WITH CASTLE ROCK CONSULTING

- Mr. Peters was one of numerous technical experts submitted to the Port, and was selected as one of the three TAP members, largely due to his experience with LCC and ability to develop new tests as well as new materials. Mr. Peters was asked to develop new testing procedures for Mission Rock, to answer LCC questions that had never been asked before. He worked with Langan, to address questions the TAP raised in its review process of the design.
- Mr. Peters worked in good faith, as a professional engineer to help answer design questions about LCC in service for the Mission Rock project, working for the Port and with the GEOR. This work advanced the technology for permeable LCC, and made a significant contribution to the success of a landmark project, which Mission Rock is sure to be assuming it is approved for permit.
- In the TAP's opinion Mr Peters has provided expertise that would be difficult to find in individuals that are in no way affiliated with the industry. It is the TAP's understanding that expertise of that nature is exactly what the City Agencies were looking for when they assembled the TAP. All TAP members have worked in good faith on this review project and in the TAP's opinion have successfully provided an independent review. Throughout that process TAP members have made various suggestions to improve the design. If SFDPW would like additional confirmation of any of the TAP's recommendations, we have provided a list of several experts/entities who have relevant experience with permeable LCC and could be engaged to provide further confirmation.

DRAIN ROCK UNDER LCC

The project has been designed without a layer of drain rock beneath the LCC. In the TAP's opinion drain rock is not required. Drain rock was included in the Pilot Program to facilitate testing but is not required in the permanent construction.

RECOMMENDATIONS FOR INSTRUMENTATION AND TESTING PROGRAM TO MONITOR LONG-TERM PERFORMANCE AND PROPERTIES OF LIGHTWEIGHT CELLULAR CONCRETE

<u>Objectives</u>
The overall objectives of the monitoring and testing program are to evaluate the long-term performance and potential changes in the properties of lightweight cellular concrete (LCC) for an extended period for the Mission Rock Project. We recommend these objectives be evaluated with data and information obtained via sampling, laboratory testing, field instrumentation, in situ monitoring, surveying, photogrammetry, and visual observations.

<u>Array Locations</u>
We recommend that the project team establish at least two array locations to be used for instrumentation and repeated in situ samplings of undisturbed specimens of the LCC. We

recommend that one array is located in an LCC treated area that is near the northern extent of the project where tidal fluctuation is expected to be relatively large. The second array should be positioned further inland, where tidal influence is expected to be less. We recommend that an area approximately 15' by 15' be used for the array locations to accommodate the recommended LCC sampling and in situ monitoring instrumentation. Perhaps, the best places for the arrays would be in LCC landscaped areas where geotechnical drilling operations can be easily carried out. An as-built survey of each array location should be made, with distances from nearby permanent features (building corners, sidewalks, planter boxes, etc) so it can readily be relocated in the future for additional drilling and sampling.

Sampling

Frequency

We recommend that annual sampling be carried out for the first 4-year post-construction period, followed by biennial sampling for subsequent years up to a minimum of 10 years.

Method of Drilling and Sampling

Intact (undamaged and uncracked) specimens of LCC must be obtained for laboratory testing. Unfortunately, the LCC specimens may be damaged by routine geotechnical drilling and sampling methods. To minimize this potential, we recommend that triple tube coring and sampling be performed at the array locations. We recommend that the full depth of the LCC be sampled at each array and specimens visually inspected for any signs of disturbance or damage from the core drilling and handling. Specimens should be obtained above and below the water table for subsequent laboratory testing. In areas where daily water table fluctuation is significant, data from the array piezometer should be used to help determine the sampling depths.

Laboratory Testing

Specimens obtained from the coring operations should be tested for the following properties: (1) unconfined compressive strength, (2) permeability, (3) degree of saturation, (4) in situ unit weight, (5) air-dried unit weight. We recommend this suite of testing for specimens retrieved above the water table (i.e., partially saturated, i.e., moist) and specimens below the water table (saturated or nearly saturated, i.e., wet density). The depth and location of the specimens from the coring operations should also be documented. Photographs of the specimens should be taken before and after testing.

Unconfined compressive strength testing shall be done following ASTM C495. Permeability testing should be done following ASTM D2434, as modified by Castle Rock Consulting (see Attachment 1 of this letter). Degree of saturation, in situ, and dry unit weights, can be determined using ASTM D7263, or modifications to that.

Field Instrumentation and In Situ Monitoring

Piezometers

We recommend that one piezometer be installed at each array to monitor short-term and long-term fluctuations of the water table. The US Bureau of Reclamation (USBR 6515) can be used as a guide for the selection of the type and installation of the piezometer. Also, the geotechnical engineer of record (GEOR) should be consulted in selecting and installing the piezometers. The project team should determine the reading schedule of the piezometer in conjunction with the City Engineer. We also recommend an increase in the frequency of monitoring one week before any coring operations at the arrays to establish the maximum and minimum water levels during the field coring and sampling events.

Magnet Extensometers and Settlement Plates

The magnet extensometer (also known as a borehole or magnet reed extensometer) is used to measure settlement in foundations and embankments (Fig. 1). Data gathered from the extensometer indicates the depths at which settlement has occurred as well as the total amount of settlement. This type of extensometer is especially useful for identifying the overall compression or settlement within targeted layers.

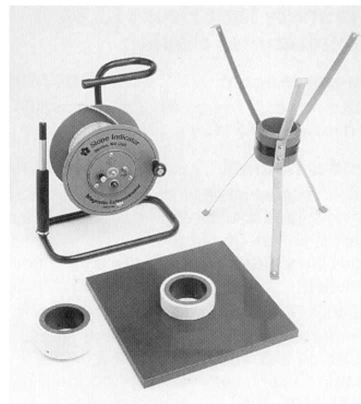


Figure 1 Magnet extensometer equipment. Tape reel (left). Spider magnet (right) and base plate magnet (center middle)

The position of the magnets must be pre-planned to capture settlement within specific layers of interest. Generally, a magnet is placed at the top and bottom of a compressible layer to measure the compression of that interval with time. Geotechnical subsurface explorations (e.g., CPT soundings) are a valuable tool for identifying compressible soil layers underlying the LCC and positioning of the spider magnets.

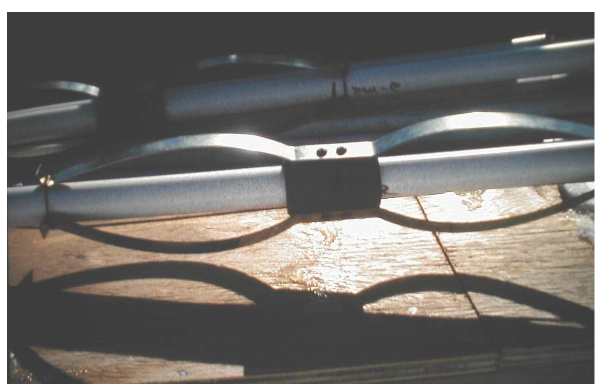


Figure 2 Spider magnet in retracted position

The first step in the installation of a borehole magnet extensometer is to drill a borehole to the desired final depth. We recommend that the borehole be completed to the bottom of the Young Bay Mud and a spider magnet installed at this depth. We also recommend a spider magnet be installed at a depth corresponding to the top of the Young Bay mud. Spider magnets are positioned in a retracted position along the access pipe at the appropriate depths (Figure 2). A locking cable is wrapped around the spider legs, and an anchor pull pin holds the locking cable in place. An anchor pull cable is attached to the anchor pull pin. The locking cable not only keeps the spider legs in the retracted position but also secures the spider magnet at the appropriate depth during installation. When the setup is complete, the access pipe sections are connected and placed down the borehole. The anchor pull cables are pulled, releasing the locking cables, and allowing the spider legs to spring into place. The borehole is grouted through a tremie pipe using a soft bentonite grout that will not impede the settlement of the ground.

We also recommend that a separate standpipe be used to measure any compression of the LCC using base plates positioned during construction at the base of the LCC and the top of the LCC.

For all installations, a protective secondary casing is placed around the top of the magnet extensometer. This casing keeps the access pipe free from debris and serves as a protective cushion. Also, because the magnet extensometer must be read from the top, reading access and safety must be taken into consideration in determining the instrument location.

Care should be taken in selecting the reference point for collecting and reducing the magnet extensometer data. Three possible reference points can be used: (1) bottom of the access tube casing, (2) depth to the bottom magnet, (3) top of the casing of the access tube.

If the extensometer is installed to a depth that is greater than the compressible layers (i.e., Young Bay Mud), then reference point (1) is often used. The selection of this reference point essentially assumes that the bottom of the casing will not settle significantly. If settlement does occur below this depth, it will not be possible to measure or estimate its magnitude. However, the results may still be used to calculate the relative movement and compression of specific layers between the magnets. The use of reference point (1) can also give erroneous readings if soil, obstructions, or other items are present or can enter and settle to the bottom of the casing. For this reason, it is often preferable to use reference point (2), where the bottom magnet is usually placed below the depth of the compressible zone. However, selecting reference point (2) also suffers from the same limitation of that of (1) because one is not able to measure the amount of settlement occurring below the reference point.

Reference point (3) can be used for installations where it is desirable or necessary to measure settlement occurring below the bottom of the casing or the bottom magnet. Using the reference point requires a corresponding optical level survey to be done at the top of the casing for each set of extensometer readings. With this information, it is possible to establish elevations for the top of the casing and each magnet position versus time. The settlement that has occurred below the bottom of the casing is simply the elevation of the bottom of the casing at the time of interest, minus the elevation for the same point established during the baseline reading. However, a slight caution is warranted for cases where rigid casing has been installed and where small settlement measurements are being attempted (e.g., creep settlements for geofoam). In this case, it is possible to have minor thermal expansion and contraction of the casing, which introduce a few millimeters of error into the calculations. We have found that even a few millimeters of thermal effects can partially obscure the small amount of creep settlement that occurs for geofoam installations. For this case, it may be better to select reference point (1) or (2) to interpret the data and neglect the settlement that is occurring below the LCC.

The project team should determine the reading schedule of the magnet extensometers in conjunction with the City Engineer. As a minimum, we recommend that the magnet extensometers be read quarterly for the first three years, followed by an annual reading in subsequent years. Also, the magnet extensometers should be read just before any coring of specimens submitted for laboratory testing.

Surface Survey Settlement Points

Surface settlement points are used to monitor the settlements of embankments, zones, and structures, MSE walls, buildings, etc. These measurements are used in conjunction with other data to provide a more comprehensive picture of the amount and settlement pattern throughout the project. The project team, in conjunction with the City Engineer, should determine locations for settlement points. Settlement points will also be required at each array location. Also, if surface effects of differential settlement appear with time (i.e., cracking of pavement, concrete, etc.), additional survey points should be installed at these locales.

Settlement points are surface monuments that have been established by optical surveying and are monitored periodically to measure settlement by changes in elevation. These points consist of two basic types: those placed within the soil and those within concrete or pavement. Settlement points placed within earth include a cased piece of rebar that has been driven into the ground (Figure 3). The rebar is a 24-inch (610 mm) length of 3/8-inch diameter reinforcing bar (i.e., rebar), and the casing is an 18-inch (457 mm) long, ½-inch (12.7 mm) diameter capped galvanized steel pipe (i.e., thread-pipe riser). The length of rebar should be of sufficient length to penetrate below the frost heave depth. Settlement points placed within concrete are either a ¼-inch (6.4 mm) diameter lead plug that is placed in a pre-drilled hole or a PK nail driven directly into the pavement of a roadway or building foundation.

Figure 3 Settlement point with casing. Rebar has been driven or set into ground and cased.

To place a settlement point within a concrete surface (e.g., building foundation or concrete pavement), a ¼-inch (6.4 mm) diameter hole is drilled into the concrete. A hammer drill works best for drilling holes in high strength concrete. For settlement points placed directly in the concrete road surface, a lead plug that is slightly longer than the depth of the hole is inserted into the hole. The exposed portion of the plug is hammered flat until the top of the plug is nearly flush with the road surface. This type of settlement point is intended to be safe from snowplows. For settlement points not directly in the roadway surface, a PK nail is hammered into the hole. These nails have a fluting that is designed to lock tightly within the concrete sides of the pre-drilled hole.

Surveying of Settlement Points

Figure 4 Self-reading digital level and rod

Experience has shown that great care must be taken to obtain accurate elevation information from surveying techniques. Generally, GPS or total station surveying will not provide sufficient accuracy to see relatively small movements expected for the Mission Rock Project. We recommend that digital differential level surveying be done using a Sokkia SDL30 Digital Level (or equivalent) and the corresponding RAB-Code (Random Bi-directional Code) staff, also known as the rod (Figure 4). The Sokkia SDL30 Digital Level is extremely operator-friendly. The technician need only aim the digital level at the rod, adjust the focus, and with a touch of a key, measure the height and distance. Readings can be taken at a distance of 5.3 to 328 ft (1.6 to 100 m). One side of the RAB-Code staff consists of a bar code, which is automatically read by the digital level. The digital level can interpret the unique code pattern on the rod and give height measurements to the nearest 0.004-inch (0.1 mm). Typically, the rod has a maximum height of 16.4 ft (5 m) and has a portable handheld level that is placed on the rod to ensure that the rod is vertical.

The selected benchmarks should be positioned on stable objects that can be relocated and reused on subsequent surveys. It also should be located so that a permanent point can be preserved without disturbance for an extended period (approximately ten years). The relative elevation of each of the survey points is then established by completing a level circuit. This circuit should be closed on the initial benchmark and adjusted, as necessary, to determine the baseline elevations of all survey points. Once the baseline survey is established, the amount of settlement is then measured by subsequent surveys by calculating the difference between the current reading and the initial baseline reading.

For the locations of the arrays, more than one benchmark should be established. In general, secondary benchmarks are placed at the array in the event something destroys the primary benchmark. In most cases, each survey can be performed solely by using the primary benchmark. However, periodically the elevations between the primary benchmark and the secondary benchmark(s) should be checked to verify that the primary benchmark has not been disturbed. If there is reason to suspect that the primary benchmark has moved, or the survey results appear to be suspect, the elevation of the primary benchmark should be checked against all secondary benchmark(s).

To ensure that survey error is kept to a minimum, each survey should be closed. This means that the survey should begin and end at the same benchmark. It is recommended that each survey should close with less than 3 mm of error. If a survey is completed and the total error is greater than 3 mm, then the survey should be repeated until the error is less than the required tolerance of 3 mm.

The project team should determine the reading schedule of the survey points in conjunction with the City Engineer. As a minimum, we recommend that the surveys be completed in conjunction with reading the magnet extensometers, i.e., read quarterly for the first three years, followed by an annual reading in subsequent years. Also, the survey points should be read just before any coring of specimens submitted for laboratory testing.

Photogrammetry and Visual Observations

These techniques offer a cost-effective and rapid way to inspect the overall performance of the infrastructure in the LCC treated areas. We recommend that Unmanned Aerial Vehicles (UAV) be used to perform an annual assessment of the infrastructure (primarily of the streets) by developing 2D Orthomosaic map(s). Also, if cracks develop in pavements or other concrete flat works, the location, size, and width of the cracks should be GPS located and photographed.

We recommend that the UAV survey be completed annually. Crack documentation and mapping should be done on an “as-needed” basis.

The TAP has some experience performing UAV reconnaissance using a DJI Phantom 4 version 2 UAV in conjunction with DJI Terra software. These are recommended for use.

Post-Earthquake Assessment

The TAP recommends that reading of field instrumentation, in situ monitoring arrays, surveying, photogrammetry, and visual observations, be repeated following earthquakes that produce 0.1 g peak horizontal ground acceleration in the Port of San Francisco area.

UAVs and field visual inspections should be done quickly to note any areas of potential damage or concern. Locations of buried utilities should also be inspected for any new cracking or damage to the LCC or associated utilities.

ACKNOWLEDGEMENT AND DISCLOSURE FORM

Tap members will provide Acknowledgement and Disclosure Forms under separate cover.

Attachment 1

Permeability Testing Procedure with Sample Modification for LCC Tested in Accordance with ASTM D2434

CRC Modifications of ASTM D2434 Constant Head Permeability Testing for Pervious Low Density Cellular Concrete (PLDCC)

Overview:

The modifications made to the D2434 test as relates to testing Pervious Low Density Cellular Concrete (PLDCC), concern sample fabrication and preparation for testing.

These modifications are based on the nature of PLDCC, and how it differs from permeable soils, for which the D2434 test is intended.

Once the PLDCC sample has been fabricated and prepared for testing, the testing procedure for determining the hydraulic conductivity of the PLDCC, including measurements taken, calculations, and application of Darcy's Law are the same as specified in the D2434 protocol for soils.

These modifications and protocols are outlined in the following text, with drawings for illustration attached in the appendix.

PLDCC Sampling:

PLDCC, being a cohesive cementitious material does not require the same sample preparation as a non-cohesive granular soil. As such, the details specified in the D2434 procedure concerning sample gradation, filling, and compaction in the permeameter cell are not applicable. The use of porous stones or manometer port screens are not necessary, so they are omitted. Instead, the PLDCC samples are cast in place in a PVC cell that replaces the acrylic cell that is supplied with the permeameter (figures 1 and 2).

The PVC cells are fabricated from schedule 40 PVC pipe, to the same dimensions and tolerances as the acrylic cells that they replace, with the exception that the PVC cells do not have the circumferential grooves around the inside wall at the manometer ports (figures 3 and 4).

The manometer ports are located at the requisite distance for the cell diameter and are drilled and tapped to accommodate threaded hose barbs for connecting to manometers. These manometer ports are plugged with screws prior to casting the PLDCC in the cell.

Additionally standard PVC "knock-out" pressure test caps are used to stop the open ends of the cell to contain the PLDCC when cast in the cell. The bottom cap is taped in place around its circumference to prevent material from seeping out. The top cap has its center knocked out and is taped around its circumference to provide a protective ring to prevent damage to the cell rim. A vented slip cap is placed over the top of the cell after filling (figure 5).

The interior wall of the PVC cell is roughened with sandpaper prior to filling, to ensure a good bond of the PLDCC to the walls, in an effort to eliminate sidewall leakage, as well as ensure that the PLDCC remains in place during vacuum saturation.

These PVC cells are an economical alternative to using the acrylic cells and can be fabricated in large numbers, to be shipped off to various projects where they can be filled, and returned to a lab for testing. The cells are also more durable than their acrylic counterparts, and can be cleaned out and reused.

Sample Preparation:

After the PLDCC samples have been cast, and have had sufficient time to cure and develop the permeable structure (usually 7 to 10 days), they may be prepared for testing.

The slip cap is removed, as well as the top ring and bottom cap. Depending on how the cell was filled, excess material may need to be removed, or scraped down below the cell rim. A metal putty knife or chisel works well as a scraping tool and helps keep the surfaces flat and relatively square to the cell ends. Both top and bottom surfaces should be scarified and blown off with compressed air to expose the cellular structure.

The cell can now be placed into the permeameter for testing. The manometer port plugs remain in place during saturation and vacuum saturation of the sample.

After saturation, the permeameter is placed on its side, with the monometer ports face up while connected to the water source with the inlet vale open, and the outlet valve closed. The plug screws are removed and a 1/8” drill with a pilot is used to drill through the sample at the manometer port, stopping at the opposite wall with water flowing through the sample to purge debris from the hole. This provides a pathway for the cross-sectional flow velocity to the manometers and permits the manometers to respond more quickly to flow changes.

Threaded hose barbs may now be installed in the manometer ports and connected to the manometers and the permeameter can be turned upright for testing.

The testing procedure from this point on is the same as that specified in the D2434 protocol when testing soils.

