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1. PURPOSE

The purpose and objective of this analysis is to recommend and characterize engineering materials that are suitable for emplacement drift ground support for the service life of the repository and that are also acceptable for the postclosure environment. In addition, the analysis estimates the quantities of materials and evaluates material performance and longevity. This task supports a level 3 milestone (RP120M3C) to deliver information to the Performance Assessment (PA) group.

The scope of this analysis is limited to the evaluation of cementitious and steel materials for three permanent ground support systems: precast concrete lining, cast-in-place concrete lining, and steel sets. Related design analyses regarding these ground support systems will be covered in a stability analysis and a constructibility analysis, which are currently being developed by the Repository Design group.

2. QUALITY ASSURANCE

The items which are the subject of this design analysis have not been classified in accordance with QAP-2-3, *Classification of Permanent Items*. However, both "Underground Excavations", of which "Emplacement Panel Openings" is the applicable subsystem, and "Ground Support" are on the Q-List (Ref. 5.1) by DOE direct inclusion. This design analysis activity has been evaluated in accordance with QAP-2-0 and found to be subject to the requirements of the Quality Assurance Requirements and Description (QARD), DOE/RW-0333P. There is no item in this analysis subject to QA controls.

Due to the preliminary nature of this analysis, all input data, excluding codes and standards, shall be treated as unqualified/unconfirmed, and therefore, the outputs are also unqualified. Because of the preliminary nature of the analysis, the formal TBV and TBD tracking system described in NLP-3-15, Rev 3, *To Be Verified (TBV) And To Be Determined (TBD) Monitoring System* is not applicable. The output/conclusions from this analysis cannot be used as input into documents supporting procurement, fabrication, or construction.

3. METHODS

Methods used in this study include literature review, interactive meetings with affected groups (for example, the PA group), consultation with experts, and some arithmetic calculations.

4. DESIGN INPUTS

4.1 DESIGN PARAMETERS

4.1.1 The host rock of the potential repository at Yucca Mountain, Nevada, is a homogeneous, high-silica, rhyolitic tuff with percentages of major constituents in Topopah Spring tuff shown in Table 4.1 (Ref. 5.26).

Table 4.1. Percentages of major constituents in Topopah Spring tuff (core samples 60, 61, and 62 from drill hole USW G-3). (References 5.26)

Constituent	Sample #60	Sample #61	Sample #62	Average	Std dev
SiO ₂	78.4	78.9	78.9	78.73	0.24
Al ₂ O ₃	12.0	12.3	12.2	12.17	0.12
Fe ₂ O ₃	1.016	0.973	1.000	0.996	0.018
CaO	0.492	0.451	0.480	0.474	0.017
MgO	0.1271	0.1281	0.1126	0.123	0.007
TiO ₂	0.1108	0.0927	0.0984	0.101	0.008
Na ₂ O	4.07	3.92	4.25	4.08	0.13
K ₂ O	3.71	3.18	2.94	3.28	0.32
P ₂ O ₅	0.01	0.01	0.03	0.02	0.01
MnO	0.0624	0.0455	0.0488	0.052	0.007

4.2 CRITERIA

The following criteria have been developed to respond to requirements given in the Repository Design Requirements Document (RDRD) (Ref. 5.2) and the Engineered Barrier Design Requirements Document (EBDRD) (Ref. 5.3), which are the reference sources for repository requirements. These criteria will not be fully satisfied by this analysis because of its preliminary nature.

4.2.1 Service Life

The ground support materials for the Repository Segment will be designed for a maintainable preclosure service life of at least 150 years, which is based on EBDRD 3.2.5.4 of Controlled Design Assumptions (CDA) Document. (Ref. 5.4) [EBDRD 3.2.5.4]

4.2.2 Durability

Ground support materials will have acceptable durability for the service life of the repository. [RDRD 3.2.5.2.8A.1, 3.7.5.E.5]

4.2.3 Material Interactions

Ground support materials, prior to their use underground, will first be reviewed for potential effects on engineered barriers, waste isolation, and on site characterization or other testing. [EBDRD 3.2.3.3.A.12]

Ground support materials will consider, to the extent practical, material chemistry and chemical interactions with groundwater that could affect waste package corrosion and radionuclide solubility. [EBDRD 3.2.3.3.A.13]

4.2.4 Creep and Shrinkage

Ground support materials shall be designed for stresses and deformations induced by creep and shrinkage [EBDRD 3.2.6.2.4].

4.3 ASSUMPTIONS

The following assumptions were made to perform the analysis and are listed along with a basis for the assumption and reference to the CDA document (Ref. 5.4). Assumptions are also referenced in the text of the analysis.

- 4.3.1** The repository host horizon will be located mainly in the TSw2 geologic unit within the primary area, for the reference mass loading of 80 - 100 metric tons of uranium (MTU) per acre. [CDA: Key 022] (Secs. 7.3.4.3, 7.4.2, 7.5.2.1)
- 4.3.2** Lining and grouting materials may be used in repository construction if it has been shown that their use does not adversely impact waste isolation nor interface with required testing activities. They will be evaluated for chemical reactions that may adversely impact waste isolation. [CDA: EBDRD 3.2.3.3.A.13] (Secs. 7.2.1, 7.2.2)
- 4.3.3** Maximum allowable air temperature in emplacement drifts during emplacement is 50 °C. [CDA: DCSS 019] (Secs. 7.3.1.1, 7.6.2.2, 8.4.2)
- 4.3.4** Maximum allowable preclosure rock surface temperature in emplacement drifts is 200 °C. [CDA: DCSS 023] (throughout Secs. 7 and 8)
- 4.3.5** Organic materials are restricted for use as rock support and other postclosure permanent material in all openings. Concrete (subject to restrictions on chemical

composition of cementitious materials) and steel are allowable preclosure construction materials in all openings. [CDA: DCSS 027] (Secs. 7.3.4.5, 7.5.2.1, 7.6)

4.3.6 A single ground support type will be used in emplacement drifts. Candidate ground support types under consideration:

- Precast concrete
- Cast-in-place concrete
- Steel sets [CDA: DCSS 034] (Secs. 7.2.1, 7.3)

4.3.7 Thermal expansion coefficients of in situ rock during heating - TSw2:

$5.07 \times 10^{-6}/^{\circ}\text{C}$ @ 25 - 50 °C
 $7.03 \times 10^{-6}/^{\circ}\text{C}$ @ 50 - 100 °C
 $8.19 \times 10^{-6}/^{\circ}\text{C}$ @ 100 - 150 °C
 $8.97 \times 10^{-6}/^{\circ}\text{C}$ @ 150 - 200 °C [CDA: TDSS 007] (Sec. 7.6.2.1)

4.3.8 Ground water chemical characteristics at the proposed repository horizon for typical conditions are represented by "(well) J-13", and for extreme conditions by "concentrated (well) J-13 acidic or alkaline". [CDA: TDSS 025] (Secs. 7.3.4.3, 7.4.1, 7.5.2.1)

4.3.9 The pore solution of the cement paste will have target pH values lower than 10 to minimize the potential geochemical impacts to waste isolation during the postclosure period in the repository environment (Ref. 5.5). (Secs. 7.4.3, 7.5.2.2)

4.3.10 Target water-cement ratio for the concrete mix will be assumed to be 0.4 to ensure adequate strength and long-term durability (Ref. 5.7; ACI 211.1, Table 6.3.4a). (Sec. 7.5.2.2)

4.3.11 The absolute volume method for proportioning concrete mixtures based on ACI 211.1 will be used. This method will provide more accurate results. (Sec. 7.5.2.2)

4.3.12 No air-entrainment admixture will be used for repository environment because there is no freezing-thawing effect. (Secs. 7.3.4.7, 7.5.2.1, 7.5.2.2)

4.3.13 Emplacement drift diameter of 5.5 m and lining thickness of 200 mm will be assumed in this analysis. These values are used in the current repository design, which allows waste packages to be placed center in-drift, on pedestals, using gantry emplacement. (Sec. 7.5.2.3)

4.3.14 Materials for emplacement drift ground support lining systems will be steel-fiber reinforced concrete or structural steel and shall have the following strength values:

- Concrete: minimum compressive strength at 28 days (f'_c) 34.5 MPa (5000 psi) (based on design needs)
- Structural steel: minimum yield strength 248 MPa (36000 psi) (ASTM A36) (common design practice) (Sec. 7.5.2.2)

4.3.15 The specific gravity values of portland cement, silica fume, and steel fiber are assumed to be 3.15 (ACI 211.1, Ref. 5.6), 2.2 (Ref. 5.6), and 7.850 (ASTM A820), respectively. (Sec. 7.5.2.2)

4.3.16 Coarse aggregate has a moisture content of 1%, an oven-dry specific gravity of 2.68, absorption of 0.5%, and an oven-dry rodded unit weight of 100 lb per cu ft (1.60 g/cm³). Fine aggregate has a moisture content of 3%, an oven-dry specific gravity of 2.64, absorption of 0.7%, and a fineness modulus of 2.8. (all these values are within the typical range for normal-weight concrete, see Ref. 5.6) (Sec. 7.5.2.2)