Appendix

D2434 Permeameter with Soil Sample

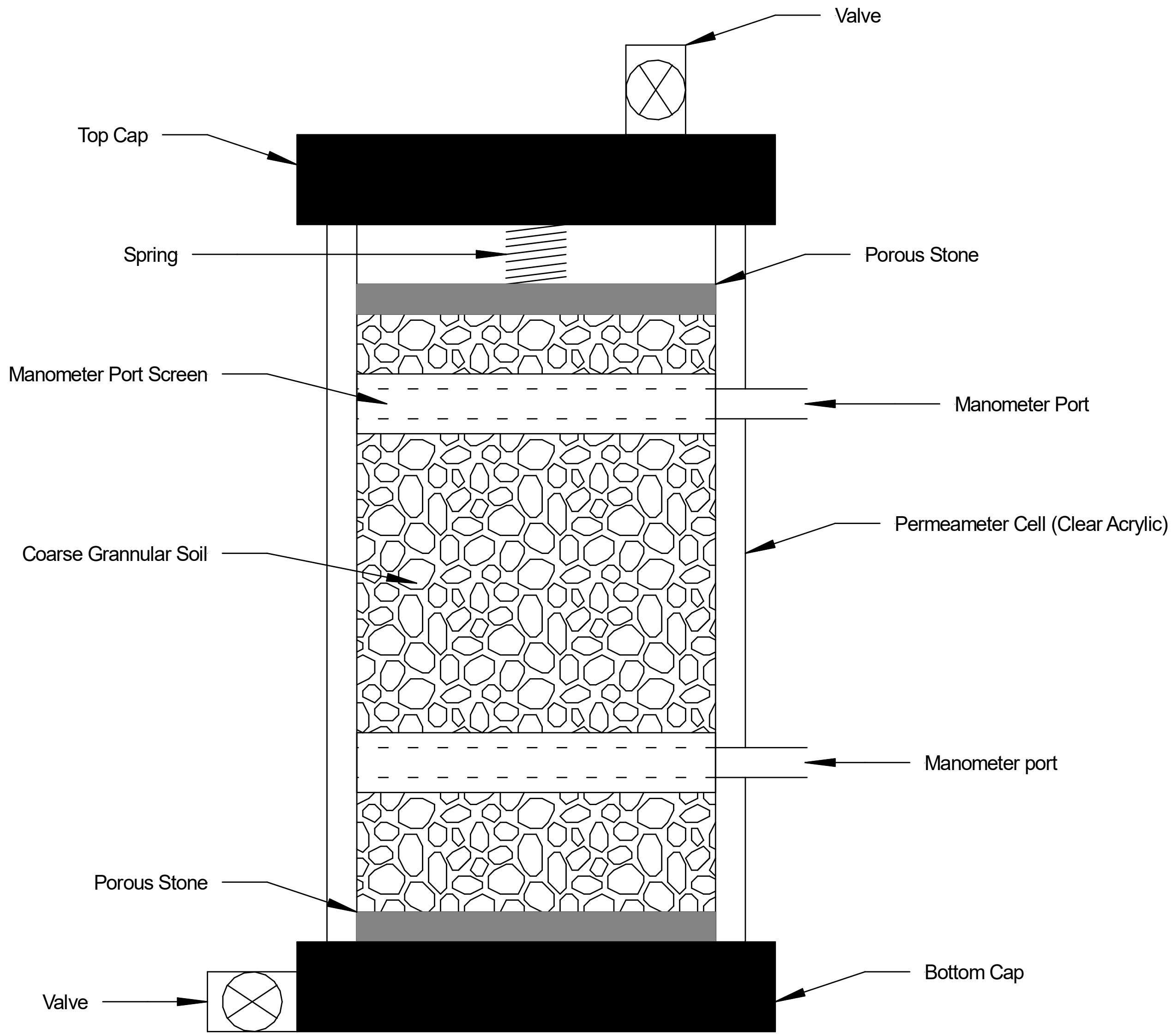


Figure 1

Appendix

D2434 Permeameter - with PLDCC Sapmle

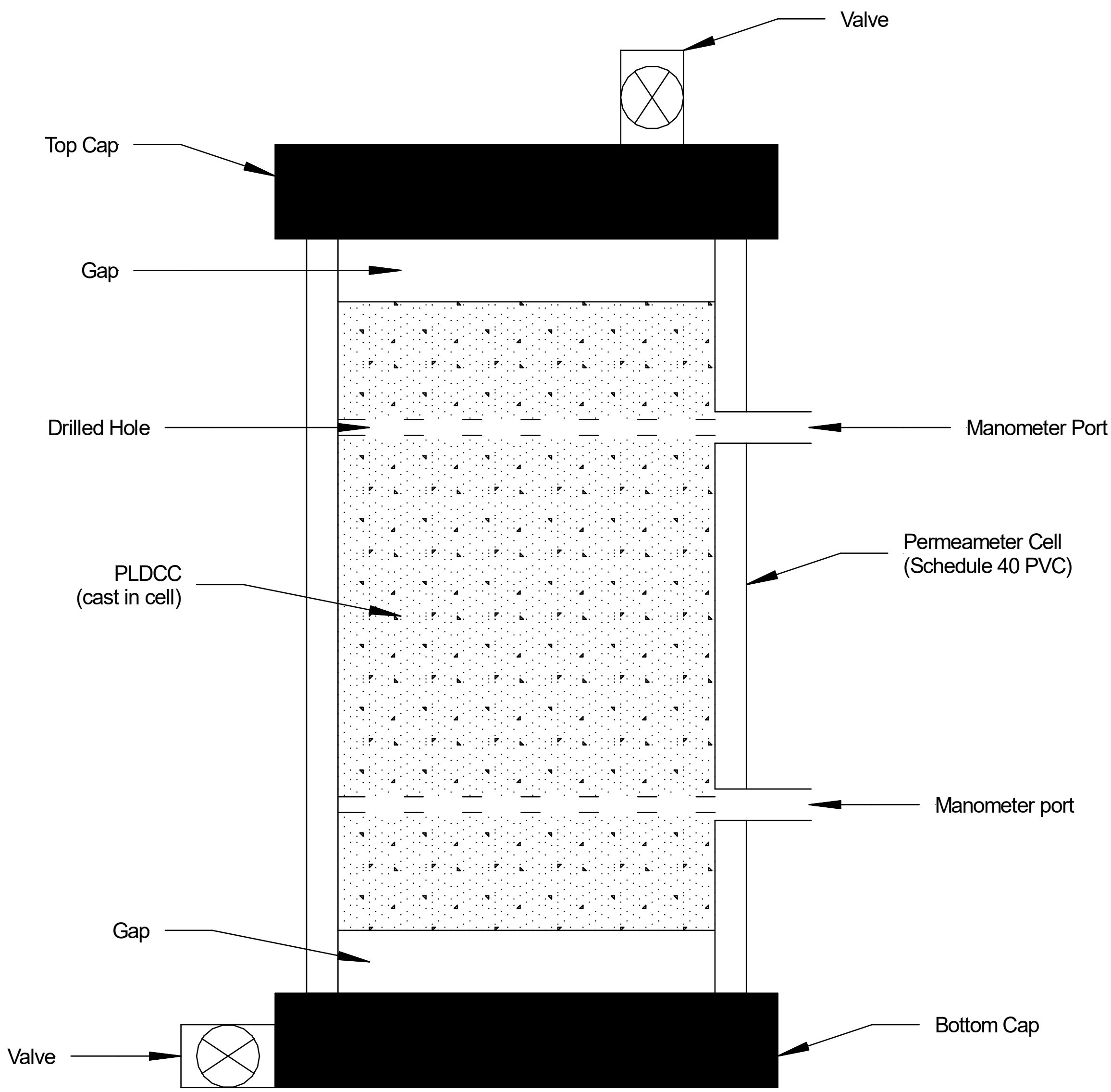


Figure 2

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Appendix

Standard D2434 Acrylic Cell Schematic

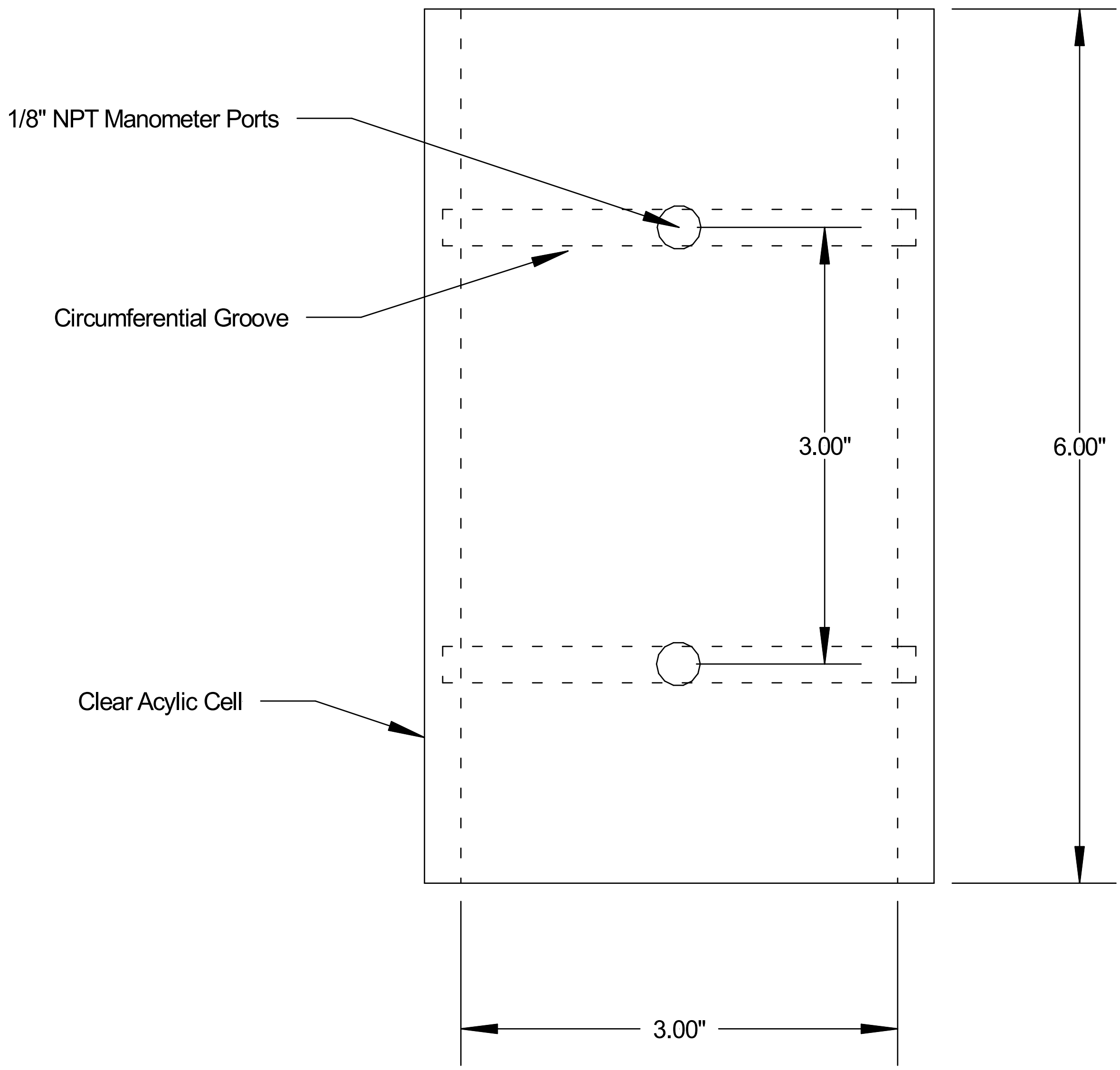

Figure 3

Appendix

PVC Cell Schematic

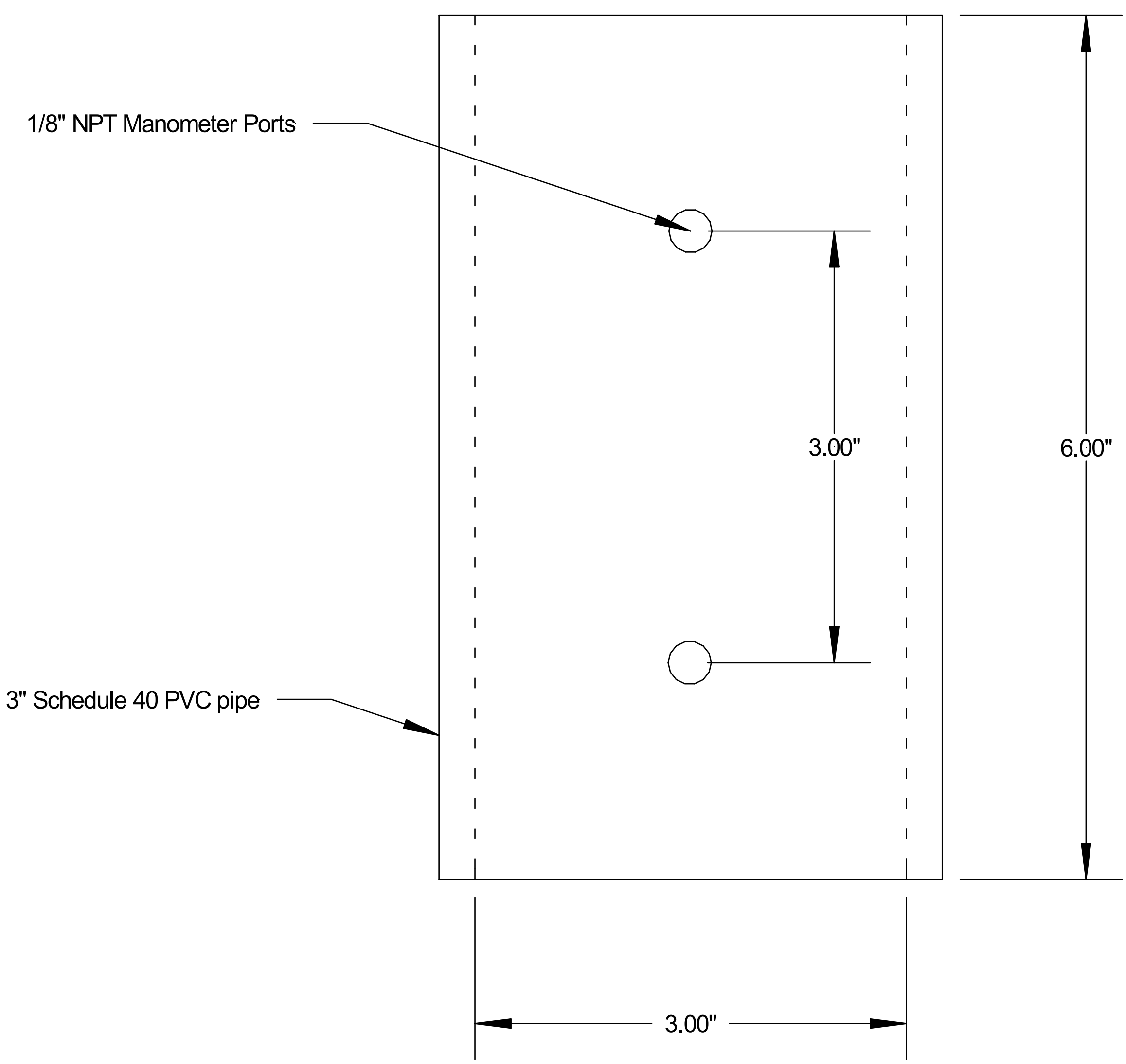

Figure 4

Appendix

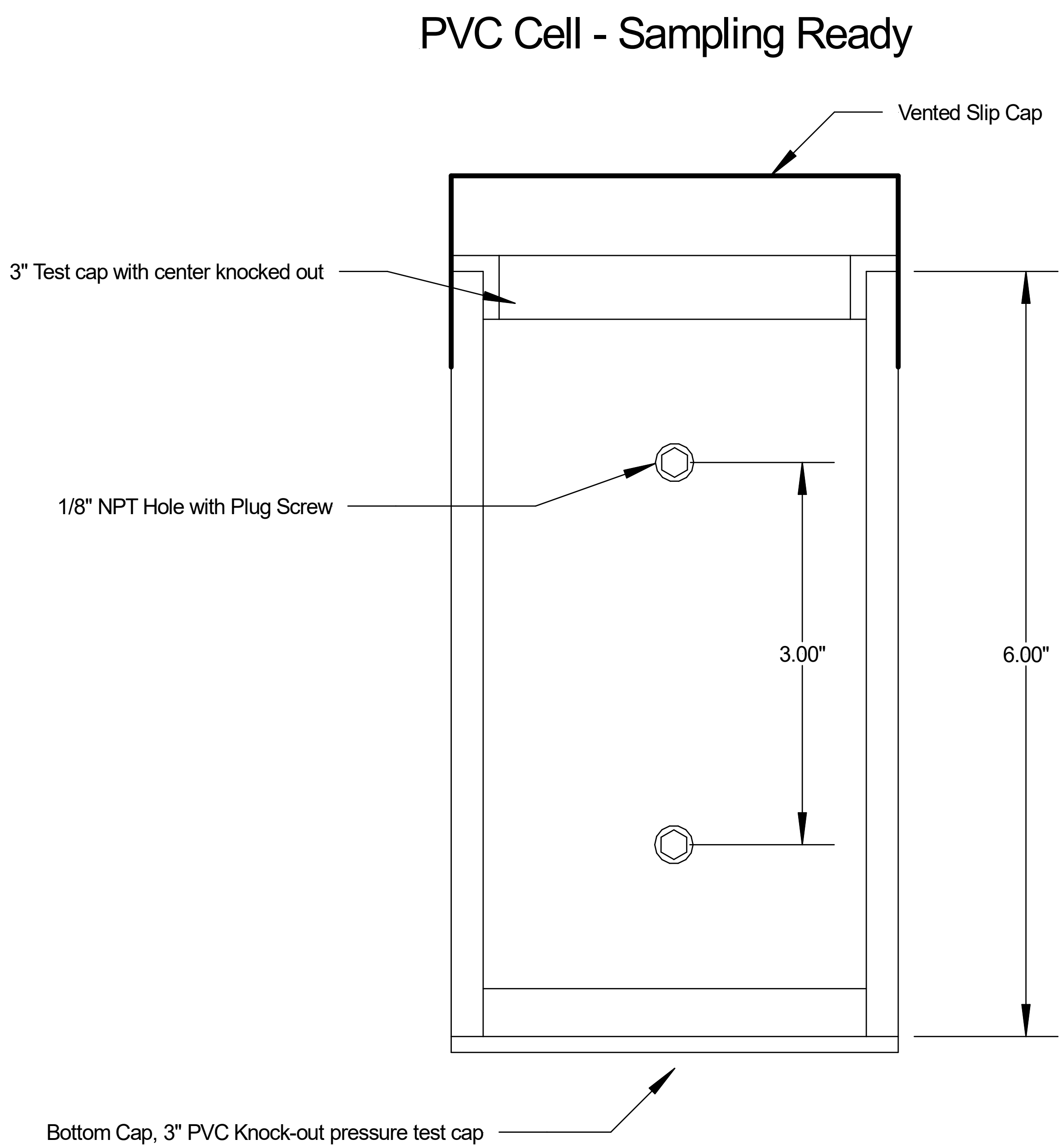

Figure 5

Attachment 2

Natural Saturation Density Testing Procedure with Sample Modification for LCC Tested in Accordance with ASTM D2434

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CRC Method for Measuring Naturally Saturated Unit Weight of PLDCC using the ASTM D2434 Testing Apparatus

Overview:

In some instances, it is desirable to know the saturated unit weight achieved by the PLDCC under service conditions. This saturated unit weight is achieved under "natural" conditions as groundwater infiltrates the PLDCC after placement.

The estimation of this saturated unit weight can be measured using the D2434 apparatus and the same PLDCC samples intended for measurement of the hydraulic conductivity. The saturated unit weight is measured prior to testing for hydraulic conductivity. The following procedure is based on a series of trials conducted by CRC in which samples were saturated under varying amounts of elevation head over various time intervals and determined to be sufficient for establishing a "natural" saturation condition.