4.4 CODES AND STANDARDS

4.4.1 American Association of State Highway and Transportation Officials (AASHTO)

AASHTO T277-93 Standard Method of Test for Rapid Determination of Chloride Permeability of Concrete

4.4.2 American Society for Testing and Materials (ASTM)

ASTM C33-93 Standard Specification for Concrete Aggregates

ASTM C143-90a Test Method for Slump of Portland Cement Concrete

ASTM C150-95a Standard Specification for Portland Cement

ASTM C227-90 Test Method for Potential Alkali Reactivity of Cement-Aggregate Combinations (Mortar-Bar Method)

ASTM C418-90 Test Method for Abrasion Resistance of Concrete by Sandblasting

ASTM C441-89 Test Method for Effectiveness of Mineral Admixtures in Preventing Excessive Expansion of Concrete Due to Alkali-

Aggregate Reaction

ASTM C494-92	Standard Specification for Chemical Admixtures for Concrete
ASTM C779-95	Test Method for Abrasion Resistance of Horizontal Concrete Surfaces
ASTM C1202-94	Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration
ASTM C1240-95a	Standard Specification for Silica Fume for Use in Hydraulic-Cement Concrete and Mortar
ASTM C1293-95	Standard Test Method for Concrete Aggregates by Determinations of Length Change of Concrete Due to Alkali-Silica Reaction
ASTM A36/ A36M-94	Standard Specification for Carbon Structural Steel
ASTM A820-90	Standard Specification for Steel Fibers for Fiber Reinforced Concrete

4.4.3 American Concrete Institute (ACI)

ACI 201.2R-92	Guide to Durable Concrete, Part 1
ACI 209R-92	Prediction of Creep, Shrinkage, and Temperature Effects in Concrete Structures, Part 1
ACI 211.1-91	Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete
ACI 318-95	Building Requirements for Structural Concrete (ACI 318-95) and Commentary (ACI 318R-95)
ACI 349-90	Code Requirements for Nuclear Safety Related Concrete Structures (ACI 349-90)
ACI 544.1R-86	State-of-the-Art Report on Fiber Reinforced Concrete
ACI 544.3R-88	Guide for Specifying, Mixing, Placing, and Finishing Steel Fiber

Reinforced Concrete

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6. USE OF COMPUTER SOFTWARE

Not used.

7. DESIGN ANALYSIS

7.1 INTRODUCTION

The purpose of this analysis is to recommend and characterize engineering materials for emplacement drift ground support. These materials must have sufficient capacity to withstand in situ stresses, thermally-induced stresses, and seismic loads for the 150-year service life of the repository. In addition, the materials used for ground support must be acceptable for the postclosure period, where the principal concern is waste isolation and geochemical behavior of materials rather than a functional ground support system.

7.2 POSTCLOSURE EFFECTS OF MATERIALS

An objective of this analysis is to recommend the materials, especially the composition of concrete and steel, that will provide the least impact to the postclosure environment in regard to waste isolation (Sec. 4.3.2). The first step in doing this is to describe the potential long-term effects of concrete and steel based, in part, on requirements to be developed in conjunction with the M&O Performance Assessment (PA) group.

7.2.1 Concrete

The postclosure evolution (i.e., change of properties with time) of cementitious materials in the repository emplacement drifts has potential detrimental effects on waste isolation due to the alkalinity of the concrete. The pore solution of ordinary portland cement-based concrete is dominated initially by alkalis (Na_2O and K_2O) and has a pH of about 13 to 13.5. Due to subsequent equilibrium reactions with the portlandite ($\text{Ca}(\text{OH})_2$), the pH of the pore solution reduces to 12 or 13 (Ref. 5.5). A beneficial effect of this level of alkalinity is the possible reduction in waste package corrosion by means of coatings (passivation films) of iron-hydroxide produced in the alkaline solution that may form on the steel waste package. To be effective such films need to be impermeable and relatively continuous across the waste package surface. Potentially harmful effects, which are the concerns here, are:

(1) Radionuclide transport

Concrete pore solutions with high pH have the potential to increase radionuclide solubility

limits via CO_3^{2-} and OH^- complexing, of which the major concern is the potential effects to dissolved actinide concentrations (Np, Pu, Am). Colloids generated from cementitious materials may also sorb radionuclides and could enhance transport if they migrate in groundwater. Alternatively, certain cement solids (i.e., CSH phases) and possible alteration minerals of cement may provide both sorption and cation exchange sites for radionuclides, thus providing some level of retardation. However, the potential radionuclide transport mechanism is considered the dominant mechanism and is of primary concern (Ref. 5.5).

(2) Natural barrier sorption

Dissolved constituents of concrete, such as hydroxide and carbonate, form at high pH. Such constituents have low or non-existent sorption capacity in contact with the negatively charged rock surface and thus could reduce or eliminate sorption of radionuclides by natural barrier components such as zeolites.

(3) Production of water

Water from dehydration reactions within the cement could be produced in the near-field environment and possibly increase the relative humidity. If this water content is comparable to the natural water content of the excavated rock volume, this would not be a major effect. The amount of water added to the near-field environment may be minimized by using precast concrete, which would reduce free water by the curing/hydration of cement paste and encourage water migration out of the cement before placement of the lining.

(4) Changes in rock mass Porosity and permeability

The porosity and permeability of the host rock could be changed by increases in silica solubility. Silica (for example, quartz and amorphous silica) solubility increases above a pH value of 10 (Ref. 5.5). Silica minerals are about 79% of the repository host horizon (Sec. 4.1.1) and formation of alteration phases could change porosity and permeability. The extent to which this would occur and the potential performance consequences are uncertain.

Preliminary recommendations from PA, based on consultation with MGDS Development and cement consultants, do not preclude the use of concrete but emphasize the need to minimize potential impacts on the repository from the use of concrete, as follows (Ref. 5.5):

If concrete is desirable to use as lining for ground support (Sec. 4.3.6), then minimizing the potential postclosure impacts on the waste isolation capabilities of the site (Sec. 4.3.2) will most likely be achieved by (a) using precast concrete, (b) designing a mix with lower Ca/Si ratio, (c) using techniques such as particle size engineering, steam-curing, or pressure-curing to reduce the

concrete permeability and water content needed for higher silica cements, (d) using tuff aggregate, and (e) investigating alternative cements, such as C_2S cement, that may have a lower pH than standard portland cements.

These recommendations and other aspects of long-term material performance are considered in this design analysis.

7.2.2 Organic Materials

Organic materials may be present in concrete in the form of admixtures, such as superplasticizers. The introduction of organic materials into the emplacement drift area may have an impact on performance (Sec. 4.3.2) through changes in the concentrations of organic acids and organic colloids which can increase waste package corrosion, radionuclide solubility and transport, and silicate mineral dissolution. In addition, organics may promote microbial activity which is another factor leading to waste package corrosion. Furthermore, organic substances can take part in oxidation/reduction reactions and, therefore, may contribute to the generation of locally reducing conditions (Ref. 5.8).

7.2.3 Steel

From the viewpoint of PA, there is little concern about the introduction of steel sets in the repository because a large number of waste packages (more than 10,000, see Ref. 5.4, Key assumption 003) will be emplaced in the emplacement drifts. The outer barrier material of the waste packages is carbon steel, which is very similar to the material for the steel sets. During the preclosure period, the potential detrimental impact caused by steel corrosion on the structural stability of steel sets in the very dry and hot environment at the emplacement areas should be very minimal. In the postclosure repository period, however, especially after the temperature cools off and relative humidity is increased, the potential for steel corrosion would probably be higher. Since there are many uncertainties and variables involved in the mechanism and prediction of corrosion during the postclosure period, it is beyond the scope of this study to investigate this issue.

There are some potential near-field geochemical impacts in the postclosure period due to the use of steel sets. Steel, which contains abundant Fe, may stabilize Fe-oxide or Fe-hydroxide solids in the near-field, which could provide effective sorption sites for many dissolved species, and may be particularly effective in retarding generally non-sorbing elements such as Tc. In addition, as the Fe in steel will oxidize, it will provide a sink for oxygen in the system and may generate locally reducing conditions, depending upon the rate of oxygen supply from the gas phase. This could lower solubility-limits on dissolved concentrations of many metal ions. Finally, colloid generation from steel may sorb radionuclides and provide additional transport capabilities (Ref. 5.8).

7.3 CHARACTERISTICS OF CONCRETE

An option for repository permanent ground support is a concrete lining, either precast or cast-in-place, in the emplacement drifts (Sec. 4.3.6). Because of the ease of construction and increased material quality control in determining long-term effects of concrete, precast concrete is of particular interest. This section considers the characteristics of concrete needed for the anticipated repository environment and service life.

7.3.1 Mechanical Properties

7.3.1.1 Compressive Strength

The uniaxial compressive strength of concrete is an important property for engineering design, mainly because compressive strength is used to evaluate the capacity of the concrete lining to withstand the imposed in situ, thermal, and seismic loads. In addition, changes in the measured concrete strength are used to determine the effects of temperature, radiation, and geochemistry.

It is well known that elevated temperature has a significant effect on concrete compressive strength, for example, various authors (Refs. 5.9, 5.10, and 5.11) show that concrete strength reduction due to heating is significant above 93 °C (200 °F). The effect of the heating is also influenced by heating rates, cycles of heating and cooling, the presence of applied stress, the type of aggregate, and the degree to which the concrete is sealed (i.e., moisture within concrete was not free to evaporate to the surroundings during testing). Evaluation of concrete performance at elevated temperature has led to limitations on the applicability of typical ambient test values. For example, the temperatures at which it becomes necessary to evaluate thermal effects on concrete strength are embodied in the ACI Manual of Concrete Practice (ACI 349), as follows:

A.4 Concrete temperatures

- A.4.1** The following temperature limitations are for normal operation or any other long term period. The temperatures shall not exceed 150 °F (66 °C) except for local areas, such as around penetrations, which are allowed to have increased temperatures not to exceed 200 °F (93 °C).
- A.4.2** The following temperature limitations are for accident or any other short term period. The temperatures shall not exceed 350 °F (177 °C) for the surface. However, local areas are allowed to reach 650 °F (343 °C) from steam or water jets in the event of a pipe failure.

- A.4.3 Higher temperatures than those given in Sections A.4.1 and A.4.2 above may be allowed for concrete if tests are provided to evaluate the reduction in strength and this reduction is applied to design allowables. Also, evidence shall be provided which verifies that the increased temperatures do not cause deterioration of the concrete either with or without load.

Comparing the above temperature limitations on concrete with calculated repository temperatures, which approach the temperature limit of 200 °C (Assumption 4.3.4) for the emplacement drift wall rock, indicates that repository temperatures will significantly exceed normal temperature levels for concrete design. On this basis, it is necessary to provide data to substantiate the concrete design analyses. An objective of this analysis is to develop a concrete formulation that will be suitable for repository design and that can be used for the testing of strength and other engineering parameters. However, until such testing is done, design parameters such as compressive strength must be based on existing test results.

Based on an extensive literature survey of the effect of elevated temperatures (Ref. 5.12, Appendix A) on compressive strength, it was found that most strength loss is experienced within the first few months of exposure, and that additional strength loss upon continued exposure is small. This finding suggests that most strength losses of concrete occur within the first few years following waste emplacement and any further strength loss will not be significant. It also suggests that a relatively short-term test program may provide sufficient basis to determine the behavior of concrete for the duration of the repository preclosure period.

Tests on Hanford Concrete

Concrete testing and evaluation for a nuclear waste storage vault at Hanford, Washington (Refs. 5.13 and 5.14) provides data on mechanical properties, including strength, that are considered representative of concrete behavior typical of potential repository materials. These data were collected by Portland Cement Association's (PCA) Construction Technology Laboratory on concrete test cylinders for a temperature range of 121 °C (250 °F) to 232 °C (450 °F).

For the PCA tests, concrete specimens were prepared using a basalt aggregate and ASTM C150 type II (low alkali) portland cement. Two concrete mix designs were tested: one for a minimum strength of 21 MPa (3000 psi) and one for 31 MPa (4500 psi) at 28 days. Specimens were continuously moist-cured at 70 °F in a fog room for 28 days minimum until placed in the oven. They were heated to the test temperature at a rate of 70 to 75 °F (21 to 24 °C) increase per day, maintained at 250, 350, and 450 °F (121, 177, and 232 °C) for up to 920 days, then removed from the oven and tested while hot. One set of specimens was continuously exposed to 350 °F (177 °C) for 1300 days. Some specimens were subjected to 14- and 28-day temperature cycles between 70 °F and 350 °F (21 and 177 °C), for up to 18 cycles (Ref. 5.13).

In general, results for the PCA tests showed that strength decreased with increasing length of exposure to elevated temperatures, and also decreased with increasing temperature. After approximately 900 days of heating, average compressive strength of cylinders heated at 250, 350, and 450 °F (121, 177, and 232 °C) were reduced 12, 23, and 34%, respectively. A comparison of PCA's test data to those reported in Lankard's survey (Ref. 5.9) for unsealed concrete is shown in Figure 7.1 (Ref. 5.14). It can be seen that the PCA data fall within the Lankard survey range. For a temperature of 200 °C (392 °F), which is the temperature limit in the repository emplacement drift (Sec. 4.3.4), it is assumed that the strength of concrete may be reduced about 30 percent over a 150-year service life.

By using the PCA data, an equation for predicting time-dependent and temperature related compressive strengths was derived (see equation 5 of Ref. 5.13). It needs to be noted that equation 5 of Ref. 5.13 is normally valid for constant temperature histories, and for temperature, strength, and time within the range of the PCA data. It should be also pointed out that this expression is used to extrapolate concrete strength out to 50 years for strength measured after about 4 years exposure to elevated temperature. Due to the uncertainty of this extrapolation, precaution should be used in applying this equation. Another caution is that the strength used is the initial specified strength, but the initial actual strengths were much higher. So some adjustment must be made to account for this.

Compared with compressive strength, the influence of elevated temperature on concrete tensile and flexural strength has received relatively little attention. For ambient conditions, the flexural strength is often approximated as 7.5 to 10 times the square root of the compressive strength and the tensile strength is about 8 to 12 percent of the compressive strength and is often estimated as 5 to 7.5 times the square root of the compressive strength (Ref. 5.6). It is uncertain whether this relationship will be valid for elevated temperature condition.

The results of splitting tensile strength data from Hanford concrete indicated that it decreased with length and degree of exposure to elevated temperature in a manner similar to that observed in compressive strength data (Ref. 5.14). The results also indicated that concrete exposed to extended periods of heating had somewhat greater strength loss.

Effect of Thermal Cycles

Davis (Ref. 5.10) noted that the loss in strength for concrete specimens subjected to wide fluctuations in temperature has been observed to be two or three times as great as for constant exposure to high temperature. However, the PCA tests showed that cyclically varying temperatures produced smaller changes in concrete mechanical properties than equivalent exposure to fixed maximum temperature (Ref. 5.14). With respect to thermal expansion, Davis (Ref. 5.10) found that changes in temperature were destructive to the concrete with sudden changes being much more severe than slower ones and that concretes having higher coefficients

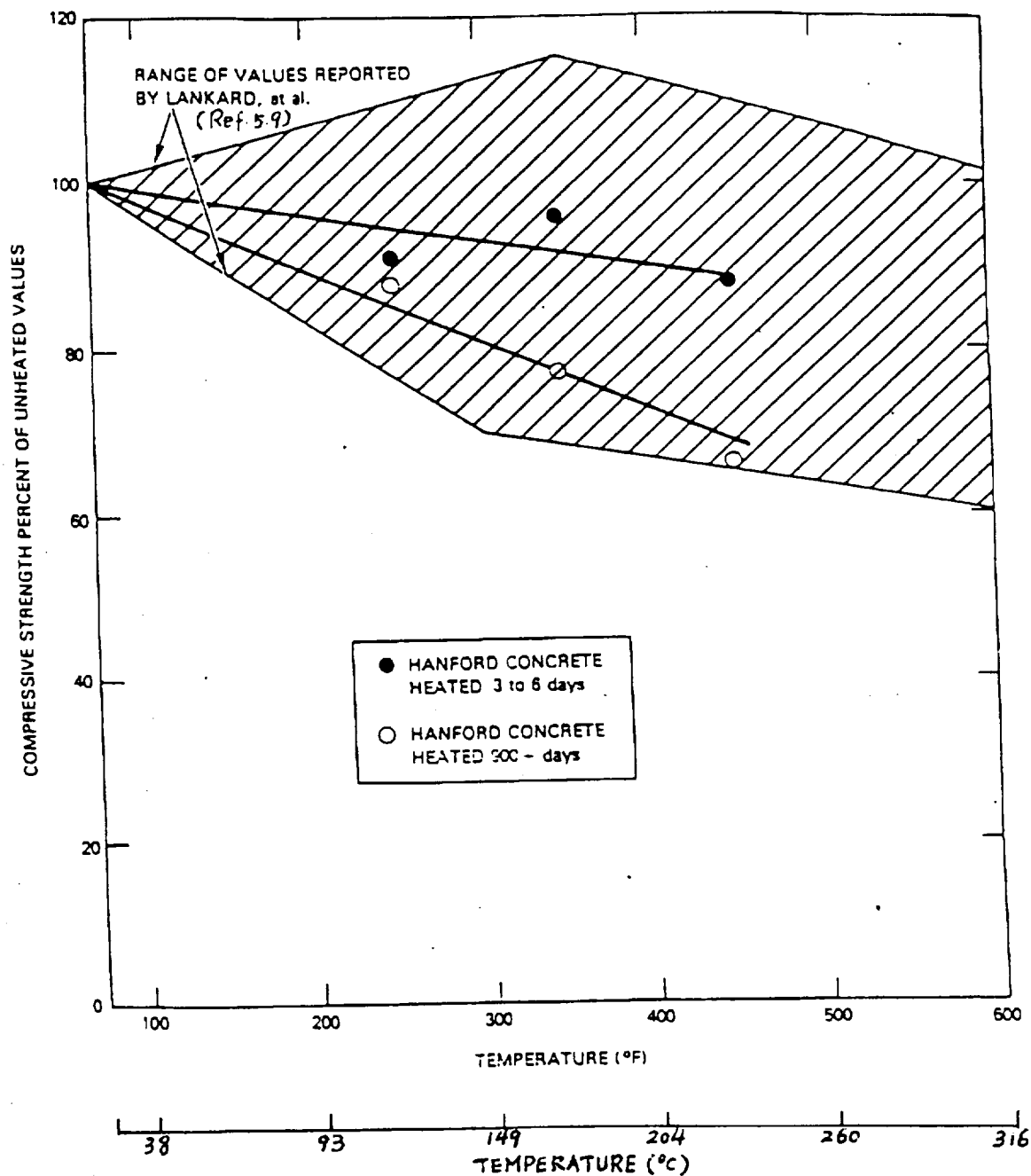


FIGURE 7.1 . Compressive Strength of Hanford Concrete Compared to Other Elevated Temperature Test Data. (Modified from Ref. 5.14)