The protocol is outlined in the following text, with drawings and sample calculations for demonstration in the attached appendix.

Method for Estimating Saturated Unit Weight of PLDCC:

If the saturated unit weight is to be measured, the PVC cell should have a tare weight recorded prior to being filled. The tare weight should not include the caps, only the cell, with manometer plug screws in place and any label for sample identification. The tare weight can be written on the cell ID label. It is best for the testing lab to ensure that this is done before sending the cells out for sampling.

After the cell has been filled and returned for testing, any excess material or spillage on the outside of the cell is cleaned off. The caps and ring are removed. Excess material on the top surface is removed to below the top rim, and both the top and bottom surfaces are scarified and blown off with compressed air to expose the cellular structure, taking care to keep the top and bottom surfaces as square as possible to the cell ends. The weight of the filled and prepped cell is recorded.

The overall height of the cell is recorded, along with the depth from the cell rim to the surface of the PLDCC at both the top and bottom ends. An average of three depth measurements around the circumference can be taken. These measurements will be used to calculate the volume of water in the gaps between the cell ends and the PLDCC surface that is excess water not contained within the PLDCC after saturation.

The cell with the unsaturated PLDCC is placed in the permeameter assembly. The tare weight of the permeameter assembly, with the PLDCC sample cell is recorded.

The permeameter is connected to the constant head water source with the inflow through the bottom valve and the purge plug removed from the top cap. Saturation is conducted under 2' of constant head. Water is allowed to flow freely though the sample until flowing out of the top vent in the permeameter, at which point the flow rate is adjusted to a slow trickle and left for a period of 30 minutes. The upward flow through the sample aids in dislodging trapped air and mimics upward infiltration of ground water through the sample at low flow.

After a period of 30 minutes, the inflow valve is closed and the purge plug replace in the top cap. The permeameter is disconnected from the reservoir, thoroughly dried off and weighed. The assembly weight with the saturated PLDCC sample and the excess assembly water is recorded.

Volume and weight measurements are used to calculate the estimated saturated unit weight of the PLDCC as follows:

The depth measurements from the top and bottom surfaces to the cell rim can be subtracted from the cell height to determine the height of the PLDCC sample within the cell. The sample height in conjunction with the cross-sectional area can be used to determine the volume of the PLDCC in the cell.

The cell tare weight can be subtracted from the unsaturated cell weight to determine the weight of the PLDCC sample, which can be used in conjunction with the calculated volume, to verify the un-saturated unit weight of the PLDCC.

The weight of the permeameter assembly containing the saturated PLDCC cell can be subtracted from the tare weight of the permeameter assembly containing the un-saturated PLDCC cell. The difference is the weight of water in the PLDCC and the assembly after saturation, from which excess water not contained within the PLDCC must be subtracted.

This excess water is a combination of the water occupying the spaces between the PLDCC surfaces and the ends of the cell, as well as water occupying the space between the top end of the cell and the bottom of the top cap of the permeameter, and water in the valves and boreholes in the permeameter base and cap.

The weight of water contained in the valves, boreholes, and gap between the cell and top cap can be measured for a particular permeameter as follows:

1. Place an empty permeameter cell with plugs in the manometer ports in the permeameter.
2. Connect the permeameter to a water source, inflow from the bottom valve.
3. Fill the cell completely with water and purge all of the air.
4. Close the valves and disconnect from the water source.
5. Tare a pan.
6. Carefully remove the top cap, open the valve to prevent suction from holding it in place.
7. Holding the top cap over the pan, shake the excess water off into the pan.
8. Carefully decant the water from the top of the cell with a pipette, past the top of the retaining ring until even with the interface between the top rim of the cell and the retaining ring.
9. Add the water to the pan.
10. Open the bottom valve and allow the water to drain from the cell.
11. Carefully remove the retaining ring and the empty cell.
12. Shake the bottom base over the pan to recover the excess water.
13. Repeating the process a few times gives a good average of the water weight that will be occupying the valves, boreholes and gap.

This excess water from the assembly can be added to the calculated weight of water occupying the gaps between the PLDCC surfaces and cell rims (based on volume from depth measurements.)

This excess water is subtracted from the difference between the tared assembly and saturated assembly weights, giving the weight of water contained within the PLDCC.

Adding the weight of water contained within the PLDCC to the initial un-saturated sample weight and dividing by the calculated volume of the PLDCC will yield the estimated natural saturated unit weight.

Figure 1 conceptualizes the excess water that must be accounted for, as well as relevant dimensions.

An example of measurements and calculations is contained in the appendix.

Appendix

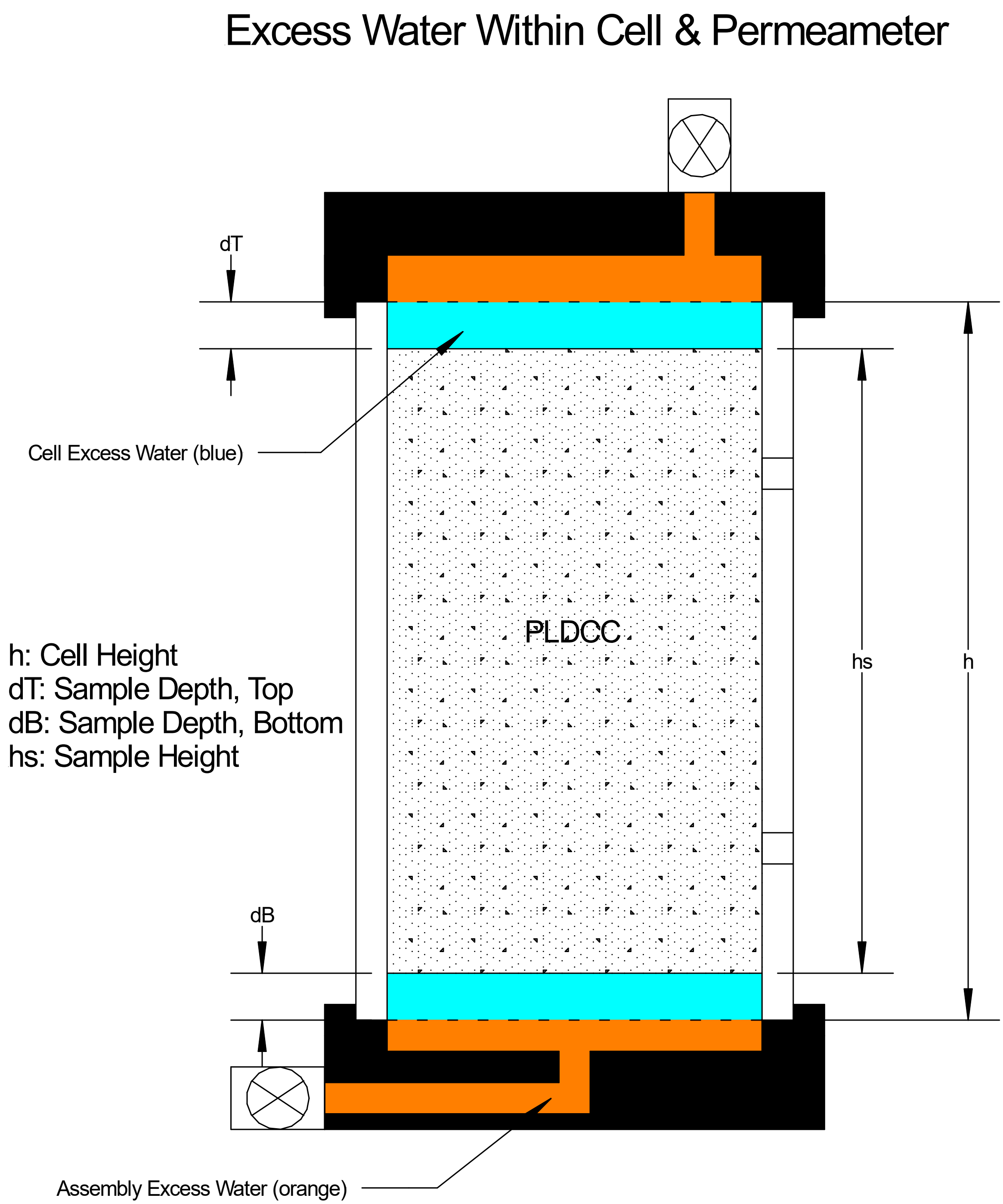

Figure 1

Appendix

Sample Determination of Natural Saturation Unit Weight

Measured Values

Cell Tare Weight, C_T :	330.0 g	Weight of empty cell with manometer plugs and Id label.
Prepared Cell Weight, C :	584.4 g	Weight of cell containing PLDCC sample after trimming & scarifying.
Assembly Tare Weight, P_T :	2201.6 g	Weight of Permeameter containin g un-saturated PLDCC cell.
Saturated Assembly Weight, P_s :	2665.8 g	Weight of dried permeameter containing saturated PLDCC sample.
Cell Diameter, D:	7.6 cm	Internal diameter of the cell.
Cell Height, h :	15.2 cm	Height of the cell.
Top Surface depth, d_T :	1.3 cm	Depth of the top surface of the PLDCC to the cell rim (average).
Bottom Surface Depth, d_B :	1.3 cm	Depth of the bottom surface of the PLDCC to the cell rim (average).
Assembly Excess Water, EW_A:	58.0 g	Weight of excess water contained in the Permeameter assembly as described.

Calculations

PLDCC Sample Height, h_s :	12.6 cm	$h_s = h - d_T - d_B$
PLDCC Sample Cross-sectional Area, A :	45.4 cm^2	$A = \frac{\pi D^2}{4}$
Unsaturated PLDCC Sample Weight, W_U :	254.4 g	$W_U = C - C_T$
PLDCC Sample Volume, V :	571.6 cm^3	$V = h_s \times A$
Unsaturated PLDCC Unit Weight, UW:	0.445 g/cm	$UW = \frac{W_U}{V}$
Unsaturated PLDCC Unit Weight, UW:	**27.8 lb/ft^3**	$UW = \left(\frac{W_U}{V}\right) \times 62.4\frac{lb}{ft^3}$
Total Weight of Water, W_W :	464.2 g	$W_W = P_S - P_T$
Weight of Excess Cell Water, EW_C :	117.9 g	$EW_C = A \times (d_T + d_B) \times 1.0\frac{g}{cm^3}$
Weight of Saturated PLDCC, W_S :	542.7 g	$W_S = W_U + W_W - EW_C - EW_A$
Natural Saturated PLDCC Unit Weight, $UW_{sat.}$:	0.949 g/cm^3	$UW_{sat.} = \frac{W_S}{V}$
Natural Saturated PLDCC Unit Weight, $UW_{sat.}$:	**59.2 lb/ft^3**	$UW_{sat.} = \left(\frac{W_S}{V}\right) \times 62.4\frac{lb}{ft^3}$

Figure 2

Attachment 3

Mix Designs Represented in RE Table on Pages 75-76 of TAP Report

Flashfill Services, LLC

P.O. Box 16091
Denver, Colorado 80216
Office 303-292-7343 Fax 303-292-1176

MIX DESIGN REPORT

Date Mix performed:	2/14/2019	**PROJECT :**	Various
Mix Number:	D-100-33	**Customer :**	Various
Date Mix Reported :	4/22/2019	**Location :**	Various
		Application :	Foamed Flashfill

Material	**per CY**	**Source / Type**
Cementitious Flyash ASTM C 618 Class C	990 lbs	CSU Drake Plant
Water	495 lbs	City, C1602
Cellular Foam	12.7 cf	Aerix Aerlite ix, C796 & C869

Physical Properties

Spread : (ASTM D6103)	18 in
Air Content:	46.6 %
Unit Weight:	55.5 pcf
W/CM Ratio:	0.50

Compressive Strength Data

Age	**PSI**		
4 Hour	70		
4 Hour	60	**Ave:**	**70**
1 Day	100		
1 Day	100	**Ave:**	**100**
7 Day	120		
7 Day	130	**Ave:**	**130**
28 Day	200		
28 Day	200		
28 Day	230	**Ave:**	**210**

Removability Modulus, RE **0.62**

Additional Properties

Modulus of Elasticity:

	21600
	26200
	30500
Average:	**26100 psi**

Permeability: **3.2 E-6 cm/sec**

Thermal Conductivity & Resistivity:

	K	p, rho
Saturated	0.553	181.0
Air Dried	0.184	561.5

Direct Shear:

	C, psf	Phi Angle
Peak	3190	50.2°
Ultimate	0	43.0°

Reviewed by:

Stan Peters, P.E.
Castle Rock Consulting, LLC

COLORADO LICENSED
STANLEY R. PETERS
26128
PROFESSIONAL ENGINEER

Flashfill Services, LLC

P.O. Box 16091
Denver, Colorado 80216
Office 303-292-7343 Fax 303-292-1176

MIX DESIGN REPORT

PROJECT : Various

Date Mix performed: 3/20/2019
Mix Number: DWB-50-70
Date Mix Reported : 4/22/2019

Customer : Various
Location : Various
Application : High Strength FlashFill (DWB Pipe Zone Material)

Material	**per CY**	**Source / Type**
Cementitious Flyash ASTM C 618 Class C	1260 lbs	CSU Drake Plant
Water	630 lbs	City, C1602
Cellular Foam	8.8 cf	Aerix Aerlite ix, C796 & C869

Physical Properties

Spread : (ASTM D6103)	15 in
Air Content:	33.9 %
Unit Weight:	72.2 pcf
W/CM Ratio:	0.50

Compressive Strength Data

Age	**PSI**		
4 Hour	160		
4 Hour	160	**Ave:**	**160**
1 Day	170		
1 Day	220	**Ave:**	**200**
7 Day	270		
7 Day	240	**Ave:**	**260**
28 Day	460		
28 Day	490		
28 Day	510	**Ave:**	**490**

Removability Modulus, RE **1.41**

Reviewed by:

Stan Peters, P.E.
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Office 303-292-7343 Fax 303-292-1176

MIX DESIGN REPORT

Date Mix performed:	8/15/2019	**PROJECT :**	Various
Mix Number:	CX-40-70	**Customer :**	Various
Date Mix Reported :	4/7/2020	**Location :**	Various
		Application :	Cellular Grout

Material	**per CY**	**Source / Type**
Portland Cement	635 lbs	Type I/II - Cemex, Lyons, CO
Water	445 lbs	City, C1602
Cellular Foam	16.6 cf	Aerix Aerlite ix, C796 & C869

Physical Properties

Spread : (ASTM D6103)	11 in
Air Content:	61 %
Unit Weight:	41.1 pcf
W/CM Ratio:	0.70

Compressive Strength Data (using 3"x6" cylinder molds, per ASTM C495 requirements)

Age	**PSI**		
1 Day	80		
1 Day	80	**Ave:**	**80**
7 Day	190		
7 Day	180	**Ave:**	**190**
28 Day	250		
28 Day	270		
28 Day	280	**Ave:**	**270**

ASTM C403 Penetration

420 psi at 24 hours

Removability Modulus, RE **0.45**

Reviewed by:

Stan Peters, P.E.
Castle Rock Consulting, LLC

COLORADO LICENSED
STANLEY R. PETERS
26128
9-7-20
PROFESSIONAL ENGINEER

Ready Mixed Concrete Company
5775 Franklin Street
Denver, Colorado 80216
www.concretecolorado.com

Concrete Mix Design Report

Date Mix ran: 1-23-19

Mix Number: 500907R

Date Mix Reported : 2-28-2019

Class / Use: Flow Fill CDOT

Material	1 Cu. Yd.	Source / Type
Cement	60 lbs	GCC Pueblo Type I-II LA, ASTM C-150
Fly Ash	60 lbs	Boral Class F Coal Creek Terminal, ASTM C-618
Intermediate Agg	1550 lbs	Brannan Aggregate Fort Lupton Pit #9, ASTM C-33
Fine Aggregate	1550 lbs	Brannan Aggregate Fort Lupton Pit Sand, AASHTO M6 ASTM C-33
Water	400 lbs	City

***Note:** Batch weights are based upon aggregates in Saturated Surface Dry condition.

Physical Properties

Slump:	7 in
Air Content:	3.5 %
Temperature:	69 F
Unit Weight:	134.2 pcf
W/C Ratio:	3.33
Yield:	1.00

Compressive Strength Data

Removability Modulus = 1.35

Age	PSI		
7 Day	**40**		
7 Day	**40**	**Ave:**	**40**
28 Day	**80**		
28 Day	**70**		
28 Day	**70**	**Ave:**	**70**

Bryon Blatter
Technical Services Manager

Reviewed by:
Castle Rock Consulting, LLC

Ready Mixed Concrete Company
5775 Franklin Street
Denver, Colorado 80216
www.concretecolorado.com

Concrete Mix Design Report

Date Mix ran: 1-23-19

Mix Number: 500917R

Date Mix Reported : 2-28-2019

Class / Use: Flow Fill Aurora

Material	1 Cu. Yd.	Source / Type
Cement	75 lbs	GCC Pueblo Type I-II LA, ASTM C-150
Fly Ash	175 lbs	Boral Class F Coal Creek Terminal, ASTM C-618
Intermediate Agg	1224 lbs	Brannan Aggregate Fort Lupton Pit #9, ASTM C-33
Fine Aggregate	1836 lbs	Brannan Aggregate Fort Lupton pit Sand, AASHTO M6 ASTM C-33
Water	285 lbs	City
Air Entraining Agent	1.5 oz	Chryso Air 260, ASTM C -260

***Note:** Batch weights are based upon aggregates in Saturated Surface Dry condition.

Physical Properties

Slump:	7 in
Air Content:	9 %
Temperature:	68 F
Unit Weight:	133.4 pcf
W/C Ratio:	1.14
Yield:	1.00

Compressive Strength Data

Removability Modulus = 1.43

Age	PSI		
7 Day	**50**		
7 Day	**50**	**Ave:**	**50**
28 Day	**70**		
28 Day	**80**		
28 Day	**80**	**Ave:**	**80**

Bryon Blatter
Technical Services Manager

Reviewed by:
Castle Rock Consulting, LLC

◂ ON-DEMAND ▸ CONCRETE

1501 Backhoe Road, Loveland CO 80537
303-260-9544

CONCRETE MIX DESIGN REPORT

Class :	CDOT Flowfill	Design Compressive Strength, f'c	**50 min**
On-Demand Mix Number:	CDFF080C	Lab Mix ID	20074
Date Batched :	2/17/2020		
Date Mix Reported :	4/15/2020		
Use:	FlowFill		

Material	**per CY**	**Source / Type**	**S.G., SSD**	**Absorption%**
Cement	90 lbs	Holcim Type IP (25) HS, C595, Florence Plant	3.15	
Coarse Aggregate	1650 lbs *	Aggregate Industries, Morrison Quarry #57-67	2.66	0.8
Fine Aggregate	1300 lbs *	LG Everist, Firestone Pit C33 Concrete Sand	2.62	1.1
Water	360 lbs	City, ASTM C1602		
Water Reducer	oz**	Sika Sikament 686, Type A & F		
AEA	2.7 oz**	Siika AEA14		

* Aggregate weights based on SSD moisture, ** AEA & admix dosage may vary as needed with changing conditions.