of expansion were less resistant to temperature changes than concrete with lower coefficients. PCA's results on this aspect were mixed. Application of one thermal cycle produced a permanent increase in concrete coefficient of thermal expansion. Further cycling, however, produced no additional changes in concrete strain behavior (Ref. 5.14).

In regard to blast cooling of emplacement drifts, based on research to date, a cycle of heating and cooling on concrete samples (from about 200 °C (Sec. 4.3.4) to 50 °C (Sec. 4.3.3) in a matter of hours) is expected to result in a maximum strength loss of about 25% (Ref. 5.12, p. 61). Since the temperature increase is expected to be slow and uniform in the repository for an unventilated emplacement drift with the exception of a potential blast cooling event occurring when emplacement drift temperature is decreased to 50 °C over a period of a few to several weeks, the impact of cyclic varying temperature should be insignificant. Notice that 200 °C is the maximum allowable emplacement drift wall rock temperature (Section 4.3.4) and that 50 °C is the maximum allowable temperature for the use of equipment and instruments (Section 4.3.3). Even though this result is conservative for repository conditions, i.e., the strength loss could be less due to longer temperature cycling time, repeated cycles of cooling and heating should be avoided.

Effect of Radiation

Although the exposure of concrete to elevated gamma and neutron fluxes can lead to measurable deterioration, numerous studies have defined radiation exposure limits, below which there is a negligible loss of compressive strength. These limits are much larger than reasonably predicted radiation exposures for concrete within the waste emplacement drifts.

Nuclear radiation can result in lattice defects within crystalline material, causing an increase in brittleness. Ionized radiation may cause the loss of free or bonded water, decreasing the hydraulic bonding strength. Finally, radiation may lead to the breakdown of atomic bonds. Attenuation of the radiation by the material often causes its internal temperature to increase. Elevated temperature caused by radiative heating may be an important mechanism causing a loss in concrete strength, perhaps the predominant strength-loss mechanism (Ref. 5.15, p. C-14).

It was indicated in the M&O Advanced Conceptual Design (ACD) Report that portland cement-based concrete with a limestone aggregate was resistant to integrated neutron fluxes of the order of $10^{19} \text{ n} \cdot \text{cm}^{-2}$ (Ref. 5.15, p. C-14). These specimens displayed limited loss in compressive strength after exposure and concomitant heating to 125°C. Samples that were exposed to an integrated neutron flux of $10^{20} \text{ n} \cdot \text{cm}^{-2}$ were essentially destroyed. It also showed that compressive strengths of serpentine aggregate-bearing concretes subjected to integrated irradiation neutron fluxes (or fluence) of 2×10^{19} to $10^{20} \text{ n} \cdot \text{cm}^{-2}$ at energies above 1 Mev and at temperatures on the order of 200°C (Sec. 4.3.4) were shown to be the same or only slightly diminished relative to those that experienced only thermal cycling in the absence of irradiation. It was also indicated that concrete specimens exposed to a gamma flux exceeding 1×10^{11} rads,

resulted in no measured deterioration of compressive strengths. Based on the results of several gamma and neutron irradiation concrete exposure studies, it was concluded that most concretes are resistant to deterioration by neutron fluxes of less than $10^{19} \text{ n} \cdot \text{cm}^{-2}$ (Ref. 5.15, p. C-15).

Gamma radiation field strength at concrete surfaces within the waste emplacement drifts is conservatively estimated at 10 (R/hr) (rems per hour) from a typical waste package. Neutron radiation field strength at the concrete surface is much less than that from gamma radiation. Based on an integrated gamma radiation exposure over 150 years (Sec. 4.2.1) and assuming no decay, this results in an integrated exposure of $1.314 \times 10^7 \text{ R}$ at the concrete surface. This exposure is about four orders of magnitude below an approximate threshold of $1 \times 10^{11} \text{ R}$ (Ref. 5.15, p. C-15), above which the measurable degradation of concrete is predicted. Radiolysis can occur upon elevated exposure of water to gamma radiation. It was reported that the gas evolved from irradiated concrete consists of hydrogen, oxygen, nitrogen, carbon monoxide, and carbon dioxide and that this gas development has minimal effect on concrete properties (Ref. 5.15, p. C-16).

7.3.1.2 Deformability

Concrete deformability is defined by the modulus of elasticity (also called elastic modulus) and Poisson's ratio. In general, the modulus of elasticity is related to compressive strength and density, and factors that affect strength should similarly influence the elastic modulus. For normal-weight concrete and for temperatures near ambient, the modulus of elasticity can be approximated as 57,000 times the square root of the compressive strength (ACI 318, Sec. 8.5).

Heating, however, has a pronounced effect upon the modulus of elasticity of concrete: for example, modulus values of 67 and 33 percent of room temperature values were reported in one series of tests on concrete samples tested at 200 °C and 350 °C, respectively (Ref. 5.10). Lankard, et al., (Ref. 5.9) compared test results from a number of investigators studying the effects of elevated temperature on the modulus of elasticity of various concretes. Figure 7.2 (Ref. 5.14) shows the range of values reported by Lankard. Also shown on Figure 7.2 are the PCA test results for short- and long-term exposure to elevated temperatures. The elastic moduli of Hanford concrete subjected to less than 7 days heating are near the lower bound of the range reported by Lankard. However, values for samples heated at 121, 177, and 232 °C (250, 350, and 450 °F) for more than 900 days fall outside this range. This result indicates that long-term exposure to elevated temperatures produces losses in concrete modulus larger than those determined from the relatively short-term tests. This difference is more noticeable at 232 °C (450 °F), where short-term test results indicated moduli losses on the order of 35 percent, whereas long-term test results show moduli losses about twice as large. For a temperature of 200 °C (or 392 °F) (Sec. 4.3.4), which is the temperature limit in the repository emplacement drift, and for prolonged periods of heating above 100 °C, it is reasonably conservative to assume that the modulus of elasticity of concrete may be reduced about 50 percent compared to that of unheated