Physical Properties		**Specifications**
Unit Weight:	125.9 pcf	- Not 138
W/CM Ratio:	4.00	
Slump :	8.25 in.	7"-10"
Air Content :	8.5 %	
Relative Yield	1.00	0.99 - 1.02
Removability Modulus, RE	**1.24**	Not 143

Compressive Strength Data

Age	PSI		
7 Day	44		
7 Day	47	Ave:	**48**
28 Day	77		
28 Day	63		
28 Day	83	Ave:	**71**

Kevin Sobczak
Quality Control Manager

Stan Peters, P.E.
Castle Rock Consulting

COLORADO LICENSED
STANLEY R. PETERS
26128
4-15-20
PROFESSIONAL ENGINEER

Production and delivery in accordance with ASTM C 685 Standard Specification for Volumetric Site Mixed Concrete. Concrete compressive performance is conditional with strict adherence to ASTM testing standards relating to concrete, and the lastest revisons of ACI 301 and 318

Attachment 4

Materials Properties Model of Aging Concrete

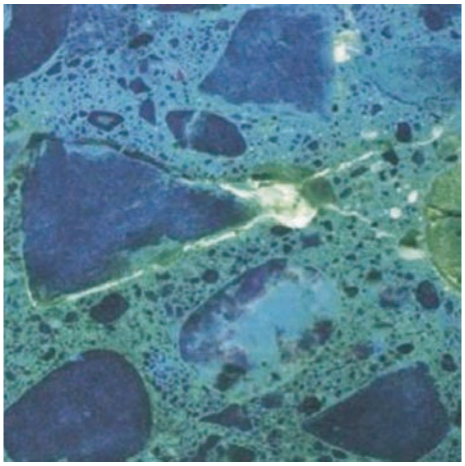
(DSO-05-05)

RECLAMATION

Managing Water in the West

Report DSO-05-05

Materials Properties Model of Aging Concrete



Dam Safety Technology Development Program

U.S. Department of the Interior
Bureau of Reclamation
Technical Service Center
Denver, Colorado

December 2005

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14. ABSTRACT
A database model of aging concrete was developed to identify the changes in materials properties over time for Bureau of Reclamation (Reclamation) mass concrete dams. Materials properties data on mass concrete were input to the Reclamation Aging Concrete Information System (ACIS). The data were analyzed for trends in the deterioration of concretes subject to aging, including alkali aggregate reaction (AAR) and general aging of early twentieth century concrete with high water-to-cement ratios. The aging concretes were compared to dams of similar age, but not suffering from aging processes. The aging concretes were also compared to known good quality concretes that were manufactured after about 1948 to specifically resist deterioration from AAR, freezing and thawing (FT), and sulfate attack. Trends were established for comparing the compressive strength, splitting and direct tensile strength, and elastic properties of aging and non-aging dams. The strength and elastic properties of aging mass concrete differed significantly from those of comparable non-aging concretes. Both spatial variations within a structure and long-term changes in strength and elastic properties were identified. The ACIS database can be used to track the long-term materials properties behavior of dams through comprehensive concrete coring and testing programs. Laboratory core test data are included for dams ranging from about 10 to more then 83 years old. These data provide the necessary supporting documentation for the Dam Safety Office Comprehensive Facilities Review evaluation process and for dams in need of corrective action.

15. SUBJECT TERMS
Concrete deterioration timeline; dam safety; materials properties model; aging concrete database; mass concrete; alkali aggregate reaction; alkali silica reaction; freezing and thawing; concrete core tests; compressive strength; splitting tensile strength; direct tensile strength; modulus of elasticity; Poisson's ratio

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Materials Engineering and Research Laboratory, 86-68180

Report DSO-05-05

Materials Properties Model of Aging Concrete

Dam Safety Technology Development Program
Denver, Colorado

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Mission Statements

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian Tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

Acknowledgments

The author would like to acknowledge the Science and Technology Program for providing the initial funding that led to the development of the aging concrete information system (ACIS) database and for documentation of the state of the art in Bureau of Reclamation mass concrete technology. These benefits have been invaluable in understanding those structures most in need of attention. The Dam Safety Office provided funding for the Materials Model of Aging Concrete and this final report. Dam Safety project funding was used to input much of the data on alkali-silica-affected dams and other aging dams currently under investigation. The Manuals and Standards funding allowed documentation of baseline materials properties of many of the Bureau of Reclamation's historic dams.

Several personnel from the Materials Engineering and Research Laboratory performed time-consuming, tedious data input including Erin Gleason, Veronica Madera, Jalena Maestas, and Kattie Bartojay. Lelon Lewis edited this report. Kurt von Fay began the initial work using Access programming software to develop the concept for the aggregate materials database, which provides an important resource to be coupled to the concrete properties database. He is also instrumental in the product development of concrete deterioration service life prediction software. Dr. David Harris and Dr. William Kepler provided the leadership for accomplishment of this program.

Carol Hovenden performed the Access database programming development and is largely responsible for the database organizational structure, operations and maintenance, and development of data reporting modules of the ACIS database.

Lastly, the author would like to thank the hundreds of Bureau of Reclamation field construction laboratory personnel, drill crews, and the Denver laboratory materials engineering technicians and engineers who have performed the backbreaking work to the perform concrete construction tests, and obtained and tested the hundreds of cores reported in the ACIS database.

Contents

Tables

Figures

Research Program Summary

The objective of this research program is to model the trends of deterioration of concrete in dams to better understand the processes, the rate of change in materials properties, and ultimately, provide the necessary supporting documentation for dams in need of corrective action. The destructive behavior of concrete deterioration is both a physical and chemical phenomenon of the cement paste, the aggregate, and the paste-aggregate interface. The development and reporting for aging concrete is funded under the Reclamation Dam Safety Research Program—Materials Model for Aging Concrete (DSO Project: AGING). The basis of the materials model for aging concrete presented in this report is to (1) identify the performance of concrete from Bureau of Reclamation (Reclamation) structures without ongoing deterioration and (2) compare with documented performance in structures presently undergoing deterioration. Structures without ongoing deterioration comprise the "baseline" materials properties, that is, what the long term properties of the concrete should be. Structures with ongoing deterioration comprise the "aging" properties of affected structures. From this comparative process, the changes in materials properties can be identified on affected structures and used to develop limits of unacceptable properties and requirements for corrective action.

The most pressing need for Reclamation's aging concrete structures is evaluating the changes in materials properties over time that affects dam safety. Current risk assessment and evaluation techniques take a "snapshot" of the dam performance under predicted loading conditions. For the most part, the condition of the concrete in the dam is assumed constant over time. However, if degradation is progressing over time, the dam condition is no longer constant. The aging structure may not be able to withstand the previously assumed loadings even if they have not changed.

This materials model is based on trends established from historic laboratory testing of Reclamation mass and structural concretes. This includes data from laboratory mixture proportioning studies, field quality control records, and core testing programs. Reclamation concretes of concern are typically from 50 to over 100 years old, constructed with much larger aggregates, and used different cements than modern-day concretes. The development of predictive modeling of concrete deterioration is an emerging technology. However, mass concrete, by virtue of its much larger aggregate sizes and different materials, does not yet fit the developing modeling technology. A materials model of aging concrete properties has been developed for dams through records of core tests entered in a comprehensive concrete database.

The benefits of development of an aging concrete materials database model include providing documentation for Reclamation Safety of Dams (SOD) and comprehensive facilities review (CFR) examinations and verification of predictive models currently under development by industry. The rate of deterioration with time is essential for accurate service life prediction. In addition, the data can be used to screen our concrete infrastructure to prioritize program funding for future rehabilitation efforts. The database has already been used to resolve outstanding Safety of Dams recommendations related to perceived decreased core strengths at Yellowtail Dam at a considerable cost savings compared to implementation of a new concrete coring and testing investigation.

Reclamation concretes are subject to wide variations in exposures and aggressive environmental degradation. Initially, Reclamation concretes were not resistant to environmental degradation processes such as sulfate attack, alkali aggregate reaction, and freezing and thawing damage. Reclamation has published more than 1,000 documents on concrete properties. Unfortunately, most of these data were published before the development of modern word processing and database technology. Extracting these data for every dam is laborious. By identifying relevant materials properties data in a relational database, trends of aging concretes in dams can be developed and compared to current structures of interest and is the focus of this research.

The aging concrete information system (ACIS) provides the necessary database of concrete materials for developing a model for concrete deterioration. Developed under the Reclamation Science and Technology (S&T) Program, ACIS is a powerful relational database of concrete materials properties from laboratory, field quality control, and hardened concrete core testing. ACIS ultimately has the capability for being linked to other existing databases currently associated with the dam safety program, such as the Dam Safety Information System (DSIS) and geographic information system technology. Data entry into ACIS has been accomplished through a variety of funded projects, including the Reclamation Science and Technology Program (primary database development), project funding (specific dam safety investigations), and the Reclamation Manuals and Standards funding (baseline of historic concrete materials properties).

Though the focus of this research program is dam safety related, similar benefits are applicable for the entire Reclamation water resources concrete infrastructure. Aging processes are affecting all Reclamation concretes but particularly those in sulfate environments and those in northern and mountain climates. Canals and associated water conveyance structures are particularly susceptible to aging-related concrete deterioration due to the lack of additional protective cover.

Conclusions and Recommendations

The properties of Reclamation dams differ significantly depending on the date of construction, geographical location and local deterioration processes, and the state of the art at the time of construction.

A comprehensive database has been developed to model the materials properties of Reclamation mass concrete. This database is capable of sorting and querying data specific to both individual dams and classes of aging concrete structures.

The materials properties of concrete incorporated into the ACIS database allow comparative modeling of the expected performance of our concrete dams with our aging structures.

Reclamation should continue to add relevant materials properties data for all concrete dam structures as these structures come up for review in the Dam Safety Office CFR program.

The strength and elastic properties of alkali-silica-reaction- (ASR) affected dams differ markedly compared to dams constructed with similar materials and mixture proportions. The ASR-affected dams have less than half the strength and elastic properties of comparable reference concretes.

The processes for ASR differ for some Reclamation dams and may be attributed to either the local materials used, the temperature environment at the dam site, or both. Parker Dam, constructed in Arizona, appears to have had early ASR reaction that has stabilized, whereas Seminoe Dam in Wyoming continues to deteriorate with time. Owyhee Dam in Idaho may have strength reduction trends similar to those at Seminoe Dam because of similar climates.

The aging concrete data should provide the necessary information to establish the ultimate or “terminal” strength parameters necessary for service life prediction models.

The concrete materials properties provide the necessary data support for analytical models used in structural analysis of our dams. This may also apply to additional dam-safety-related analysis under development.

Further research is needed to incorporate this concrete materials properties database with geographical information systems under development.

Aging Concrete Dams

Reclamation's Aging Concrete Infrastructure

More than half of Reclamation's infrastructure is more then 50 years old. The concretes used between 1902 through about 1948 in particular, were not purposely made to resist degradation from the environment. The three primary methods of concrete deterioration in our dams are (in order of specific identification and date of solution):

- *Sulfate attack*.—The chemical and physical destruction of the cement paste by aggressive, sulfate-laden waters (1937)

- *Alkali-aggregate (alkali-silica) reaction (AAR or ASR)*.—The chemical reaction between alkali compounds in cement with certain amorphous-silica-bearing aggregates, resulting in concrete "growth" by expanding silica gel (1942)

- *Freezing and thawing deterioration (FT)*.—The physical destruction of primarily cement paste by ice formation within the cement pores (1948)

A fourth mechanism specific mostly to Reclamation conveyance structures is corrosion of reinforcing steel. This is primarily related to pipelines or structures constructed accidentally with insufficient cover. However, when other mechanisms deteriorate surrounding concrete, corrosion may ultimately become the primary means of deterioration.

A Timeline for Reclamation Aging Concrete

Reclamation concrete development followed the established trends of the emerging state of the art of concrete technology in the twentieth century. Reclamation concrete is closely aligned with the development of materials properties technology for aggregates and cement, identification of and solutions for deterioration mechanisms caused by the environment and improvements in design and construction practices. These developments are summarized in figure 1. The construction of the early dams raised many questions for both designers and constructors. Concrete materials engineering developed as a means for solving problems at the materials science level. However, these materials science issues were interconnected to improvements in design and construction of major dams. Research in aggregates, cements, and materials properties were spurred on by Abrams in 1918 with his pioneering work in the design of concrete

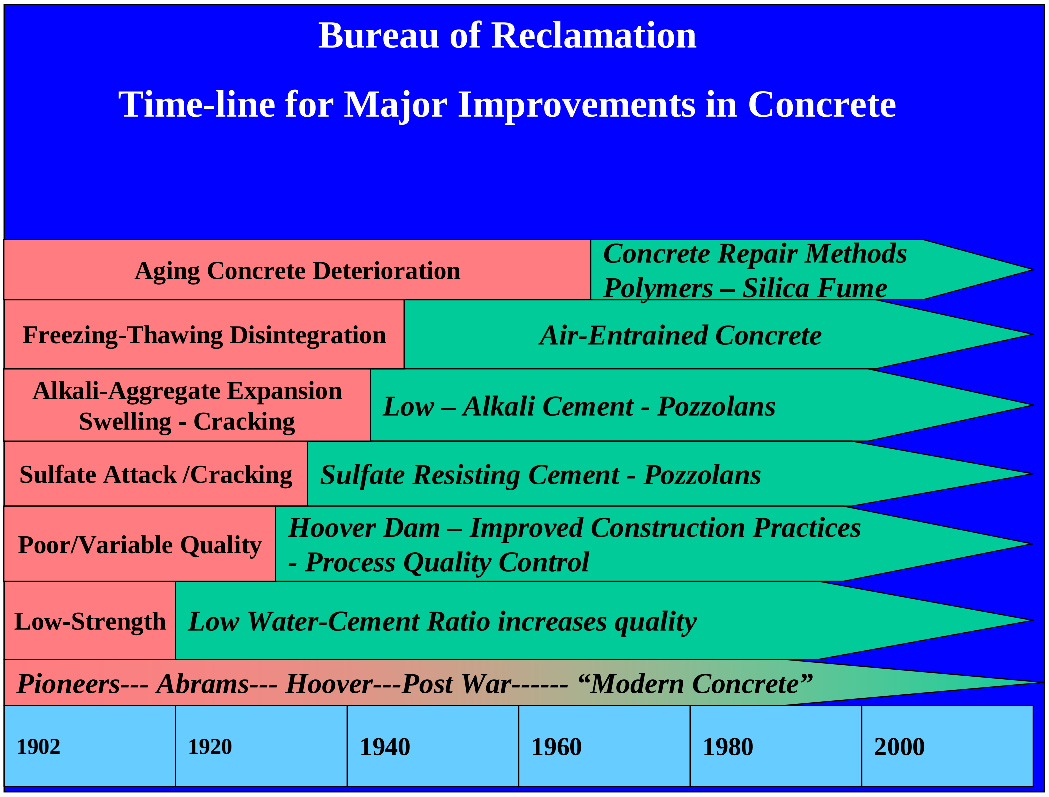

Figure 1.—Timeline for improvements in durable Reclamation concrete.

mixtures and his water-to-cement-ratio "law" (Abrams, 1918). The development of cement chemistry was spurred on in the late 1920s by the need to understand the chemical processes of hydration in order to reduce cracking from thermal heat generation for large dams and in particular Hoover Dam. The design and construction of Hoover and Grand Coulee Dams in the 1930s led to the development of concrete production on a massive scale, including improvements of concrete mixing, transporting, placing, and cooling. Close control of concrete quality led to reductions in the water and cement contents, yielding greater economy and more volumetric stability. The low-heat cements originally developed for Hoover Dam mass concrete also were found to resist deterioration in a sulfate environment. Subsequently, this materials science methodology became the foundation for the investigations in durability of concrete to resist sulfate attack, alkali-aggregate reaction, and freezing and thawing deterioration.

Trends of concrete materials properties have been developed for these different generations of concrete dam construction. Some dams have exceeded their expectations, while others have not. Comparing these concretes and their exposure conditions proved beneficial to discoveries of the necessary properties for durable concrete. For example, "sand cement" was developed by intergrading cement with finely ground sands and silts, thought to act as a pozzolan.

Unfortunately, this was later found not to be the case resulting in low strength concrete with poor durability in aggressive freezing and thawing environments. Exposed concrete at Arrowrock Dam in Idaho and Lahontan Dam in California required significant repair within 20 years and ultimately total rehabilitation of the service spillways whereas similar concrete used at Elephant Butte Dam in New Mexico was, for the most part, unaffected. The mixtures for Hoover and Grand Coulee Dams have proved superior in their respective environments compared to almost identical concretes constructed at the other locations with alkali-reactive concrete aggregates. The database of these concretes was useful for comparing the trends for ASR currently under investigation at Seminoe Dam and Parker Dam.

Concrete Deterioration and Dam Safety

The role of concrete deterioration in dam safety was documented in report No. DSO-03-05 titled *Effects of Concrete Deterioration on Safety of Dams* (Dolen, 2003). Processes of deterioration were identified specific to Reclamation concrete structures including concrete dams, embankment dams, and appurtenant works such as spillways and outlet works. Failure modes for concrete deterioration were developed for different types of Reclamation structures. One of the most difficult problems facing dam safety managers and engineers is predicting the remaining service life of structures known to be deteriorating after failure modes have been identified. If the concrete is actively deteriorating (such as in a concrete dam), the probability of failure due to the same event will gradually increase as the dam properties degrade until the risks are no longer acceptable. If there is no active deterioration, the risks essentially remain the same unless other modes of failure develop due to changes in loading conditions from flooding or seismic events.

Concrete Materials Properties Investigations and the Aging Concrete Information System

The CFR evaluation process for Reclamation concrete dams typically looks at the original design and performance under the anticipated loading conditions. As a part of this process, the materials properties of the dam are evaluated based on the original tests, if available, and any subsequent postconstruction test programs. The engineer must understand both the expected properties and the actual performance of the field mixtures used in the dam. The ACIS was developed to facilitate this process.

Construction of a major concrete structure involves a set of deliberate steps to optimize the most economical mixture for the strength under assumed loading conditions and for overall construction placement and quality. A comprehensive concrete mixture proportioning program produces perhaps thousands of data records to meet these needs. Reclamation has developed a comprehensive testing and evaluation program for mass concrete. The testing needed to optimize a mixture for construction includes the following data sets:

- *Materials source field investigations*.—Perform many evaluation tests on several sources of aggregates and cementitious materials leading to selection of candidate testing of concrete mixtures.

- *Laboratory concrete and materials investigations*.—Perform many tests from different sources of materials:

    - Aggregate quality tests (many tests from several samples)

    - Concrete workability and aggregate proportions tests (several tests on many mixtures)

    - Concrete strength optimization tests (several tests on many mixtures)

    - Select candidate mixtures for final materials properties (many tests on a few mixtures)

    - Recommendation of optimum concrete mixtures for construction

- Construction concrete mixture investigations

    - Trial batches after aggregate processing (several tests of a few mixtures)

    - Select final mixture proportions to begin construction

    - Construction quality control testing (several tests from many batches of a few mixtures)

    - Redesign of mixtures (if needed for strength and economy)

Following construction, periodic core testing is performed to confirm the assumed properties of the design are achieved, to evaluate possible construction related defects, for periodic monitoring, and if necessary, to answer specific materials properties questions. Postconstruction testing may include the following data sets:

- Postconstruction testing:

    - *Confirmation core testing*.—Materials properties (many tests from a few locations)

- ◦ *Confirmation core testing*.—Construction defects (few tests from a few specific locations)
- ◦ Periodic core testing (few or many tests from a few locations)
- ◦ Concrete deterioration core testing (few or many tests from a few or many locations)

The development of a concrete materials properties database must identify the source of the materials and associated mixtures tested. Laboratory tests establish a baseline of expected performance under standardized laboratory conditions, but may not represent the specific concrete mixtures sampled by core tests. Field quality control tests provide short term (typically 7 days to 1 year or less) strength properties of the actual field mixture and may or may not represent the full mass mixture. Core testing provides either short term or long term materials properties, and the core samples may vary with core diameter and location within the structure. The ACIS program is therefore divided into four distinct, but interconnected modules:

- Materials sources identification for either the laboratory samples or actual construction materials
- Laboratory mixture proportions and physical properties test results
- Field mixture proportions and quality control test results
- Postconstruction concrete core test results

Due to the broad scope of possible materials and mixtures, a specific dam safety investigation and data analysis most often focuses on the actual mixtures used in the dam and past test results. Laboratory materials properties used for the initial design can serve as an important historical record in the absence of construction and postconstruction records, when needed. The construction data records may be generalized, representing average test properties from the entire dam or they may track the day-to-day records for essentially different placements of the same basic mixture. ACIS has the flexibility to include both circumstances for data storage and reporting. An example of comprehensive core test records for Yellowtail and Parker Dams is included in the appendix.