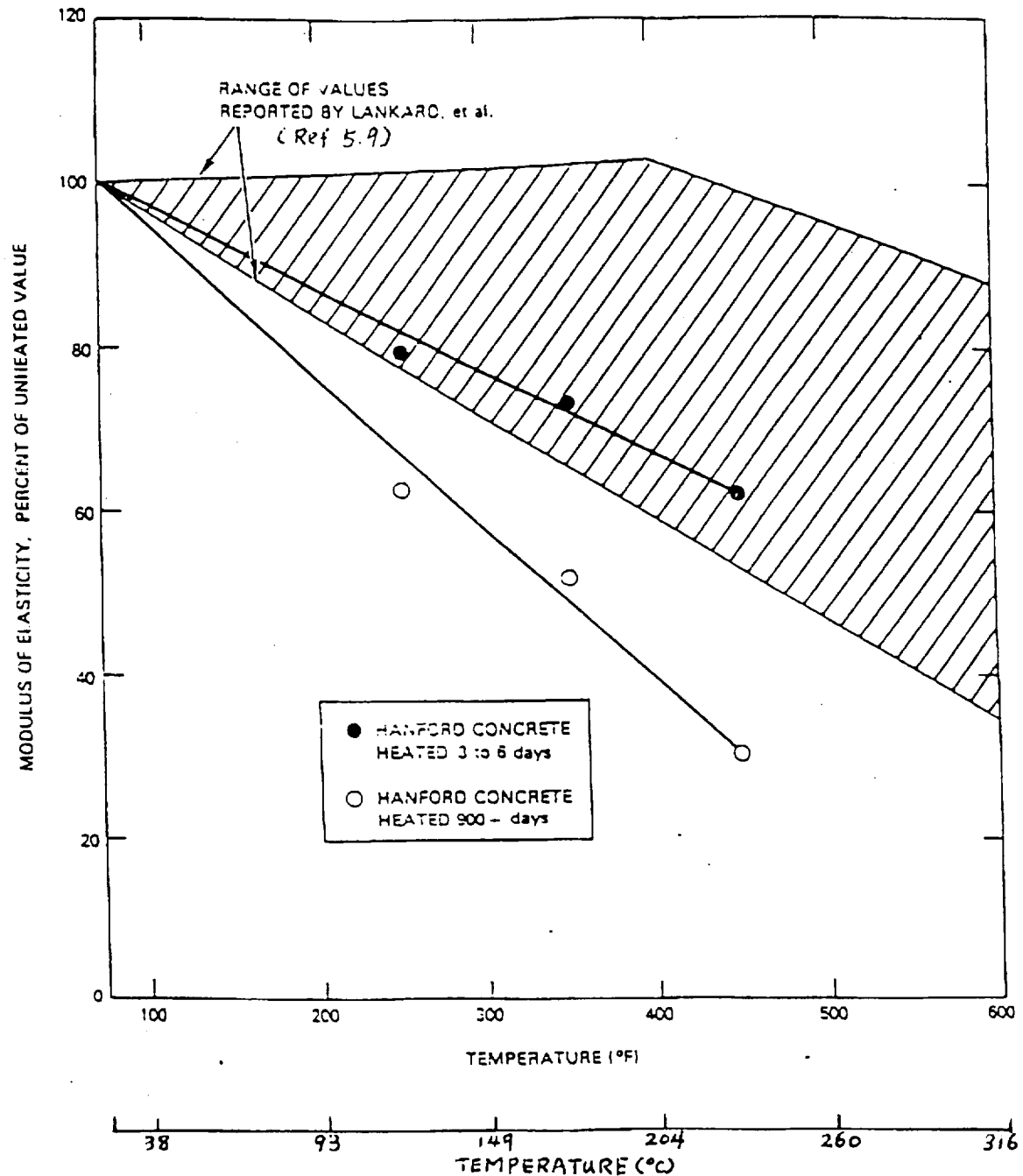


FIGURE 7.2 . Modulus of Elasticity of Hanford Concrete Compared to Other Elevated Temperature Test Data. (Modified from Ref. 5.14)

concrete.

By using the PCA data, an equation for predicting time-dependent and temperature related modulus of elasticity was derived (see equation 4 of Ref. 5.13). It should be noted that equation 4 of Ref. 5.13 is normally valid for constant temperature histories, and for those conditions within the range of the PCA data. Since the general trend of temperature history for the rock temperature at an emplacement drift wall is variable with rapid increase in the first few years and gradual slowdown after about 10 years, and levelling off about 50 years after waste emplacement (see Figure B-6 in Appendix B of Ref. 5.15), equation 4 of Ref. 5.13 based on constant temperature may not be applicable.

In PCA's study, a mathematical expression for predicting the modulus of elasticity in incremental form for non-constant temperature histories is given as follows (Ref. 5.13):

$$\Delta E_n = -b\Delta T_n - c \Delta t_n \exp [(E_n - a + bT_n)/c] \quad (7-1)$$

where:

ΔE_n = the change in modulus due to change in temperature and time during the n^{th} time step (psi)

ΔT_n = the change in temperature during the n^{th} time step ($^{\circ}\text{F}$)

Δt_n = the incremental time during the n^{th} time step (days)

E_n = the modulus of elasticity prior to the n^{th} time step (psi)

T_n = the temperature prior to the n^{th} time step ($^{\circ}\text{F}$)

Values for a, b, and c are as follows:

$$a = 6.0 \times 10^6$$

$$b = 7.0 \times 10^3 \text{ for positive } \Delta T \text{ and } 3.5 \times 10^3 \text{ for negative } \Delta T$$

$$c = 1.95 \times 10^5$$

The reduced value of b for negative ΔT is for partial recovery of the modulus with a temperature decrease. However, chemical and physical changes in the concrete do not support a phenomenon of partial recovery of properties on cooling. Loss of water is the major factor in the degradation of concrete properties at elevated temperatures. Because there is no recovery of water upon cooling (unless the test specimen is subjected to soaking), there is no basis for assuming recovery of properties (Ref. 5.13). Marechal (Ref. 5.16) also found that losses in moduli of elasticity due to drying or heating are irrecoverable. Based on this information, it is considered appropriate to only use equation (7-1) with the b-value for positive ΔT .

Several investigators have examined the influence of elevated temperatures on the Poisson's ratio of concrete (Refs. 5.17 and 5.18). In one study, there were indications that the magnitude of Poisson's ratio decreases with increasing temperature, however, results from published test

results indicate large scatter. Overall, no general trend of the effect of temperature on Poisson's ratio has been established.

PCA's test results show that Poisson's ratio for heated concrete exhibits no well defined relationship with respect to time or temperature. The Poisson's ratio data showed large scatter compared to values for mature, unheated concrete, but, in general, values varied from 0.10 to 0.14. Much of this variation occurred during initial heating, and no systematic changes were observed in the data with additional exposure to elevated temperatures.

7.3.1.3 Creep/Stress Relaxation

The time-dependent increase of strain in hardened concrete subjected to sustained stress is defined as creep (ACI 209R). Because this increase can be several times as large as the strain from loading, creep is of considerable importance in structures. Creep is also important in another, beneficial sense: if a concrete structure is deformed by external movement, stress increases in the concrete due to the imposed deformation can subsequently be relaxed due to creep of the concrete, thus reducing the need for a compensating increase in strength. This is basically the situation of a concrete lining in a circular emplacement drift, with the external movement caused by thermal heating and expansion of the rock mass, which in turn deforms the lining. Further discussion of the beneficial, stress relieving aspects of creep is provided by Ross (Ref. 5.19) and Neville (Ref. 5.21). The ACI code also allows creep to be used as a benefit in design, for example, quoting in part, Section A.3.4 (ACI 349):

When thermal stress is combined with the stress due to other loads to determine a design stress, the magnitude of the design stress must not be less than the magnitude of the stress due to other loadings alone unless...among other factors...the reduction of long term stresses due to creep...are considered.

When concrete is loaded, the deformation caused by the load can be divided into two parts: a deformation that occurs immediately (elastic strain) and a time-dependent deformation (i.e., creep) that begins immediately but continues at a decreasing rate for as long as the concrete is loaded. It is difficult to differentiate accurately between the immediate elastic strain and the onset of creep, but it is not of practical importance as it is the total strain induced by the applied loading that matters. Also, the creep definition lumps together the basic creep (under which no moisture moves to or from the environment) and drying creep (which is caused by drying). It is difficult and not practical to separate one from the other. The total creep is what matters in the structural design of the ground support in the emplacement drift.

Creep of concrete at elevated temperature is influenced by a number of factors besides the applied stress. Factors significant to concrete creep are discussed as follows:

Effect of water/cement ratio - Creep is proportional to the water/cement ratio of the concrete; as the water/cement ratio increases, creep increases. As the water/cement ratio decreases, creep decreases and strength increases (Ref. 5.22). Within the range of normal concrete mixes, creep is also proportional to the amount of hardened cement paste. Hence, in those situations in which it is necessary to keep creep to a minimum, such as in prestressed concrete, it is desirable to use the minimum cement content and minimum water-cement ratio that will produce concrete possessing other desired properties, including adequate strength.

Effect of aggregate - Aggregate in the concrete acts as a restraint to reduce the potential deformation of the cement paste, which is the component that undergoes significant creep. Aggregate content and modulus of elasticity of aggregate are probably the most important parameters influencing creep (Refs. 5.21 and 5.22). The higher the modulus of elasticity and percentage content of the aggregate in the concrete, the greater the restraint offered by the aggregate to the potential creep of the hydrated cement paste. Aggregate size, grading, and surface texture have little influence (Ref. 5.22). Because of the great variations in aggregate within any mineralogical and petrological type, it is not possible to make a general statement about the magnitude of creep for concrete made with various aggregate types. However, it has been found in some tests that creep in concrete specimens with different aggregates can vary as much as five times between the maximum and minimum creep measured (Ref. 5.21).

Effect of admixtures - According to Buil and Acker (Ref. 5.23), for cement containing 25 percent of silica fume, silica fume has no effect on basic creep but significantly reduces the drying creep or shrinkage. For concrete containing fly ash or ground granulated blast furnace slag, long-term creep is reduced because of the long-term hydration process. It is generally stated that admixtures that increase drying shrinkage also increase creep. However, no reliable pattern of influence of these admixtures has been established. A complication in evaluating the effects of admixtures is that their use is often accomplished by changes in water and cement content, which also affect creep. Water-reducing and set-retarding admixtures have been found to increase the basic creep in many, but not all cases. There are indications that lignosulfonate-based admixtures may lead to a larger increase than carboxylic-acid-based admixtures (Ref. 5.24). As discussed in Section 7.5.2.1, silica fume and superplasticizers are proposed for use in the concrete mix design to decrease potential long-term geochemical impacts during the postclosure period.

Effect of curing conditions - The time of moist curing of the concrete prior to loading affects the magnitude of creep. This is to be expected since the degree of hydration is lower for shorter curing times and the porosity of the paste is higher. An increase in temperature of curing reduces both basic and drying creep. The amount of reduction depends on the temperature and its duration, but even quite short periods at elevated

temperatures may cause significant reduction in creep. Steam curing may be a way to achieve this purpose (Ref. 5.6).

Creep tests were conducted on Hanford concrete mixes at 250 and 350 °F (121 and 177 °C) in the PCA study (Ref. 5.14). A total of six specimens (6 x 12 in. or 15 x 30 cm cylinders) were tested for about 650 days under two loadings, 500 psi (3.45 MPa) at 350°F (177 °C) and 1500 psi (10.34 MPa) at 250 and 350 °F (121 and 177 °C). The creep strain data could be accurately modeled by a relatively simple logarithmic expression, $\epsilon = A \log_{10}(t) + B$, where A and B are constants dependent on load, temperature, and material variables. This equation was found to satisfactorily model concrete creep behavior at elevated temperatures by another study (Ref. 5.25).

The predicted creep strain from PCA study (Ref. 5.13) was derived as follows:

$$\epsilon_{cr} = a_0 - \epsilon \quad (7-2)$$

where ϵ_{cr} = predicted creep strain at t days (millionths)

a_0 = average strain readings at zero day, after load applied and test temperature reached (millionths)

ϵ = predicted strain at t days (millionths)

The values of a_0 for the three temperature/loading conditions are 625, 375, and 315 for 500 psi/350 °F (3.45 MPa/177 °C), 1500 psi/ 250 °F (10.34MPa/121 °C), and 1500 psi/350 °F (10.34 MPa/177 °C), respectively.

The fitted equation for ϵ in equation (7-2) for PCA data in terms of their temperature and loading conditions is (Ref. 5.13):

$$\epsilon = 3064.152 - 0.863 L - 6.152 T - 127.526 \log_e (t + 1) \quad (7-3)$$

where T = temperature of test (°F)

L = applied load or stress (psi)

t = time at temperature T (days)

It needs to be pointed out that equation (7-3) should only be applied at or very close to the three load/temperature combinations for which it was developed. Since the temperature and loading conditions for the repository emplacement drift situations are closest to the 1500 psi/350 °F (10.34 MPa/177 °C) case, the value of a_0 of 315 is the most appropriate to be used. However, it needs to be emphasized that creep tests of the proposed concrete mix should be conducted and further work to verify the applicability of this equation should also be done. If any change is found to be necessary, then the modified or newly developed equation for predicting the creep

should be used.

7.3.2 Shrinkage

Shrinkage is the volume decrease of hardened concrete with time. The volume decrease is due to physical-chemical changes in moisture content which occur without stress and are attributable to actions external to the concrete (ACI 209R). Based on the definition of ACI 209R, there are three kinds of shrinkage: (1) Drying shrinkage, due to moisture loss in the concrete, (2) Autogenous shrinkage, caused by the hydration of cement, and (3) Carbonation shrinkage, which occurs when the various cement hydration products are carbonated in the presence of CO₂. It should be noted that the effect of shrinkage on concrete in the emplacement drift will be of short duration compared to the effect of creep or stress relaxation. By the time waste packages are emplaced and heating begins, significant shrinkage, whether of precast concrete or of cast-in-place concrete, will already have occurred.

7.3.2.1 Drying Shrinkage

Drying shrinkage of hardened concrete is much more important than the other two types of shrinkage. Inadequate allowance for the effects of drying shrinkage in concrete design and construction can lead to cracking or warping of the elements of the structure. Especially in the emplacement drift environment, the moisture loss in the concrete may happen due to the elevated temperature conditions. This affects cast-in-place concrete linings much more than precast linings if precast segments were steam cured.

The most important controllable factor affecting shrinkage is the amount of water per unit volume of concrete. In order to minimize the drying shrinkage, the water content of concrete needs to be kept as low as possible. This can be achieved by keeping the total coarse aggregate content of the concrete as high as possible, and using low slump and placing methods that minimize water requirements (Ref. 5.6).

Any practice that increases the water requirement of the cement paste, such as use of high slumps, excessively high freshly mixed concrete temperatures, high fine-aggregate contents, or small-size coarse aggregate, will increase shrinkage. It has been shown that for each 1 percent increase in mixing water, concrete shrinkage increases about 2 percent. The type of cement, cement fineness and composition, and cement content have relatively little effect on drying shrinkage of normal-strength concrete.

Aggregate in concrete, especially coarse aggregate, physically restrains the shrinkage of hydrated cement paste. The type of aggregate also affects the extent of drying shrinkage, which usually can be reduced by avoiding aggregate containing excessive amounts of clay. Quartz, granite, feldspar, limestone, and dolomite aggregate generally produce concretes with low drying shrinkages.

Steam curing will also reduce drying shrinkage (Ref. 5.6).

Many water-reducing type chemical admixtures increase drying shrinkage substantially, particularly those containing accelerator to counter the retarding effect of the admixture. When used in low to moderate amounts, the effect of fly ash, ground granulated blast-furnace slag, and silica fume on the drying shrinkage is generally small and of little significance (Ref. 5.6).

7.3.2.2 Autogenous Shrinkage

If no additional water beyond that added during mixing is provided, it is possible that the concrete will begin to dry out even if no moisture is lost due to surroundings. This can happen in concretes with a low w/c ratio (theoretically below 0.42) and is due to the internal consumption of water during hydration. The phenomenon is known as self-desiccation and leads to autogenous shrinkage (Ref. 5.22).

Autogenous shrinkage tends to increase at higher temperature, with a higher cement content, and with cements which have a high C_3A and C_4AF content (Ref. 5.21). However, it is usually very small, except at extremely low water/cement ratios. For practical purposes (other than in large mass concrete structure), it need not be distinguished from drying shrinkage.

7.3.2.3 Carbonation Shrinkage

Hardened cement paste will react chemically with carbon dioxide. The amount present in the atmosphere (~0.04%) is sufficient to cause considerable reaction with cement paste over a long period of time. This is accompanied by shrinkage. The extent to which cement paste can react with carbon dioxide, and hence undergo carbonation shrinkage, is a function of relative humidity (RH) and is greatest around 50 percent RH. At high relative humidities, carbonation is low because the pores are mostly filled with water and CO_2 cannot penetrate the paste very well. At very low humidity, the absence of water is believed to lower the rate of carbonation. Carbonation shrinkage is greatest when carbonation occurs after drying, rather than during drying, except at low humidities where carbonation shrinkage is greatest when it occurs with drying (Ref. 5.22).

7.3.3 Thermal Properties

Properties needed to characterize the thermal and thermal/mechanical behavior of concrete include thermal expansion, thermal conductivity, and specific heat. The basic heat transfer parameter, thermal diffusivity, is calculated by dividing thermal conductivity by the product of specific heat and density (Ref. 5.27).

7.3.3.1 Thermal Expansion

The coefficient of thermal expansion represents the change of concrete volume or, as usually measured on test specimens, the change in length with change in temperature. It is expressed in terms of percent or millionths per °C or millionths per °F. This property is a fundamental input parameter for calculating the thermally induced deformation or stress. Thermal expansion is a physical phenomenon common to all materials. However, it is complicated in concrete by differential expansion of its constituents, which may produce high internal stresses.

Zoldners (Ref. 5.27) notes that the mineralogical composition and structure of the aggregate are major factors in determining the coefficient of thermal expansion of concrete. Although the hardened cement paste component has a coefficient of thermal expansion that is generally higher than that of the aggregate, the coefficient of thermal expansion of concrete is nearly directly proportional to that of the contained aggregate because the aggregate occupies about 70 to 80 percent of the total volume of the hardened concrete. Zoldners (Ref. 5.27) reports that the main factor influencing the thermal expansion of rock, and therefore concrete, is the proportion of quartz present. Rocks with a high quartz content, such as quartzite and sandstone, have the highest coefficient of thermal expansion, averaging about $12 \times 10^{-6}/^{\circ}\text{C}$. Rocks containing little or no quartz, such as limestone, have the lowest coefficient averaging about $5 \times 10^{-6}/^{\circ}\text{C}$. Rocks with medium quartz content, such as igneous rocks (granite, rhyolite, basalt, etc.) have intermediate coefficients of thermal expansion. An average value of coefficient of thermal expansion of concrete is $9.9 \times 10^{-6}/^{\circ}\text{C}$ (Ref. 5.28).

When cement mortar or concrete is exposed to elevated temperatures, the observed volume change is a result of a complex process consisting of thermal expansion of aggregate and hydrothermal volume change of cement paste. After initial expansion of cement paste, its volume change is reversed to contraction or shrinkage, the rate of which depends on the rate and duration of heating. However, shrinkage of cement paste is overshadowed in mortar and concrete by the aggregate expansion (Ref. 5.27). In general, the coefficient of thermal expansion tends to increase with temperature (Ref. 5.6), with the extent of increase mainly depending on the composition of cement and aggregate. See Section 7.3.1.1 for a discussion of the effect of thermal cycles on the expansion coefficient.