ACIS and Materials Model for Aging Concrete

Concrete Deterioration Model for Dams

The goal for modeling the behavior of deteriorating concrete dams is to predict their remaining service lives. There is no deterioration model specifically developed for Reclamation mass concrete structures and particularly those structures constructed in the early twentieth century. Deterioration can be evaluated by comparative modeling coupled with predictive process modeling. This research program used comparative modeling, either by comparing good concrete to bad or by comparing accumulated data over time. Predictive service life models need verifiable performance. Typically, a model is developed, laboratory mixtures are tested under simulated (usually accelerated) conditions to calibrate the model, and finally the calibrated model is applied to the structure in question, often a new structure. This type of predictive process modeling has limited use for Reclamation concretes unless their output can be verified by historic performance. Reclamation has a unique role in modeling concrete behavior since our structures have 50 to 100 years of verifiable performance for calibration of predictive models. Reclamation also possesses a wealth of laboratory and field testing records to back up the documented performance. Reclamation was at the forefront of development of concretes to resist the aggressive environments beginning as early as 1928 when investigations were begun for the construction of Boulder/Hoover Dam. The combination of robust predictive modeling coupled to a comparative model of verifiable data and performance will provide a great leap forward in dam safety service life prediction worldwide. The materials properties trends for aging concrete are obtained by sorting and filtering relevant data from the ACIS database to compare the properties over time. The ability of ACIS to search and sort testing data records efficiently allows specific processes to be examined for many structures, or historical trends for specific structures.

Reclamation also has a good body of materials properties for structures of varying age that have not undergone deterioration with time, forming the database of “good concrete” for comparative purposes. This can be used to identify the projected “state of condition” for comparison with other structures of concern. For example, Hoover Dam, constructed from 1933 to 1936, used essentially the same state of the art from a concrete technology standpoint as Parker Dam, located about 60 river miles downstream and constructed from 1937 to 1938. Both were constructed with the same construction practices and concrete quality control programs, and used the same cement from plants located in California.

Parker Dam was constructed by one of the “seven companies” that built Hoover Dam, used the same equipment for concrete production, and likely even used some of the same personnel. Hoover has little deterioration of any kind, whereas Parker was the first dam identified in Reclamation’s inventory to suffer from alkali aggregate reaction, specifically alkali-silica reaction. The same comparison exists for Seminoe Dam, using mixtures and cements similar to those used at Grand Coulee Dam. Grand Coulee concrete has performed quite well over time, but Seminoe has been suffering from extensive deterioration from ASR combined with freezing and thawing deterioration. Once the predicted performance is identified from the good concretes, the deteriorating concretes can be evaluated for spatial and time-dependent changes.

Strength and Elastic Properties of Aging and/or ASR-Affected Dams Compared to Unaffected Dams

Averaging and Sorting of ACIS Test Data

The ACIS materials properties database is comprised of thousands of data records. These data were organized using Microsoft Access and Excel software. Querying is best performed using Access, and data analysis of the queried data was performed with Excel. The data input to ACIS allows entering both the average of several tests from one core program or individual drill holes, and individual tests. Reported average values are simply the average of the data set in question, such as core test age. Many of the data in the tables report the “weighted average” based on the actual number of tests performed. Thus, data represented by the average of 30 tests hold more weight than only one test for computing the overall, weighted average. For example, the weighted average compressive strength of mass concrete cores without aging is 5,590 lb/in^2, based on 227 tests, whereas the average without weighting is 5,160 lb/in^2. The data averages may also report the most current representative test data for dams that have had multiple test programs, where noted.

Compressive Strength and Elastic Properties Development of Aging Dams

Compressive strength and elastic properties of dams not subject to ASR were studied to determine changes in materials properties over time. The compressive strength development of the entire data set (not subject to ASR) showed possible “anomalous” trends as shown in figure 2 and table 1. Dams from early structures in the overall data set include East Park Dam, constructed in 1910. This 83-year-old concrete does not appear to have extensive deterioration. However, concretes from this era had higher water-to-cement ratios and do not have the ultimate strength potential of the later dams represented by Hoover Dam at 60 years of age. The average strength and elastic properties of the aging, pre-1920s concrete dams and post-Hoover “modern” dams are summarized in table 1. The long term compressive strength and elastic properties of the “modern” mass concrete dams—essentially the post-Hoover era dams are based on the most recent core

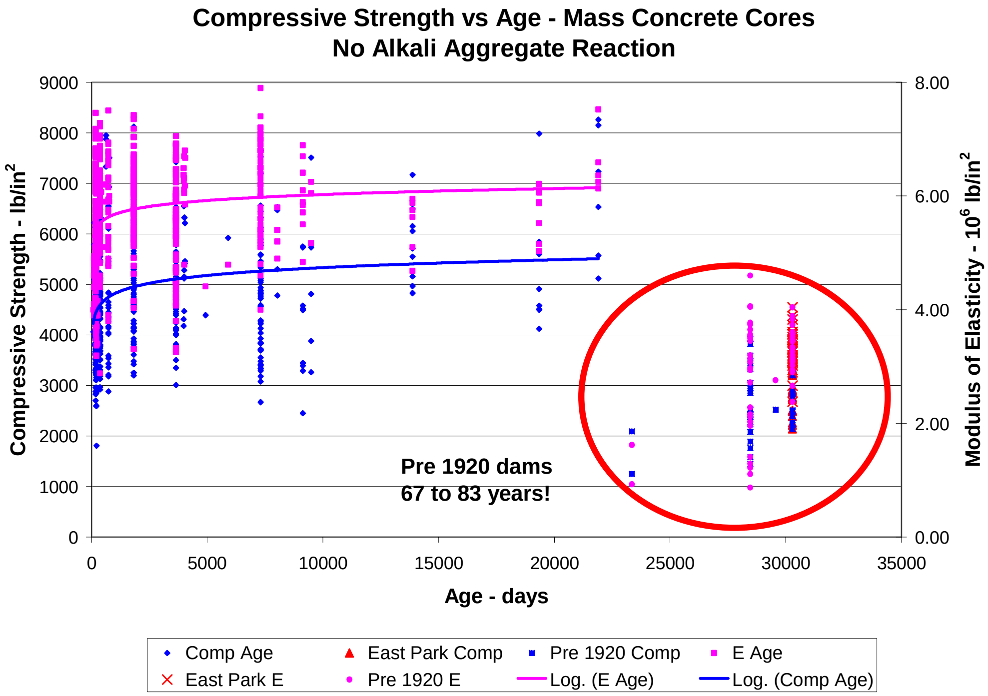

Figure 2.—Compressive strength development of concrete dams not subject to ongoing deterioration.

Table 1.—Average compressive strength and elastic properties of cores from Reclamation concrete dams not subject to aging compared to aging concretes constructed in the early twentieth century

	Average test age, days (yr)	Compressive strength, lb/in^2	Modulus of elasticity, 10^6 lb/in^2	Poisson's ratio
Concrete dams (0 to 60 years old) *	10,418 (28.5)	5590	5.42	0.18
East Park Dam	30,295 (83)	2980	3.32	0.21
ACIS aging dams (1902 to 1920) *	29,100 (79.7)	2490	2.59	0.23

* Average is weighted for number of tests for a given sample set.

tests available for each structure. The old dams constructed prior to about 1920 have less than half the compressive strength and modulus of elasticity of comparable "modern dams" of the post-Hoover era. The pre-1920 dams form a separate data set for CFR evaluation purposes. These dams are the most vulnerable to concrete degradation and may require special precautions during modifications. For example, the low compressive strength of exterior mass concrete required longer reinforcing steel embedment lengths during recent outlet works structural modifications. The data then trends from the low to higher strengths in the 1920s, though some dams perform better than others. The "modern" concrete dams generally begin in the late 1920s or early 1930s, provided no other destructive mechanisms are in progress such as ASR or FT.

Compressive Strength and Elastic Properties Development of ASR-Affected Dams

Three Reclamation concrete dams have suffered from significant deterioration attributed to AAR, and in particular, ASR. Parker Dam, constructed in 1937 to1938, is located on the Colorado River about 60 river miles downstream of Hoover Dam and was the first Reclamation dam to be identified with ASR. American Falls Dam, constructed in 1927, was actually the first Reclamation structure to suffer from ASR, and it was ultimately replaced in 1977. Seminoe Dam was constructed in 1938 and has gradually experienced deterioration over time (Mohorovic, 1998). Both Parker and Seminoe dams have comparable "reference" dams constructed with similar materials and mixtures at about the same time frame with little deterioration (Dolen, 2006). The primary difference in the performance of the ASR-affected and reference dams lies in the cement alkali content, and/or aggregates used for construction. The Colorado River aggregate source used at Hoover Dam is essentially "nonreactive" and the Bill Williams River aggregate source at Parker Dam is very reactive. Evaluation of the data shows that Parker's materials properties have not realized the same performance as Hoover Dam over time even though they used the same suppliers of type IV cement and had comparable mixtures. The concrete has about 60 percent of the compressive strength and modulus of elasticity and has shown little strength development over time.

This same comparative relationship exists between Grand Coulee Dam, constructed between 1933 and 1942, and Seminoe Dam, constructed in 1938. Due to the superior durability of the Grand Coulee materials, many laboratory test results were compared to or duplicated for other mix design investigations from that era. Modified (low heat) Type II cement, developed first for Grand Coulee Dam, was also used for the Seminoe testing program. However, ASR was unknown at the time of these tests, and possible decreases in strength and elastic properties for mixes with reactive aggregates were not investigated at the time. In retrospect, ASR may have been detected through close examination of these results or if long term tests had been performed.

Concrete dams affected by ASR are probably the most studied dams in the Reclamation dam inventory. Both Parker and Seminoe Dams have been cored and tested periodically for concrete degradation. American Falls Dam was studied extensively prior to replacement in 1977. Deterioration at Parker Dam due to ASR was identified within 2 years of construction. Damage from ASR at Seminoe Dam was not attributed primarily to ASR until the late 1990s, more then 50 years after construction. Six-inch diameter cores have been obtained and tested for strength and elastic properties and contribute to the database of ASR in mass concrete. Both the strength properties and condition of the concrete were analyzed by selective sorting in ACIS for ASR. Average strength and elastic properties of the ASR-affected and aging dams are summarized in table 2.

Table 2.—Average compressive strength and elastic properties of concrete dams subject to aging.

	Test age, days (yr)	Compressive strength, lb/in^2	Modulus of elasticity, 10^6 lb/in^2	Poisson's ratio
ASR-affected dams *	19,367 (53.1)	3695	2.28	0.20
ASR cores from the top 20 ft	17,512 (48.0)	3180	2.09	0.20
ASR cores from below 20 ft	17,888 (49.0)	4090	2.35	0.10
ACIS aging dams (1902 to 1920) *	29,095 (79.7)	2490	2.59	0.23

* Average is weighted for number of tests for a given sample set.

Strength and elastic properties were examined for ASR-affected dams compared to dams without ASR. The sorting can be performed on individual structures or for all structures by changing the querying properties. Figure 3 shows the compressive strength development of mass concrete cores with and without ASR. It is interesting to note the high compressive test results in figure 3 at 42 years (15,330 days) age, which were identified as coming from mass concrete placed in the lower portion of Parker Dam. Four different sources of cement were randomly delivered early in the construction of Parker Dam and one source of cement met the criterion for low-alkali cement. The high strength test results were from deep cores tested at the base of the dam in 1980 and may represent the unaffected concrete where the low-alkali cement was used. If so, these tests may represent the potential strength of Parker Dam if low-alkali cement had been used for all construction. Some other deep cores did not achieve the higher strengths and may represent placements that used high-alkali cement.

Data from Friant Dam provide a good comparison of the effects of ASR (Hartwell, 1990). Mass concrete was placed using both high and low alkali cements and with or without 20 percent pozzolan. The average compressive strength of ASR-affected mass concrete (high-alkali cement, no pozzolan) at 46 years age is about 3,220 lb/in^2, the modulus of elasticity is about 1.7 x 10^6 lb/in^2, and Poisson's ratio is 0.38. Tests from similar concrete with high-alkali cement and no pozzolan at 4 years age averaged about 6,760 lb/in^2, 6.0 x 10^6 lb/in^2, and 0.22, respectively. The average compressive strength decreased about 57 percent, and the average modulus of elasticity about 72 percent due to ASR. Tests performed on mass comparable concrete that used low-alkali cement plus 20 percent pozzolan showed no decrease in between 4 and 46 years age.

For some mass concrete dams, the strength and elastic properties also vary spatially with depth below the top of the dam. The tops of dams have less restraint and are more likely to expand and deteriorate. Table 2 shows average

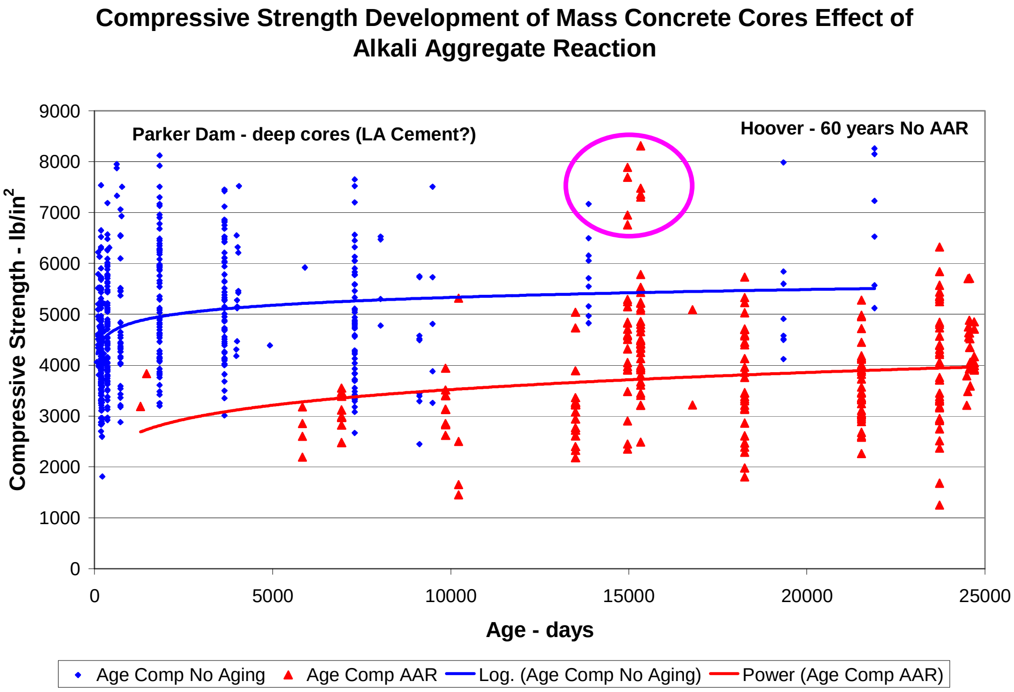

Figure 3.—Compressive strength development over time for mass concrete dams with and without ASR. The data represent tests from different dams.

compressive strength and elastic properties of ASR-affected cores sorted by depth below the top of drill holes, essentially the top 20 feet of these dams. The average compressive strength of cores from the top 20 feet of these drill holes is about 3,180 lb/in^2, compared to 4,090 lb/in^2 for cores tested more than 20 feet below the top of the drill holes, and 5,590 lb/in^2 for non-ASR-affected mass concrete. The average modulus of elasticity changed from 2.09 x 10^6 lb/in^2 to 2.35 x 10^6 lb/in^2, for cores tested above and below the 20-foot depth. These data compare with the non-ASR-affected concrete modulus of about 5.4 x 10^6 lb/in^2.

The decrease in modulus of elasticity is more apparent than compressive strength in ASR-affected dams for both early and long term ages. This can lead to apparent "low stresses" using conventional linear elastic structural analysis. However, these analyses should be used with caution as the behavior may be best represented using nonlinear analysis.

Figure 4 shows the relationship between compressive strength and modulus of elasticity for all concrete cores with and without ASR. Although the correlation coefficient for the equations is poor, the trend lines are added to show the demarcation between the two classes of concretes. Individual correlations between compressive strength and modulus of elasticity are normally much better for individual dams using the same aggregate types. The trends show that the strength to modulus of elasticity relationship is a good indicator of ASR and may be used in developing failure criteria for predictive models.

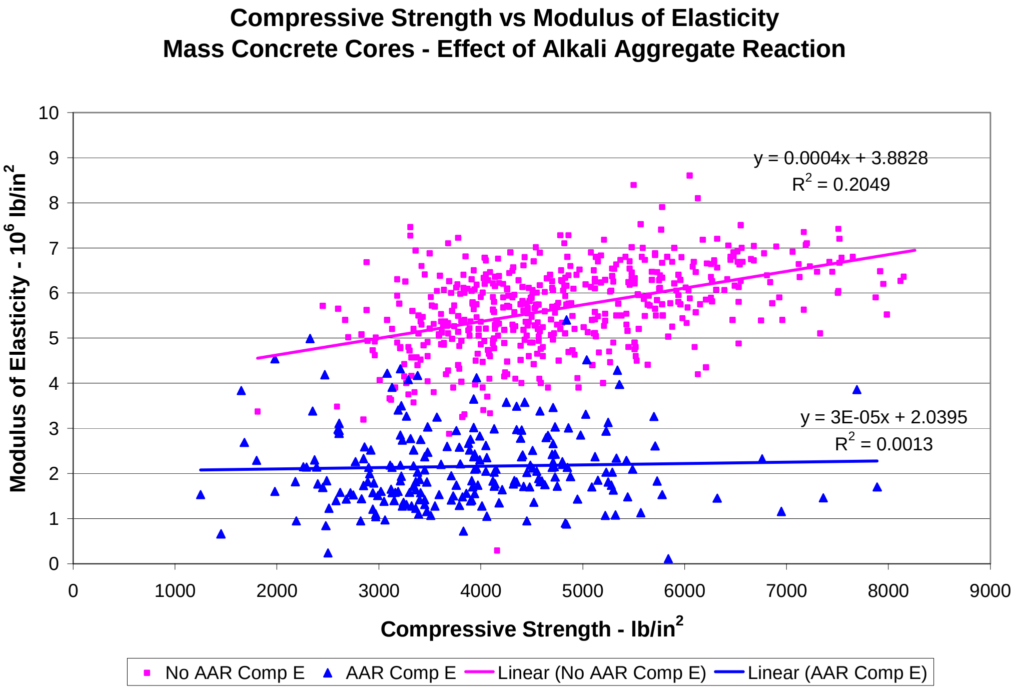

Figure 4.—Comparison of strength to modulus of elasticity in compression for mass concrete dams with and without ASR.

Tensile Strength Properties of Aging/ASR and Non-ASR-Affected Dams

Tensile strength is becoming more critical in the structural analysis of concrete dams, particularly for dynamic analysis due to earthquakes. Tensile strength tests were normally not performed until the 1970s, and the tensile strength development for dams constructed prior to this era is unknown. The results of direct and splitting tensile strength of good quality concrete and aging/ASR-affected concrete and are shown in table 3 and figure 5. The tensile strength data are entered in the database as average values for normally only a few tests for each mixture. The aging data also include some tests of old dams not subject to ASR. However, it is clear that the tensile strength of aging/ASR-affected dams averages about 50 percent of the direct tensile strength and 30 percent less in splitting tensile strength compared to dams without ASR degradation or aging. Also, the aging concrete data are often based only on “testable” concrete and do not represent the condition of the deteriorated concrete that could not be tested. Lift line ratios may not be directly comparable since the aging dams often have more disbonded lift lines. This input parameter is being added to more recent test programs and is a factor for some older and newer dams. Shear bond properties are not shown for this data set and have not yet been analyzed due to insufficient records.