7.3.3.2 Thermal Conductivity

The coefficient of thermal conductivity represents the uniform flow of heat through a unit of thickness over a unit area of concrete subjected to a unit temperature difference between the two faces. Thermal conductivity of concrete is a property which is of primary importance in the design of heat-exposed concrete installations. The conductivity of concrete is determined by conductivities of its constituents (Ref. 5.27). Of all the factors likely to influence the conduction of heat in concrete, are four major variables as follows: (1) conductivity of the cement, (2) conductivity of the aggregate, (3) mix proportion, and (4) compactness of placed concrete. The effect of the concrete's moisture content is another important factor (Ref. 5.27). The thermal

conductivities of paste, aggregate, and concrete are independent of temperature within the normal climatic range. In general, the thermal conductivity of normal-weight concrete ranges from 1.5 to 3.5 W/m·K with an average of 2.5 W/m·K (Ref. 5.22).

Only limited data are available on the effect of elevated temperatures on thermal conductivity of concrete and mortars. Since moisture content decreases with exposure time and temperature, conductivity of mortars and concrete decreases with increasing temperature (Ref. 5.27). Above 100 °C, thermal conductivity decreases linearly with temperature. (Ref. 5.22). Above 400 °C, at which complete dehydration occurs, a gradual disintegration of cement paste takes place, resulting in further decrease in thermal conductivity. Since the maximum temperature at the rock wall of the emplacement drift is 200 °C (Sec. 4.3.4), linear behavior of thermal conductivity versus temperature can be predicted for typical concrete formulations in the emplacement drift environment.

7.3.3.3 Specific Heat

Specific heat is the amount of heat required to raise the temperature of a unit mass of concrete by one degree. Thermal conductivity and specific heat are fundamental input parameters for calculating the temperature rise of the heated concrete. Specific heat, or heat capacity, is little affected by the type of aggregate, since the specific heats of rocks do not change much with mineralogical types. The specific heat of cement paste, however, is strongly dependent on porosity (w/c ratio), water content, and temperature. The specific heat of concrete also depends on these factors. Typical values of specific heat of concretes with normal-weight aggregates range from 840 to 1170 J/kg·K, with an average of 1005 J/kg·K (Ref. 5.22). Little data are available on the effect of temperature on specific heat of concrete. PCA's study on Hanford concrete indicates heating specimens produced an increase in values of specific heat (Ref. 5.14).

7.3.4 Durability

Concrete is inherently a durable material for many construction applications, and with proper design for the expected environment and placement under quality control, concrete is capable of maintenance-free performance for decades without the need for protective measures (Ref. 5.22). However, it is important to examine the factors affecting the durability of concrete for environments beyond normal experience, such as the anticipated repository conditions that include sustained temperatures up to 200 °C (Sec. 4.3.4) and high radiation levels for a service life of 150 years (Section 4.2.1).

7.3.4.1 Permeability

The permeability of concrete plays an important role in durability because it controls the rate of entry of moisture that may contain aggressive chemicals and the movement of water during

heating. Permeability, in turn, is strongly dependent on the water/cement (w/c) ratio of the concrete. As the w/c ratio is decreased, the porosity of the paste is decreased and the concrete becomes more impermeable. The w/c ratio has a dual role to play in concrete durability, since a lower w/c ratio also increases the strength of concrete, and hence, improves its resistance to cracking from internal stresses that may be generated by adverse reactions (Ref. 5.22).

The permeability of mature hardened paste kept continuously moist ranges from 0.1×10^{-12} to 120×10^{-12} cm per sec for w/c ratios ranging from 0.3 to 0.7. The permeability of rock commonly used as concrete aggregate varies from approximately 1.7×10^{-9} to 3.5×10^{-13} cm per sec. The permeability of mature, good-quality concrete is approximately 1×10^{-10} cm per sec. (Ref. 5.6); permeabilities of an order of magnitude lower than this value are considered to represent concretes with a very low permeability (Ref. 5.21). A low-permeability concrete requires a low w/c ratio and an adequate moisture-curing period. Increasing the wet-curing period will reduce the water permeability (Ref. 5.6). However, if the w/c ratios are greater than 0.7, complete discontinuity of capillary pores (which controls the permeability) can never be achieved, even with continuous moisture curing, and such concretes will have relatively high permeabilities (Ref. 5.22).

Various methods are available for determining the permeability of concrete to various substances. Both direct and indirect methods can be used. Resistance to chloride-ion penetration, for example, can be determined by ponding chloride solution on a concrete surface, and at a later stage, determining the chloride content of the concrete at particular depths (ASTM C 1202). The rapid chloride permeability (electrical resistance) test (AASHTO T277) correlates well with permeability and resistance to chloride-ion penetration of concrete (Ref. 5.6).

7.3.4.2 Carbonation

Carbonation of concrete is a process by which CO_2 from the air penetrates the concrete and reacts with the hydroxides, mostly calcium hydroxide, to form carbonates. The major effects of carbonation are: 1) increases shrinkage on drying (see carbonation shrinkage in Section 7.3.2.3) and 2) lowers the alkalinity of concrete (Ref. 5.6).

With respect to durability, the importance of carbonation lies in the fact that it reduces the pH of the pore water in the hardened portland cement paste from between 12.6 to 13.5 to a value of about 9. When all $\text{Ca}(\text{OH})_2$ has become carbonated, the value of pH is reduced to 8.3 (Ref. 5.21). On one hand, higher alkalinity (e.g. $\text{pH} > 11.5$) is needed to protect embedded steel from corrosion, which has more impact on concrete reinforced with rebar than steel-fiber reinforced concrete (see discussion in Section 7.4.4). On the other hand, this low pH value is beneficial in reducing the radionuclide solubility, which is desirable from the long-term geochemical viewpoint during the postclosure period.

The amount of carbonation is significantly increased in concretes with a high water-cement ratio, low cement content, short curing period, low strength, and highly permeable or porous paste. It should be emphasized that the effect of curing on carbonation of concrete is substantial. The absence of wet curing, which results in high porosity, has a detrimental effect on the concrete by increasing the depth of the carbonation. By using the wet curing method, increasing the period of wet curing from 1 day to 3 days reduces the depth of carbonation by about 40% (Ref. 5.21).

At normal dosages, silica fume, ground slag, and fly ash are reported to increase carbonation, but usually not to a significant amount in concrete with short moist-curing periods; they have little effect on carbonation in concrete with long-moist curing periods (Ref. 5.6).

7.3.4.3 Sulfate Resistance

The most widespread and common form of chemical attack is the action of sulfates on concrete (Ref. 5.22). Naturally occurring sulfates of sodium, potassium, calcium, or magnesium are sometimes found in soils or dissolved in groundwater adjacent to concrete structures, and they can attack concrete. Sulfate attack has occurred at various locations through the world, and is a particular problem in arid areas, such as the northern Great Plains area of the United States and the prairie provinces of Canada, and in parts of the western United States (ACI 201.2R). The consequences of sulfate attack include not only disruptive expansion and cracking, but also loss of strength of concrete due to the loss of cohesion in the hydrated cement paste and of adhesion between it and the aggregate particles (Ref. 5.21).

Resistance of concrete to attacks by sulphates is related to the amount of cement in the concrete and the calculated amount of C_3A (tricalcium aluminate) in the cement. The resistance is enhanced for concrete with high cement content and for cement low in C_3A (Refs. 5.6; 5.20). The use of silica fume, fly ash, and ground slag generally improves resistance of concrete to sulfate. Among them, silica fume provides excellent sulfate resistance to concrete, better than fly ash or ground slag in some studies (Ref. 5.6). Another way of reducing sulfate attack that can be used for precast concrete is high-pressure steam curing. Curing at elevated temperatures below 100°C may not improve sulfate resistance, and may even decrease it. Sulfate resistance improves with an increase in curing temperature above 100°C (Ref. 5.22).

Type II portland cement was selected in the ESF invert for providing increased resistance to sulfate attack (Ref. 5.31), which is conservative based on the SO_4^{2-} content value of J-13 well water (Sec. 4.3.8) from the saturated zone below the repository horizon. Since sulfate resistance related to the mechanical properties of concrete is a long-term consideration, use of Type V portland cement appears to be more desirable, because it has high sulfate resistance due to its low C_3A content. Especially, sulfate attack is a particular problem in arid areas such as in parts of the western United States (ACI 201.2R), and the SO_4^{2-} content within the porewater in the rock mass is much higher in the unsaturated zone above (see Table 9.1 of Ref. 5.46) the repository (Sec.

4.3.1). Moreover, from a long-term geochemical viewpoint, the reflux after boiling water due to the elevated temperature (i.e., higher than 100 °C) after waste emplacement could bring the higher concentration of SO_4^{2-} content from the unsaturated zone to the repository horizon.

The resistance of concrete to chemical attack is increased by allowing it to dry out before exposure following proper curing methods. A thin layer of calcium carbonate (produced by the action of CO_2 on lime) is then formed, blocking the pores and reducing the permeability of the surface zone. It follows that precast concrete is generally less vulnerable to attack than concrete cast in situ (Ref. 5.21) since better quality control and curing can be ensured in the precast method.