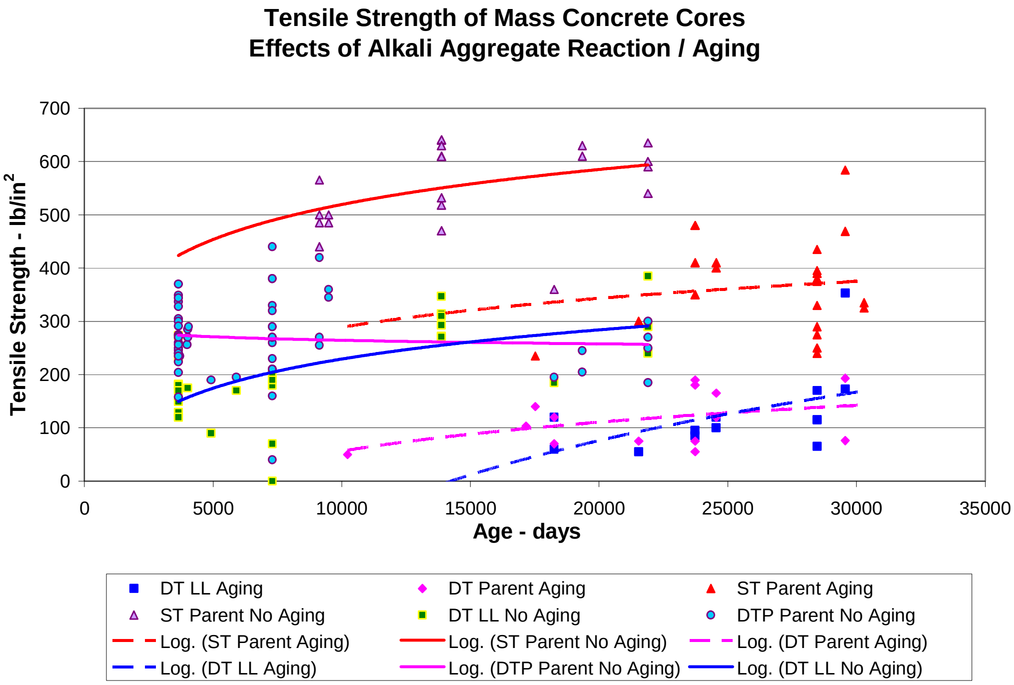

Figure 5.—Comparison of the effects of aging and ASR on tensile strength of mass concrete dams.

Table 3.—Effect of aging on tensile strength of mass concrete cores expressed as a percentage of average compressive strength,[1] based on data from the ACIS concrete materials database

	Tensile strength, lb/in^2 (%)[2]	
	No aging[3]	With aging[3]
Direct tensile strength (parent concrete)	245 (4.4)	105 (3.1)
Direct tensile strength (lift lines)	185 (3.3)	115 (3.4)
Splitting tensile strength (static)	520 (9.3)	365 (10.9)
Splitting tensile strength (dynamic)	745 (13.3)	420 (12.6)

[1] Average is weighted for number of tests for a given sample set.
[2] (%) Tensile strength expressed as a percent of comparable compressive strength.
[3] Average core test age for no aging dams is 10862 days (30 years); average age for aging dams is 25931 days (71 years).

Applications of Materials Properties Modeling

Strength Trends at Parker and Seminoe Dams

The strength trends at both Parker and Seminoe Dams have been studied extensively for the effects of ASR. Cracking in Parker Dam was identified as ASR after examinations confirmed the process first identified in 1942 (Stanton, 1942). Extensive concrete coring and testing have been performed since 1940, and the strength trends are well documented. Seminoe Dam suffered from early freezing and thawing near the dam crest, but ASR was not identified as a significant contributing factor to degradation until more then 60 years after construction. The deterioration at Seminoe Dam seems more alarming because the mass concrete appears to have nearly reached its projected ultimate strength potential before the onset of ASR. The slow rate of reaction may be due to the nature of the aggregates and the cold temperatures at the site. Another northern climate dam with the potential for similar behavior is Owyhee Dam, in Idaho. Tests near the crest of Owyhee Dam are revealing behavior similar to that at Seminoe Dam, and potentially reactive aggregates are prevalent in the vicinity of the dam.

From a comparative standpoint, Parker Dam concrete mixtures, cements, and construction methods are almost identical to those for Hoover Dam, the primary difference being that Hoover Dam used primarily non-reactive aggregates from the Colorado River, and Parker Dam used reactive aggregates from the Bill Williams River. The Type IV cement developed for use in Hoover Dam was also used for Parker Dam. In fact, some of the concrete manufacturing equipment used for Hoover Dam was transported directly to Parker Dam. Many of the Reclamation field staff and contractor personnel likely came from Boulder City. One key piece of equipment not used at Parker Dam was the cement blending plant. Several different sources of cement were used in the dam, resulting in spatially varying strength and elastic properties due to individual shipments with differing alkali contents. The performance of both dams has been reported extensively, and thus, comparison of these dams shows the change in materials properties attributed to ASR. Looking more closely at Parker Dam, concrete core results reveal spatial relationships, with high strength concrete in some sections in the bottom of the dam similar to Hoover Dam concrete, and poorly performing concrete in the upper portion of the dam. As previously mentioned, it is suspected that these tests represent unaffected concrete where the low-alkali cement was supplied to the dam. Some Type IV cement was used early in construction of Grand Coulee Dam, for which core tests at 1 to 3 years were available. The mass concrete core tests show exceptional compressive strength exceeding 7,000 lb/in^2.

Both laboratory and field data were compared for these three dams. Figure 6 shows results of compressive strength tests over time and the difference in strength gain expected (Hoover and Grand Coulee Dams) compared to the actual results at Parker Dam. The Parker data at 1 through 90 days of age are the average results of construction quality control cylinders, and the rate of strength gain compares favorably to laboratory trends. The core test results are shown only at 67 years of age to compare to the Hoover 60-year tests. Also interesting to note are the laboratory compressive strength results from 1935 using the Parker cement (supplied by the Metropolitan Water District)for both Bill Williams aggregate and Brett Pit aggregate shown in figure 7. This comparative testing was often done during the early mixture design studies conducted in the 1930s.

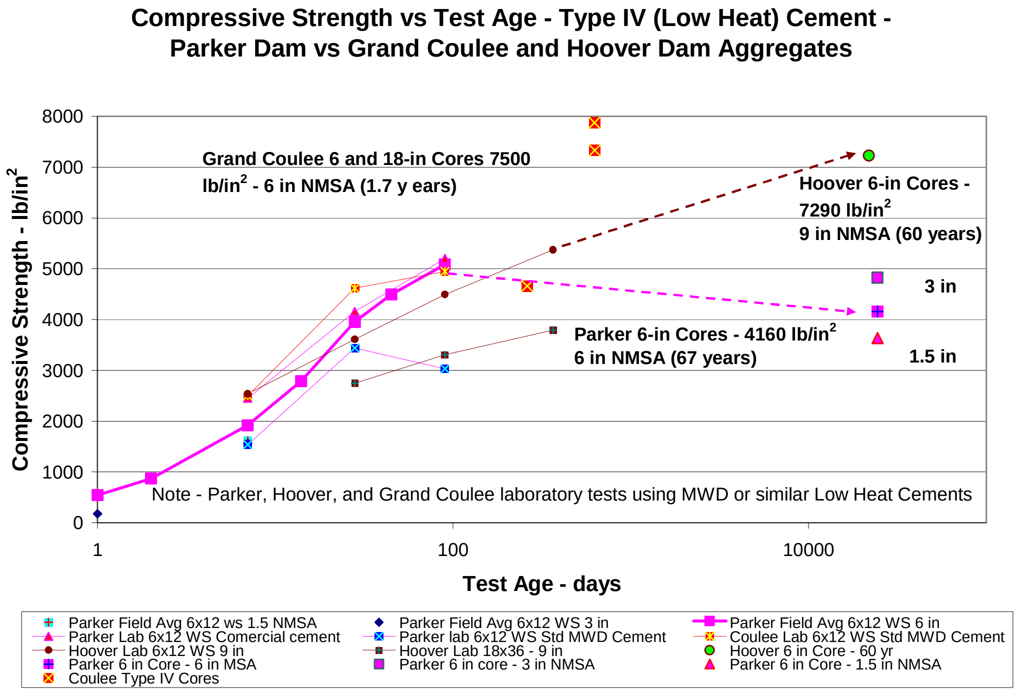

Figure 6.—Compressive strength development of mass concretes with Type IV cement for Parker Dam, Hoover Dam, and Grand Coulee Dam.

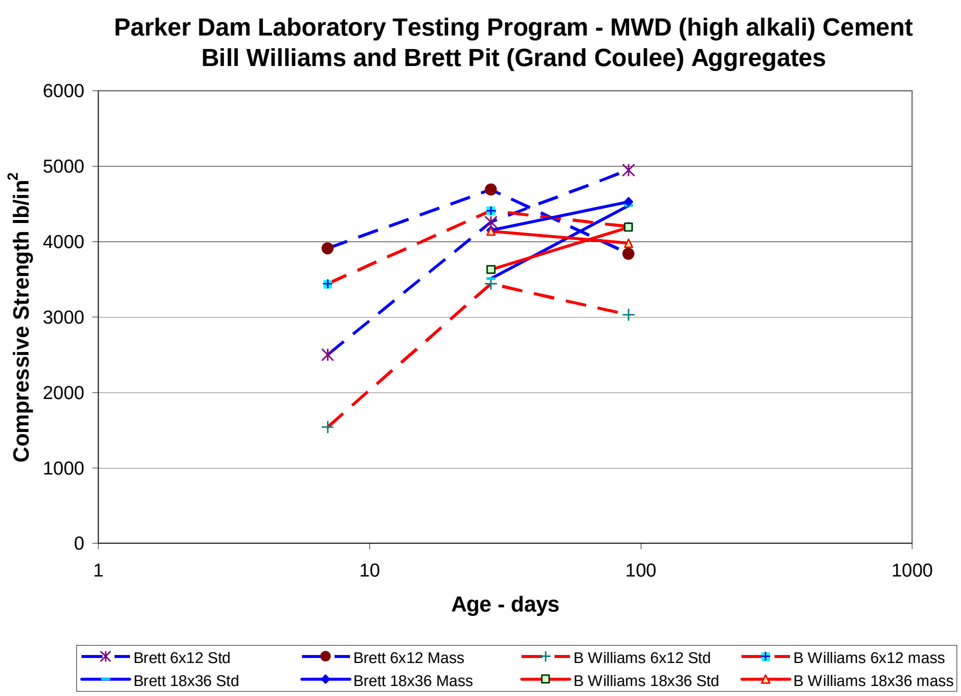

Figure 7.—Comparison of compressive strength development of laboratory concrete mixtures using Parker Dam cement with Parker (Bill Williams) and Grand Coulee (Brett Pit) aggregates.

Three of the four curing conditions with the Parker aggregates decreased in strength between 28 and 90 days. Only one of the four conditions for the Brett aggregate had a decrease in compressive strength between 28 and 90 days. Tests were not conducted beyond 90 days in the Parker mixture design studies because it is an arch dam. These laboratory tests may have been an unidentified precursor of the ASR that would attack the dam once it was constructed.

The compressive strength trends in figure 8 from the two ASR-affected dams show a relatively constant state for Parker Dam and a decreasing strength trend with Seminoe Dam. Some of the data scatter is due to the overall sampling not sorted by elevation and includes tests of concrete not significantly affected by ASR either due to the cement alkali content of individual block placements or location in the dam. When sorted by elevation, the rate of change can also be observed for the two dams as shown in figures 9 and 10. The compressive strength trends do not show an overall change with time for Parker Dam, even though some spatial trends may be present. For Seminoe Dam, it is readily apparent that the overall compressive strength is decreasing over time and that the compressive strength has significant spatial deterioration near the top of the dam as shown in figure 10. The deterioration is extending more deeply into the dam over time. The modulus of elasticity also shows the same trends as shown in figure 12. If ASR is suspected in dams, the compressive strength to modulus of elasticity ratios and spatial orientation may provide the best supporting documentation for CFR evaluation purposes.

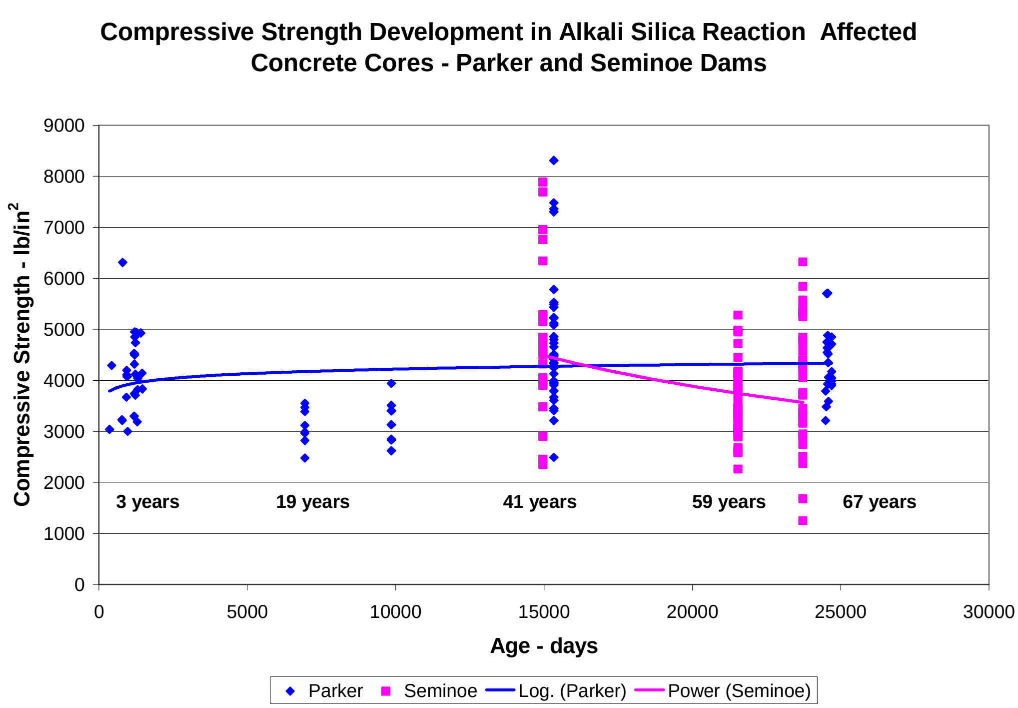

Figure 8.—Compressive strength trends for mass concrete cores at Parker and Seminoe Dams.

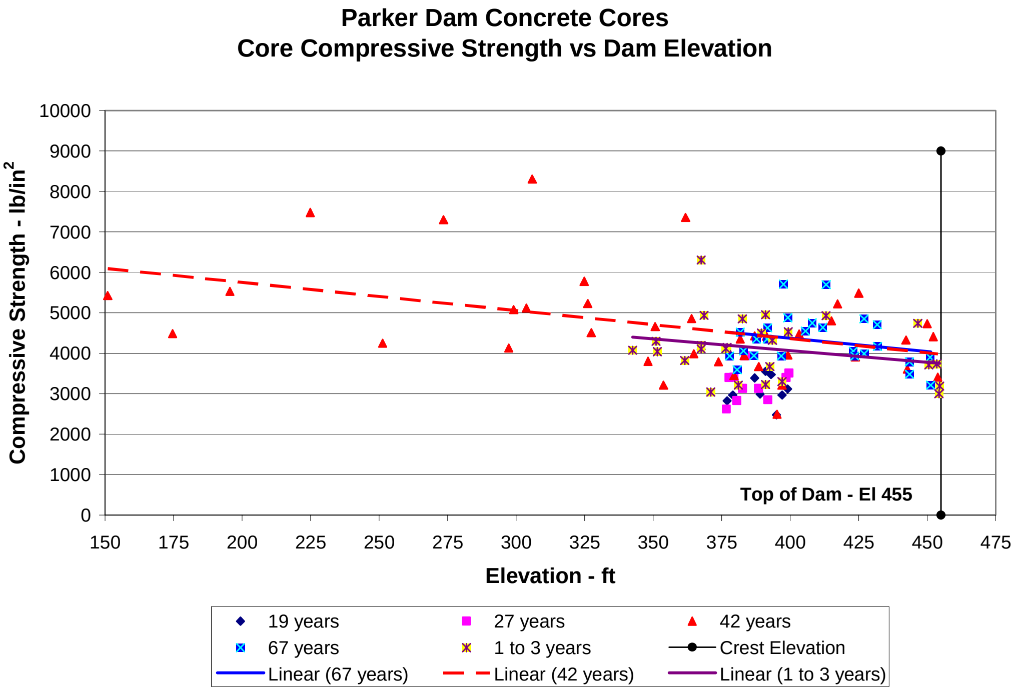

Figure 9.—Compressive strength of mass concrete at Parker Dam, Arizona sorted by elevation within the structure. The top of the dam is at elevation 455.

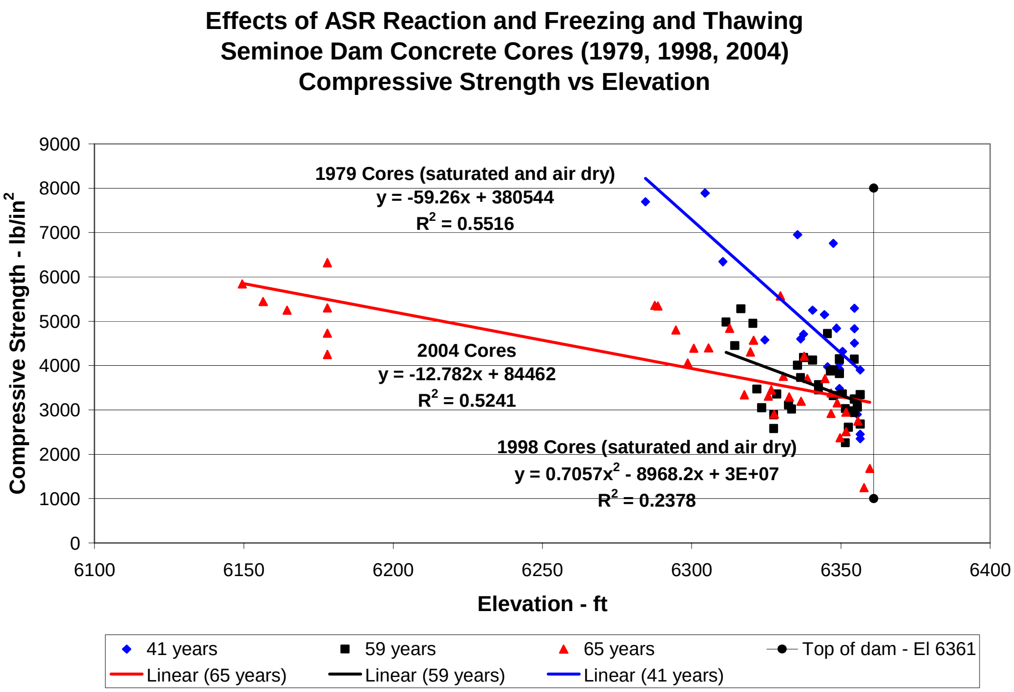

Figure 10.—Compressive strength trends of mass concrete at Seminoe Dam, Wyoming, sorted by elevation showing changes in strength over time.

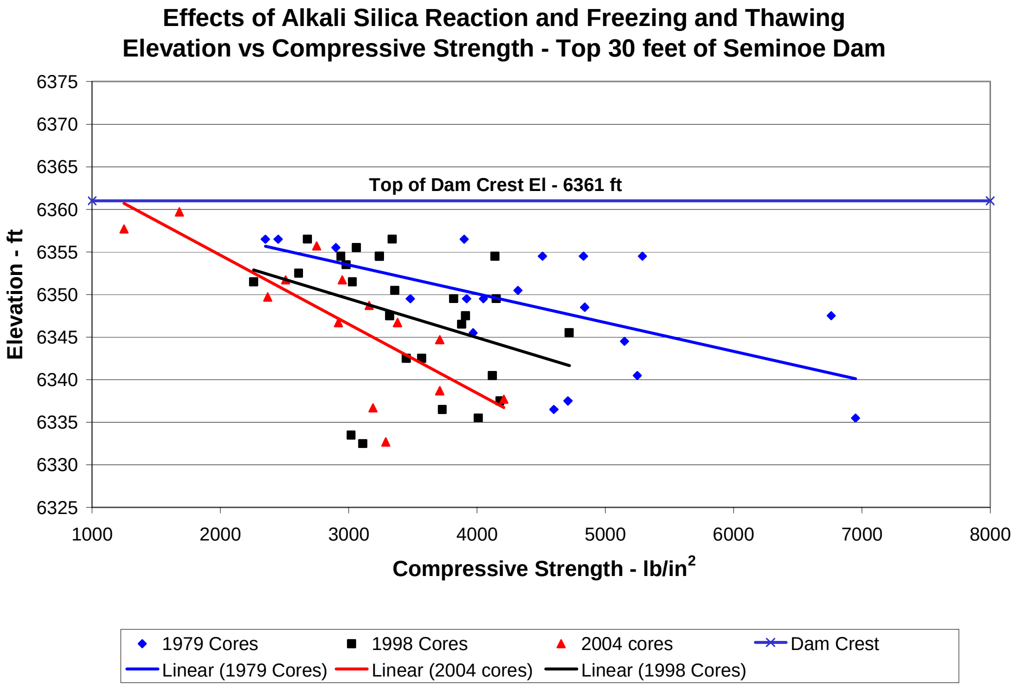

Figure 11.—Compressive strength development in mass concrete at the top of Seminoe Dam, Wyoming, sorted by elevation showing decreasing strength over time.

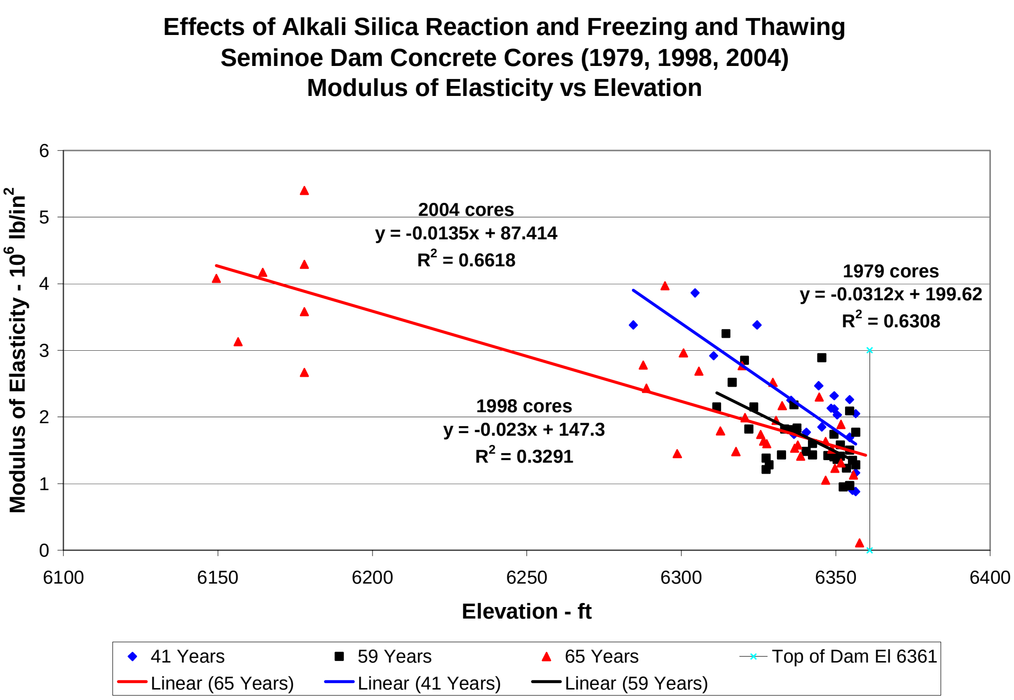

Figure 12.—Modulus of elasticity in compression trends of mass concrete at Seminoe Dam, Wyoming, sorted by elevation showing changes in modulus over time.

Yellowtail Dam Issue Evaluation

The Yellowtail Dam issue evaluation presented a unique opportunity to use the ACIS database to examine strength trends to resolve an outstanding dam safety recommendation. Yellowtail Dam is a concrete thick arch structure approximately 525 feet high located about 45 miles southwest of Hardin, Montana. Mass concrete in the dam was placed in 1963, 1964, and 1965. Four mass concrete mixtures with 6-inch nominal maximum size aggregate (NMSA) were used in the dam. This included “interior concrete” (the primary mass mixture) and “exterior concrete” with a higher cementitious materials content for increased durability. The cementitious materials content was decreased in July 1963 after high compressive strengths were recorded from control cylinders cast early during construction. The remaining concrete construction was completed with revised mixtures and purposely lower ultimate compressive strengths. Thus, four potential mixtures could be sampled, each with differing materials properties. Ten-inch diameter concrete cores were extracted from the dam from the control cable gallery (elevations 3185 and 3207) and the filling line gallery (elevation 3462) for periodic testing at 6 months, 1 year, 5 years, 10 years, and 25 years of age (Graham, 1969). During the 2001 Comprehensive Facility Review, a cursory summary of the results from previous core programs showed apparent anomalous behavior in properties between 10 and 25 years after construction. Specifically, compressive strength, modulus of elasticity, and Poisson’s ratio showed a relatively high variability, and an apparent decrease in strength was recorded between 10 and 25 years, resulting in the following SOD recommendation.

> 2001-SOD-B—Sample and test the concrete at Yellowtail Dam to determine the strength properties and compare to past tests.

A detailed examination of the results of all core tests was performed using ACIS. Individual tests were entered to determine strength trends related to the core tests spatially by core location, test age, and depth (Dolen, 2005). The results of compressive strength and elastic properties from this analysis are summarized in table 4. Although the overall behavior showed decreasing strength, the results of a detailed examination revealed spatial variability between the different mixtures placed in the dam and additional variability due to different (vertical) lifts placed within individual blocks for the same mixtures placed on different dates. The apparent decrease in compressive strength was likely attributed to variability of tests performed at different locations (and with different mixtures) and to the different moisture conditioning of cores tested at 10 years of age. Spatial variability was identified for the same concrete mixtures within individual blocks sampled from lift to lift. This may be due to concrete mixture variability during construction, core test variability, or within lift variability for the 7½-foot deep lifts. When “apples and apples” core tests were compared, the lower strength tests were identified in concrete not previously tested, and some of the confusion of test evaluation was due to comparing concrete mixtures before to those after the cementitious materials were decreased.

Table 4.—Compressive strength of 25-year cores compared to reference core tests by spatial orientation—Yellowtail Dam issue evaluation—Yellowtail Dam, Montana

			Compressive strength, lb/in²					
Mix	Drill hole	Elevation	6 mo	1 yr	5 yr	10 yr[1]	25 yr	Percent 1 yr
INT9/1963	18-13-V	3179.8	4460	6310	6660	7520	7510	119
INT/9B1963	18-13-V	3176.6	*No comparable data for this lift*				4810	
INT6/1963R	10-9-V	3204.5	4100	4400	5810	6550	5730	130
INT6B/1963R	10-9-V	3198.5	*No comparable data for this lift*				3880	
INT6B/1963R	10-9-V	3194.7	*No comparable data for this lift*				3260	
INT2/1964	5-9-V	3459.6	3300	3250			3390	104
INT2B/1964	5-9-V	3450.1	*No comparable data for this lift*				3450	
INT8/1964	24-10-V	3459.6		3400			3440	101
INT8B/1964	24-10-V	3453.6	*No comparable data for this lift*				4520	
INT8C/1964	24-10-V	3447.9	*No comparable data for this lift*				3290	
EXT3/1964	5-10-V	3459.5	4410	5090			4580	90
EXT3B/1964	5-10-V	3453.7	*No comparable data for this lift*				5730	
EXT3B/1964	5-10-V	3449.7	*No comparable data for this lift*				5750	
EXT5/1964	24-11-V	3459.5	3440	3900			4490	115
EXT5B/1964	24-11-V	3452.5	*No comparable data for this lift*				2450	
Average[2]			4280	5360	6240	7040	6620	
Average (all tests)							4420	
Average[3]			3940	4390			4860[3]	110[3]
Standard deviation (25 years—all tests)							1283	

[1] 10-year cores tested dry (may test about 10-20% higher than saturated test specimens).
[2] Average based on two comparable tests each at 6 mo, 1, 5, 10, and 25 yr.
[3] Average of comparable tests at 25 yr. 25-yr tests as a percent of 1-yr tests only where comparable data exists from the same lift as previous core programs (6 tests). Insufficient comparable data available for 5- and 10-yr tests.

This analysis resulted in a recommendation that the strength properties were not decreasing in the dam, and a comprehensive coring program related to this issue was not necessary. The estimated cost savings for performing a concrete coring and testing investigation at this dam was about $250,000.

Concluding Remarks and Recommendations

The concrete materials properties model developed for mass concrete provides a valuable resource for Reclamation and the Dam Safety Program. Compressive strength, elastic properties, and tensile properties can be identified for three different types of mass concrete; the pre-1920s dams, the post-Hoover dams, and the ASR-affected dams. Verifiable data are needed to document a dam's current condition for dam safety reviews. Knowledge of the expected materials properties for concrete dams is a resource for designers performing initial examinations and comparison to the current condition. Analysis of possible changes in materials properties over time must be noted in structural analysis for long term stability. If the properties are decreasing due to aggressive deterioration, the potential for a dam safety modification exists, and program funding will be needed for design and construction. Verifiable data will be needed to present the case to program managers and the public.

Significant effort has been expended to identify the changes in materials properties due to alkali-silica reaction due in part to current investigations at Seminoe and Parker Dams. Freezing and thawing properties have been entered for mass concrete at Warm Springs and Black Canyon Dams and some structural concretes for aging embankment dam spillways and outlets. This database can be expanded with additional records. These materials properties are important for developing predictive models of concrete deterioration. Freezing and thawing predictive modules under development could be verified from this testing database.

The ACIS concrete materials database is only as good as the data input. Early age concrete properties are difficult to locate and verify due to the lack of a central depository of concrete testing before the creation of the Reclamation Concrete Laboratory in the early 1930s. These early 1900s structures are also the dams most likely to require attention in the next decade. Several early designs require particular attention. The early thin arch dams such as Gerber and Warm Springs Dams are located in aggressive environments and subject to deterioration from freezing and thawing. Early multiple thin arch or slab and buttress dams constructed between 1910 and 1930 may also be in need of investigation. These dams lack the inherent strength and durability to resist the long term effects of aging, and they often have thinner cross sections and thus, less mass to loose before lowering the factors of safety.

Analysis of verifiable materials properties is also necessary for security issues in dam safety. Models developed for nonlinear analysis in seismic or high energy applications normally require input parameters of concrete materials properties. Mass concrete materials properties differ from typical structural concretes due to their varying strengths, elastic properties, materials, and mixture proportions.

Reclamation's database of mass concrete properties is likely the most comprehensive in the world. Aging concrete durability was most recently a featured topic in the 2003 International Committee on Large Dams (ICOLD) Congress in Montreal, Canada. Both the U.S. Society on Dams and ICOLD expressed interest in publishing the results of aging properties and processes of mass concrete dams.

The incorporation of the concrete materials properties with geographical information systems will provide data for decision makers in real time. This trend is necessary as Reclamation becomes more dependent on the Internet for its information. A major need is the transfer of hundreds of documents of materials properties into modern information technology systems. Reclamation's early entry into the development of mass concrete technology is also a handicap as the data becomes unavailable unless transferred from hard copies to modern data storage. Lastly, Reclamation's technical staff itself is aging with the potential for an accompanying loss of institutional knowledge. Documentation of materials properties is necessary to transfer this information to the next generation of dam and dam safety engineers.

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28

Appendix

Data Reports for Mass Concrete Cores—Yellowtail and Parker Dams

Core - Compressive Strength / Elasticity Report

Filter: Feature = YELLOWTAIL

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density

Project: PSMBP-YELLOWTAIL

YELLOWTAIL

DAM

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
DH-63-10-1															
	11/20/1963	10	7+43												
				0	180	0	2	INT-5	1	3450					152
				0	180	4	6	INT-5	1	3780	1	5.49	0.23		151
DH-63-10-2															
	11/21/1963	10	7+45												
				0	180	0	2	INT-6	1	4310	1	5.3	0.23		150
				0	180	2	4	INT-6	1	3890	1	5.05	0.18		154
DH-63-18-1H															
	11/14/1963	18	12+22												
				0	210	0	2	INT-10	1	4740					154
				0	210	4	6	INT-10	1	3570	1	6.04	0.27		153
				0	210	6	8	INT-10	1	4540	1	5.71	0.22		151
DH-63-18-1V															
	8/7/1963	18	12+20												
				0	180	0	2	INT-11	1	4570	1	5.03	0.08		150
				0	180	2	4	INT-11	1	2880	1	5.62	0.14		154
				0	180	4	6	INT-11	1	5430	1	5.54	0.24		154
DH-63-18-2															
	11/15/1963	18	11+80												
				0	225	2	4	INT-9	1	4460	1	5.66	0.26		152
				1	0	0	2	INT-9	1	5120	1	6.1	0.26		151
				1	0	2	4	INT-9	1	4530	1	5.28	0.29		153
				1	0	4	6	INT-9	1	5220	1	5.38	0.22		154
DH-63-18-3															
	11/18/1963	18	12+00												
				0	180	0	2	EXT-1	1	4500	1	5.47	0.26		153
				0	180	2	4	EXT-1	1	5900	1	6.21	0.24		154
DH-63-5-5															
	6/25/1965	5	4+90												

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age		Depth		Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
				Yrs	Days	From	To								
				1	0	0	2	INT-1	1	2970	1	5.01	0.2		153
				1	0	2	4	INT-1	1	3520	1	5.72	0.3		152
DH-64-14-2															
	11/24/1964	14	9+92												
				0	180	0	2	INT-13	1	4070	1	4.74	0.21		148
				0	180	2	4	INT-13	1	3720	1	4.98	0.2		153
DH-64-18-4A															
	5/3/1963	18	12+23												
				1	0	0	2	INT-11	1	5940	1	5.8	0.21		151
DH-64-18-4B															
	6/5/1964	18	12+23												
				1	50	0	2	INT-9	1	6310	1	6.07	0.23		152
DH-64-18-6															
	6/8/1964	18	12+88												
				1	0	0	2	EXT-1	1	4470	1	5.36	0.22		153
				1	0	2	4	EXT-1	1	3810	1	5.38	0.25		150
DH-64-20-2															
	11/17/1964	20	13+20												
				0	180	0	2	INT-12	1	5120	1	5.58	0.27		146
				0	180	2	4	INT-12	1	4430	1	4.96	0.19		151
DH-64-24-1															
	11/9/1964	24	15+52												
				0	180	0	2	EXT-4	1	4390	1	5.75	0.21		153
				0	180	2	4	EXT-4	1	3980	1	5.05	0.19		152
DH-64-24-2															
	11/9/1964	24	15+48												
				0	180	0	2	EXT-5	1	4180	1	5.32	0.2		151
				0	180	2	4	EXT-5	1	2700	1	5.02	0.18		149
DH-64-5-1															
	12/1/1964	5	4+85												
				0	180	0	2	EXT-2	1	3610	1	5.37	0.22		150
				0	180	2	4	EXT-2	1	3610	1	5.53	0.25		152
DH-64-5-2															
	12/1/1964	5	4+80												
				0	180	0	2	EXT 3	1	4120	5	4.95	0.24		152
				0	180	2	4	EXT 3	1	4690	1	5.07	0.21		152
DH-64-5-3															
	11/30/1964	5	4+90												
				0	180	0	2	INT-1	1	4000	1	5.36	0.21		152

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
				0	180	2	4	INT-1	1	2830	1	5.08	0.21		150
DH-64-5-4															
	12/1/1964	5	4+85												
				0	180	0	2	INT-2	1	3640	1	4.89	0.24		146
				0	180	2	4	INT-2	1	2960	1	4.62	0.21		153
DH-64-9-1															
	11/30/1964	9	7+04												
				0	180	0	2	INT-3	1	4570	1	5.74	0.19		154
				0	180	2	4	INT-3	1	3680	1	5.33	0.24		154
DH-64-9-2															
	11/30/1964	9	7+05												
				0	180	0	2	INT-4	1	3250	1	4.42	0.2		
				0	180	2	4	INT-4	1	3300	1	4.72	0.23		150
DH-65-10-3															
	6/4/1964	10	7+45												
				1	0	1	3	INT-6	1	4390	1	4.83	0.24		151
				1	0	3	5	INT-6	1	4410	1	5.82	0.24		156
DH-65-10-4															
	6/5/1964	10	7+44												
				1	0	0	2	INT-5	1	4310	1	5.9	0.22		154
				1	0	2	4	INT-5	1	4150	1	5.3	0.26		150
	6/29/1965	20	13+25												
				1	0	0	2	INT-12	1	5640	1	5.74	0.21		152
				1	0	2	4	INT-12	1	5480	1	6.32	0.25		156
DH-65-14-4															
	6/28/1965	14	9+90												
				1	0	0	2	INT-13	1	3890	1	5.22	0.23		150
				1	0	2	4	INT-13	1	3790	1	4.82	0.2		153
DH-65-18-10															
	6/29/1965	18	12+15												
				1	0	0	2	INT-7	1	5710	1	5.65	0.22		150
				1	0	2	4	INT-7	1	4420	1	5.18	0.21		154
DH-65-18-9															
	9/14/1968	18	12+00												
				5	0	0	2	EXT-1	1	4630	1	6.02	0.19		154
				5	0	2	4	EXT-1	1	5610	1	5.49	0.26		153
DH-65-24-7															
	6/30/1965	24	15+30												
				1	0	0.3	2	INT-8	1	3620	1	5.12	0.23		152

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
				1	0	2	4	INT-8	1	3180	1	5.94	0.3		152
DH-65-24-8															
	7/1/1965	24	15+48												
				1	0	0	2	EXT-4	1	4440	1	5.36	0.22		153
				1	0	2	4	EXT-4	1	4160	5	0.29			154
DH-65-24-9															
	7/1/1965	24	15+48												
				1	0	0.5	2	EXT-5	1	4130	1	4.99	0.21		150
				1	0	2	4	EXT-5	1	3970					149
				1	0	4	5.5	EXT-5	1	3870					151
				1	0	5.5	7	EXT-5	1	3630	1	5.27	0.25	154	
DH-65-5-6															
	6/25/1965	5	4+86												
				1	0	0	2	INT-2	1	3370	1	4.57			147
				1	0	2	4	INT-2	1	3130	1	5.2	0.24		153
DH-65-5-7															
	6/24/1965	5	4+75												
				1	0	0	2	EXT-2	1	4900	1	5.49	0.25		152
				1	0	2	4	EXT-2	1	3910	1	6.04	0.23		154
DH-65-5-8															
	6/24/1965	5	4+80												
				1	0	0	2	EXT 3	1	5130	1	5.04	0.2		151
				1	0	3	4	EXT 3	1	5050	1	5.13	0.2		156
DH-65-9-3															
	6/26/1965	9	7+02												
				1	0	0	2	INT-3	1	4030	1	6.33	0.25		156
				1	0	2	4	INT-3	1	4150	1	5.39	0.25		154
DH-65-9-4															
	6/25/1965	9	7+06												
				1	0	0	2	INT-4	1	2920					146
				1	0	2	4	INT-4	1	3500	1	5.21	0.23		148
				1	0	4	6	INT-4	1	4050	1	5.21	0.23		153
DH-68-10-5															
	10/3/1968	10	7+45												
				5	0	0	2	INT-6	1	5860	1	5.77	0.24		151
				5	0	2	4	INT-6	1	5760	1	6.11	0.21		153
DH-68-10-6															
	10/3/1968	10	7+44												
				5	0	0	2	INT-5	1	4890	1	5.77	0.24		154

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
				5	0	2	4	INT-5	1	4230	1	5.18	0.21		151
DH-68-18-7															
	7/31/1968	18	12+25												
				5	0	2	4	INT-9	1	6350	1				150
				5	0	4	6	INT-9	1	6960	1	5.4	0.2		152
DH-68-18-8															
	8/3/1968	18	12+10												
				5	0	0	2	INT-10	1	5350	1	5.76	0.23		154
				5	0	2	4	INT-10	1	4690	1	5.41	0.21		152
DH-74-10-7															
	3/22/1974	10	7+46												
				10	330	0	2	INT-5	1	4180	1	6.37	0.26		152
				10	330	2	4	INT-5	1	4310	1	6.49	0.29		155
DH-74-10-8															
	3/22/1974	10	7+45												
				10	335	0	2	INT-6	1	6550	1	6.27	0.27		151
DH-74-18-10															
	3/9/1974	18	12+34												
				11	37	4	6	INT-9	1	7520	1	6.67	0.25		157
DH-74-18-11															
	3/16/1974	18	12+04												
				10	350	0	2	EXT-1	1	5120	1	6.31	0.22		
				10	355	2	4	EXT-1	1	5150	1	6.7	0.26		
DH-74-18-12															
	3/19/1974	18	12+04												
				10	350	0	2	EXT-1	1	5280	1	6.05	0.23		154
				10	350	2	4	EXT-1	1	4470	1	6.11	0.24		152
DH-74-18-9															
	3/11/1974	18	12+32												
				11	22	0	2	INT-10	1	5420	1	6.8	0.25		153
				11	22	2	4	INT-10	1	5460	1	6.18	0.28		151
				11	22	4	6	INT-10	1	6210	1	5.85	0.19		152
DH-88-10-9															
	7/7/1988	10	7+45												
				26	0	2	4	INT-6	1	5730					
				26	0	8	10	INT-6	1	3880	1	5.17	0.25		155
				26	0	12	13	INT-6	1	3260	1	6.25	0.26		156
DH-88-18-13															
	6/28/1988	18	12+31												

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
				26	0	4	6	INT-9	1	7510	1	6.04	0.25		154
	12/1/1988	18	12+30												
				26	0	8	10	INT-9B	1	4810	1	6.05	0.25		152
DH-88-24-10															
	7/10/1988	24	15+30												
				25	0	2	4	INT-8	1	3440	1	4.84	0.22		151
				25	0	8	10	INT-8	1	4520	1	6.7	0.29		155
				25	0	14	16	INT-8	1	3290					155
DH-88-24-11															
	7/24/1988	24	15+45												
				25	0	2	4	EXT-5	1	4490	1	5.89			154
				25	0	9	11	EXT-5	1	2450	1	5.71	0.21		154
DH-88-5-10															
	8/3/1988	5	4+80												
				25	0	2	4	EXT 3	1	4580	1	6.89	0.28		152
				25	0	8	10	EXT 3	1	5730	1	5.84	0.26		155
				25	0	12	13	EXT 3	1	5750	1	6.1	0.25		154
DH-88-5-9															
	7/19/1988	5	4+87												
				25	0	2	4	INT-2	1	3390	1	5.5	0.26		152
				25	0	12	13	INT-2	1	3450	1	6.41	0.28		156

Core - Compressive Strength / Elasticity Report

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Project: PARKER-DAVIS

PARKER DAM AND POWERPLANT

DAM

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
DH-1938-2,2A															
	11/1/1938	D	1+95												
				2	68	0	10	M6AZNOV1937	3	6310	1	3.8	0.17		148
				2	208	0	10	M6AZNOV1937	5	4195	3	3.8	0.17		146
DH-1938-6-1															
	11/1/1937	E	2+06												
				2	218	0	10	M6AZNOV1937	5	4075	1	3.7	0.18		
DH-1938-6-3															
	11/1/1938	D	1+80												
				2	209	0	10	M6AZNOV1937	4	4110	3	4	0.13		148
DH-1938-6-4															
	12/1/1938	D	1+56												
				2	193	0	10	M6AZDEC1937	6	3670	4	3.43	0.13		147
DH-1938-6-5															
	11/1/1938	Q	8+41												
				1	63	0	10	M6CANov1937	6	4295	1	3.4	0.14		147
DH-1938-6-6															
	11/27/1938	Q	8+44												
				0	357	0	10	M6CANov1937	3	3040	3	3.17	0.12		149
DH-1938-6-7															
	12/2/1938	R	8+60												
				2	51	0	10	M6CADEC1937	6	3210	3	3.07	0.15		147
DH-1938-8,8A															
	12/1/1938	R	8+69												
				2	47	0	10	M6CADEC1937	8	3230	4	3.13	0.17		146
DH-1940-10-27															
	10/7/1940	E	2+25												
				3	234	0	2	MASS 1.5MSA	1	4080					149
DH-1940-6-11															
	1/1/1940	F	2+60												

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
				3	194	0	1	MASS 1.5MSA	1	3190					150
DH-1940-6-47_50															
	11/7/1940	E	2+25												
				3	315	0	1	M6 JAN38	1	4930	1	2.08	0.15		152
DH-1940-6-51_54															
	11/12/1940	E	2+25												
				3	360	0	1	M6 DEC37	1	4140	1	3.2	0.17		152
DH-1940-6-7															
	9/7/1940	E	2+45												
				2	235	0	1.5	M6 SEPT37	1	3000	1	2.35	0.13		147
DH-1941 ALL6															
	1/1/1941	VARIES													
				4	0	0	10	M6 AVG	3	3830	3	3.4	0.2		
DH-1941-10-64															
	2/20/1941	J	4+91.7												
				3	90	0	1.5	M6 AVG	1	4530	1	3.79	0.22		153
				3	90	2.5	3.5	M6 AVG	1	3300	1	2.48	0.28		
				3	100	6	7	M6 AVG	1	4320	1	2.72	0.19		152
				3	107	8.5	9.5	M6 AVG	1	4950	1	3.83	0.22		152
				3	107	10	11	M6 AVG	1	4500	1	2.76	0.19		
				3	112	17	18	M6 AVG	1	4850	1	3.85	0.19		151
				3	135	23	24	M6 AVG	1	4120	1	3.51	0.22		154
				3	171	31	32	M6 AVG	1	4940	1	3.79	0.15		154
				3	203	38	39	M6 AVG	1	3820	1	2.62	0.14		154
				3	210	48	49	M6 AVG	1	4040	1	3.58	0.25		154
DH-1941-10-86															
	5/23/1941	E	2+40												
				3	100	1	2	MASS 1.5MSA	1	3740	1	1.42	0.17		150
				3	127	4	5	MASS 1.5MSA	1	3710	1	1.86	0.15		151
				3	136	8	9	MASS 1.5MSA	1	4740					154
DH-1945-6-1															
	5/19/1945	J	4+94												
				7	240	0	1	M6 AVG	1	4980	1	2.73	0.14		153
				7	240	1	2	M6 AVG	1	4670	1	2.11	0.12		152
				7	240	2	3	M6 AVG	1	4630	1	3.21	0.16		152
				7	240	3	4	M6 AVG	1	4330	1	2.52	0.16		153
				7	240	5.5	6.5	M6 AVG	1	4200	1	2.02	0.04		154
				7	240	7	8	M6 AVG	1	4920	1	3.25	0.15		152
				7	240	8	9	M6 AVG	1	4600	1	2.41	0.14		152

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
				7	240	10.5	11.5	M6 AVG	1	4340	1	2.8	0.13		151
DH-1945-6-2															
	5/20/1945	E	2+22												
				7	150	0	1	MASS 1.5MSA	1	3030	1	1.79	0.11		151
				7	150	1	2	MASS 1.5MSA	1	2390	1	1.77	0.13		150
				7	150	3.5	4.5	MASS 1.5MSA	1	3020	1	2.09	0.16		152
				7	150	5.5	6.5	MASS 1.5MSA	1	3650	1	2.09	0.13		151
				7	150	7	8	MASS 1.5MSA	1	4180	1	2.82	0.15		151
				7	150	9.5	10.5	MASS 1.5MSA	1	5520	1	2.58	0.15		153
DH-1949-10-1A															
	4/27/1949	E	2+30												
				11	180	0.9	2.3	MASS 1.5MSA	1	2940					153
DH-1949-10-1B															
	4/27/1949	E	2+30												
				11	180	3.2	5	M6 AVG	1	2980	1	3.27	0.28		155
DH-1949-10-2A															
	4/27/1949	Q	8+08												
				11	180	3.8	5.5	MASS 1.5MSA	1	3810	1	2.38	0.2		155
DH-1949-10-2B															
	4/27/1949	Q	8+08												
				11	180	0.2	1.8	M6 AVG	1	3225	1	1.64	0.11		153
DH-1956-10-1A-1															
	8/24/1956	E	2+30												
				18	215	2	3.6	MASS 1.5MSA	1	3020	1	2.01	0.13		151
				18	215	4	5.6	MASS 1.5MSA	1	3620	1	1.48	0.11		151
DH-1956-10-1A-2															
	8/24/1956	E	2+30												
				18	300	2.4	4	M6 AVG	1	4230					153
				18	300	4.9	6.5	M6 AVG	1	3700	1	1.74	0.1		152
				18	300	8	9.6	M6 AVG	1	3990	1	1.51	0.08		153
				18	300	13	14.6	M6 AVG	1	4260	1	3.43	0.17		13
DH-1956-10-2A1															
	8/24/1956	Q	8+08.5												
				18	215	1.4	3	MASS 1.5MSA	1	3200	1	1.25	0.13		153
				18	215	3	4.6	MASS 1.5MSA	1	2890	1	1.45	0.14		151
DH-1956-10-2A-1															
	8/24/1956	Q	8+08												
				18	215	0	1.6	MASS 1.5MSA	1	3450	1	2.49	0.16		151
				18	215	2.2	3.8	MASS 1.5MSA	1	3500	1	1.19	0.11		151

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
DH-1956-10-2A-2															
	8/24/1956	Q	8+08												
				18	300	0	1.5	M6 AVG	1	3590					153
				18	300	1.6	3.1	M6 AVG	1	3480					153
				18	300	4.5	6.1	M6 AVG	1	4290					152
				18	300	7.5	9.1	M6 AVG	1	4310	1	3.68	0.2		153
				18	300	10.4	12	M6 AVG	1	4800	1	1.32	0.08		151
				18	300	13.4	15	M6 AVG	1	5120	1	3.05	0.17		151
				18	300	17.4	19	M6 AVG	1	4270	1	1.78	0.07		153
DH-1956-10-3															
	1/1/1956	K	5+07												
				19	0	0	2	M6 AVG	1	3120	1	0.95	0.18		
				19	0	2	4	M6 AVG	1	2970	1	0.84	0.12		151
				19	0	4	6	M6 AVG	1	2480	1	0.95	0.13		152
				19	0	6	8	M6 AVG	1	3470	1	1.1	0.12		151
				19	0	8	10	M6 AVG	1	3550	1	1.16	0.15		152
				19	0	10	12	M6 AVG	1	2990	1	1.28	0.16		151
				19	0	12	14	M6 AVG	1	3390	1	1.51	0.14		153
				19	0	20	22	M6 AVG	1	2970	1	1.1	0.1		153
				19	0	22	24	M6 AVG	1	2820	1	1.04	0.17		152
DH-1964-6-3															
	1/1/1964	K	5+14												
				27	0	0	1	M6 AVG	1	3510	1	1.57	0.08		152
				27	0	1	2	M6 AVG	1	3400	1	1.57	0.16		151
				27	0	7.7	8.7	M6 AVG	1	2850	1	1.07	0.17		153
				27	0	11.1	12.1	M6 AVG	1	3130	1	1.65	0.06		
				27	0	15.9	16.9	M6 AVG	1	3940	1	1.58	0.12		152
				27	0	16.9	17.9	M6 AVG	1	3130	1	1.55	0.09		152
				27	0	19	20	M6 AVG	1	2830	1	2.13	0.12		152
				27	0	20	21	M6 AVG	1	3410	1	1.44	0.14		155
				27	0	21.8	22.8	M6 AVG	1	3400	1	1.87	0.28		153
				27	0	22.8	23.8	M6 AVG	1	2620	1	1.73	0.18		158
DH-1980-6-1															
	6/1/1980	D	1+80												
				42	0	4.5	5.5	M6 AVG	1	4730	1	1.7	0.1		153
				42	0	11.8	12.8	M6 AVG	1	3610	1	1.07	0.03		155
				42	0	37.3	38.3	M6 AVG	1	5220	1	2.26	0.12		152
				42	0	39.5	40.5	M6 AVG	1	4800	1	2.43	0.15		157
				42	0	71.3	72.3	M6 AVG	1	3940	1	2.37	0.33		154

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
				42	0	75	76	M6 AVG	1	3450	1	1.46	0.13		153
				42	0	92.7	93.7	M6 AVG	1	7360	1	2.75	0.19		155
DH-1980-6-2															
	6/1/1980	Q	8+25												
				42	0	1.1	1.2	M6 AVG	1	3410	1	2.41	0.13		154
				42	0	2.4	3.4	M6 AVG	1	4410	1	1.84	0.15		151
				42	0	12.3	13.3	M6 AVG	1	4330	1	1.84	0.15		155
				42	0	29.5	30.5	M6 AVG	1	5490	1	2.6	0.18		155
				42	0	30.8	31.8	M6 AVG	1	3910	1	2.29	0.14		154
				42	0	51.3	52.3	M6 AVG	1	4480	1	3.58	0.17		156
				42	0	72.8	73.8	M6 AVG	1	4350	1	3.01	0.14		152
				42	0	90.5	91.5	M6 AVG	1	4860	1	2.03	0.1		154
DH-1980-6-3															
	1/1/1980	F	2+80												
				42	0	0.4	1.4	M6 AVG	1	3960	1	2.3	0.14		152
				42	0	2.5	3.5	M6 AVG	1	3210	1	1.53	0.8		151
				42	0	8.2	9.2	M6 AVG	1	4420	1	1.92	0.07		153
				42	0	12.5	13.5	M6 AVG	1	4430	1	2.2	0.11		152
				42	0	34.7	35.7	M6 AVG	1	3990	1	1.71	0.13		153
				42	0	45.7	46.7	M6 AVG	1	3210	1	2.11	0.13		154
				42	0	48.8	49.8	M6 AVG	1	4660	1	2.37	0.11		153
				42	0	72.1	73.1	M6 AVG	1	4510	1	2.84	0.18		153
				42	0	74.7	75.7	M6 AVG	1	5780	1	2.51	0.07		152
				42	0	95.8	96.8	M6 AVG	1	5120	1	2.18	0.1		153
				42	0	100.4	101.4	M6 AVG	1	5080	1	2.85	0.05		153
DH-1980-6-4															
	1/1/1980	K	5+20												
				42	0	4.3	5.3	M6 AVG	1	2490	1	2.18	0.1		153
				42	0	11	12	M6 AVG	1	3670	1	1.72	0.2		154
				42	0	25.7	26.7	M6 AVG	1	3790	1	2.21	0.2		154
				42	0	51.4	52.4	M6 AVG	1	3800	1	1.7	0.04		153
				42	0	73.4	74.4	M6 AVG	1	5230	1	2.09	0.14		154
				42	0	93.6	94.6	M6 AVG	1	8310	1	5.04	0.18		153
				42	0	102.2	103.2	M6 AVG	1	4130	1	1.84	0.05		154
				42	0	126	127	M6 AVG	1	7300	1	5.03	0.23		154
				42	0	148.2	149.2	M6 AVG	1	4250	1	2.94	0.13		152
				42	0	174.6	175.6	M6 AVG	1	7480	1	3.7	0.14		151
				42	0	203.9	204.9	M6 AVG	1	5530	1	2.59	0.1		152
				42	0	224.8	225.8	M6 AVG	1	4485	1	2.58	0.26		152

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age Yrs	Test Age Days	Depth From	Depth To	Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
				42	0	248.5	249.5	M6 AVG	1	5430	1	3.49	0.15		154
DH-2005-6-1-1.5															
	1/12/2005	F	2+52.5												
				67	36	3.3	4.3	MASS 1.5MSA	1	3210		1.29	0.49	2310	148
				67	60	11	12	MASS 1.5MSA	1	3480		1.84	0.28	1360	149
				67	242	22.7	23.7	MASS 1.5MSA	1	4170		2.76	0.21	2000	150
				67	242	27.5	28.5	MASS 1.5MSA	1	3990		2.37	0.28	1720	149
				67	242	31	32	MASS 1.5MSA	1	3930		2.66	0.27	1780	150
DH-2005-6-1-3															
	1/12/2005	F	2+52.5												
				67	80	8.6	9.5	MASS 3MSA							153
				67	90	7.7	8.6	MASS 3MSA	1	4640		2.05	0.18	1670	152
				67	106	20.4	21.4	MASS 3MSA	1	4880		2.79	0.26	1910	154
				67	106	22.8	23.8	MASS 3MSA	1	3930		1.93	0.12	1780	152
				67	110	27.3	28.2	MASS 3MSA	1	4630		2.61	0.2	1640	151
DH-2005-6-1-6															
	1/12/2005	F	2+52.5												
				67	116	1.9	2.8	M6 AVG	1	4350		1.75	0.17	1760	156
				67	123	7.8	8.8	M6 AVG	1	4420		1.81	0.16	2000	152
				67	129	6.9	7.6	M6 AVG							154
DH-2005-6-2-1.															
	1/26/2005	L	5+52.5												
				67	36	11	12	MASS 1.5MSA	1	3790		2.2	0.27	2240	149
				67	60	42.6	43.6	MASS 1.5MSA							151
				67	72	41.4	42.4	MASS 1.5MSA	1	5700		3.03	0.25	2790	151
				67	242	3.5	4.5	MASS 1.5MSA	1	3900		2.83	0.24	1560	150
				67	242	22.8	23.8	MASS 1.5MSA	1	4710	1	2.13	0.17	2540	150
				67	242	27.6	28.6	MASS 1.5MSA	1	4850	1	2.62	0.26	2150	149
				67	242	31.5	32.5	MASS 1.5MSA	1	4050		3.02	0.19	2020	150
DH-2005-6-2-3															
	1/26/2005	L	5+52.5												
				67	80	0.5	1.5	MASS 3MSA							151
				67	80	1.5	2.5	MASS 3MSA	1	4750		3.26	0.16	2040	152
				67	85	4	5	MASS 3MSA	1	4550		1.72	0.29	1920	151
				67	110	12	12.9	MASS 3MSA	1	5710		3.65	0.28	1920	153
				67	119	18	19	MASS 3MSA	1	4350		2.97	0.17	2420	151
DH-2005-6-2-6															
	1/26/2005	I	5+52.5												
				67	126	2.8	3.8	M6 AVG	1	3940	1	1.36	0.31	2670	156

Drillhole Number	Core Date	Dam Block	Drillhole Station	Test Age		Depth		Related Field Mix	No. of Comp. Strength Tests	Average Compressive Strength	No. of Mod. of Elasticity Tests	Average Modulus of Elasticity	Average Poissons Ratio	Average Failure Strain	Average Density
				Yrs	Days	From	To								
				67	130	5.6	6.6	M6 AVG							152
				67	130	6.6	7.5	M6 AVG	1	4060	1	2.09	0.09	1210	155
				67	130	8.9	9.8	M6 AVG	1	3590		2.35	0.5	1700	153
				67	133	11.7	12.6	M6 AVG	1	3930		1.53	0.1	2350	155