7.3.4.4 Alkali-Aggregate Reaction

Chemical reactions of aggregates in concrete can affect the performance of concrete. The reaction that has received greatest attention involves a reaction between alkalis (Na_2O and K_2O) from the cement, or from other sources, with hydroxyl, and certain siliceous constituents that may be present in the aggregate. This phenomenon was originally, and is still sometimes, referred to as "alkali-aggregate reaction", but in recent years it has been more properly designated as "alkali-silica reaction" (ACI 201.2R). The alkali-silica reaction generally forms reaction products that can cause excessive expansion and cracking or popouts in the concrete (Ref. 5.6).

The minerals usually associated with this reaction are opal, chalcedony, tridymite, cristobalite and certain zeolites. The typical rocks that contain these minerals are rhyolites, dacites and andesites (including volcanic tuffs) and cherts. Any aggregate containing a significant portion of any of these materials should be considered potentially reactive (Ref. 5.20).

It is desirable to try to avoid the susceptible aggregate. However, it is usually not a practicable or an economical solution. If a reactive aggregate must be used, there are several measures that can be taken to minimize the reaction. Most important is to use a low-alkali cement. A limit of 0.6 percent total alkali in the cement is commonly specified (Refs. 5.20, 5.21, 5.22; ACI 201.2R); 0.4 percent if a highly reactive aggregate is used (Ref. 5.20). Since the Yucca Mountain tuff contains most of the reactive materials, it will be further tested to determine its potential reaction with portland cement, and thus its suitability for use as concrete aggregate.

No adverse expansion will occur when external moisture is not available (Ref. 5.22). Alkali reactivity can be significantly reduced by keeping the concrete as dry as possible and the reactivity can be virtually stopped if the internal relative humidity of concrete is kept below 80% (Ref. 5.6). A low w/c concrete is very impermeable and may also help to limit the supply of water needed to cause the reaction (Ref. 5.6).

Use of pozzolans such as silica fume, fly ash, and ground granulated blast-furnace slag also can

significantly reduce the alkali-silica reaction. However, pozzolans with high water-soluble alkali contents should be avoided as they may increase reactivity. The quantity and type of mineral admixture should be determined by test for a particular concrete mixture (see ASTM C441). The alkali-silica reactivity can be tested by ASTM C1293 or ASTM C227 (alkali-silica reaction) (Ref. 5.6) in conjunction with petrographic analyses.

7.3.4.5 Biological Effects

The chemical attack by sulfates on concrete occurs either in direct contact with sulfidic soils or rock or through groundwater where anaerobic sewer conditions prevail. In this subsection, the effect of sulfate attack by bacteria action is addressed.

Because biotic sulfide production requires anaerobic conditions, oxygen that is present must be eliminated by rapid metabolic activity or by limiting its transport, or both. Rapid metabolic activity uses substantial amounts of readily metabolizable organic carbon or carbon dioxide and other required nutrients. It follows that bacterial sulfide production is maximized in high-organic, saturated environments. A further requirement is a large or renewable sulfate reservoir. Without a large or renewable sulfate reservoir, sulfide production slows and ultimately ceases as the sulfate ion is consumed.

The conditions that promote sulfide-ion production were examined to assess the likelihood that sulfuric acid will be generated in the waste emplacement drifts. The waste emplacement drifts and surrounding area are not saturated. Additionally, organic carbon concentrations are low or nonexistent, depriving sulfate-reducing bacteria (SRB) of its necessary carbon source. Because the fundamental requirements for bacterially mediated sulfide production do not exist in the host formation, it is extremely unlikely that sulfur-metabolizing bacteria will colonize the area (Ref. 5.15, p. C-18). Consequently, only the potential for sulfide generation within the waste emplacement drifts must be evaluated.

The waste emplacement drifts lack significant metabolizable organic carbon. Organic materials are restricted for use in the emplacement drift (Sec. 4.3.5). Additionally, minimal water is anticipated within the waste emplacement drifts, which will prevent large volumes of saturated material from developing. In addition, radiation will develop immediately after waste emplacement, followed by lethal heating of the drift.

The combined radiation and heat will pose lethal challenges to all bacteria (Ref. 5.15, p. C-21). The heat will dry the drifts, eliminating a water source for the bacteria. Even the most thermophilic bacteria cannot withstand 160°C temperatures. Temperatures exceeding 80°C will develop in the drifts within nominally 25 years after waste emplacement, preventing any bacterial activity for the remainder of the waste retrieval period (Ref. 5.15, p. C-21). It is generally accepted that the upper temperature boundary for bacterial activity is 120-150°C (Ref.

5.29). Consequently, even in an improbable worst-case condition, bacterial degradation can only occur for a short period (e.g., 20 years) after waste emplacement.

7.3.4.6 Resistance to Abrasion

The abrasion resistance of concrete is defined as the "ability of a surface to resist being worn away by rubbing and friction" (ACI 201.2R, Ch. 3). Abrasion of tunnel invert may result from equipment operations and waste package transportation. In the case of precast concrete segments, impact loading due to the handling, transportation, and erection process of the concrete segments may also cause rubbing and friction to the concrete surface and induce cracking.

Test results indicate that abrasion resistance is closely related to the compressive strength of concrete. Strong concrete has more resistance to abrasion than does weak concrete. Since compressive strength depends on water-cement ratio and curing, a low water-cement ratio and adequate curing are necessary for abrasion resistance. The type of aggregate and surface finish or treatment used also have a strong influence on abrasion resistance. Hard aggregate is more resistant than soft aggregate.

The addition of steel fiber to the concrete will increase the ductility, the energy absorption capacity to inhibit the cracking caused by the handling and erection of the concrete segments.

It is difficult to assess the abrasion resistance of the concrete, since the damaging action depends on the exact cause of wear, and no one test procedure is satisfactory in evaluating all the conditions. Abrasion tests can be conducted by rotating steel balls, dressing wheels, or disks under pressure over the surface (ASTM C 779) or by sandblasting (ASTM C 418) (Ref. 5.6).

7.3.4.7 Resistance to freezing and thawing

Since the potential repository at Yucca Mountain is not subjected to freezing and thawing effects, and there is no need to have deicing chemicals, therefore, the problem of deterioration of concrete caused by freezing and thawing does not exist. This is the reason that air-entraining admixtures were not used in the precast concrete for the ESF invert. For the same reason, air-entraining admixtures will not be used in the concrete for the repository ground support (Sec. 4.3.12).

7.4 COMPONENTS FOR CONCRETE

An evaluation of suitable materials for concrete components to be used for repository emplacement ground support is presented in this section based on the discussion in Sections 7.2 and 7.3.

7.4.1 Cement Type

In designing concrete materials for ground support systems suitable for repository emplacement drift, one of the most important factors is to select the proper cement type to meet the design functions and requirements. In order to be used in repository emplacement drifts, concrete must have adequate strength, low permeability, low heat of hydration, and durability during the 150-year (Sec. 4.2.1) preclosure period. It should also be acceptable from the long-term geochemical viewpoint during the postclosure period.

There is a wide variety of cements that are used to various extents in the construction and building industries, or to solve special engineering problems. The chemical compositions of these cements can be quite diverse, but by far the greatest amount of concrete used today is made from portland cements. During this screening process, both portland cements and other special cements such as C_2S cements (belite rich cements) and expansive cements have been considered.

C_2S cements were produced at the turn of the century to make extremely good concrete, which had higher C_2S content than portland cements produced today. This old cement is generally considered to be equal to, or superior to, modern fast-reacting cements. An important feature of this cement is that it produces 1/3 less Ca than C_3S , which may be favorable for the potential repository at the Yucca Mountain. However, due to its slow strength development, it has been replaced by the modern cements (mainly portland cements). Today, some research groups in European countries have pursued the development of this belite-rich cements, and in some countries cements of this type are being used (Ref. 5.5, Attachment V). Despite the fact that this type of cement has certain advantages, inability of belite cements to become a commercial alternative to conventional portland cement reflects the high development costs and time that will be incurred to bring any of these cements to the market place in North America. Therefore, C_2S cements are not being considered as the candidate cement types for the current study.

The expansive cements, or alkali-activated cements, were also considered in the screening process. This type of cement is used to produce what is known as shrinkage compensating concrete (Ref. 5.30). It has the advantage of high reactivity (i.e., expands upon hydration) compared with most commercial cements. However, it has two major drawbacks: 1) it is not resistant to sulfate or acid attack, and 2) the cost is relatively higher than the conventional portland cement. In addition, this type of cement has not been used for a long time compared with portland cements. The proven long-term performance of this type of cement is not certain compared with that of portland cements. Because of the above reasons, expansive cements are not considered further as candidate cement types for the current study.

Based on the above discussions and the proven performance of portland cement in concrete construction and in the ESF, it is desirable to select portland cement with the addition of silica material as the candidate material for the concrete to be used in the repository emplacement

drifts. Different types of portland cements can be manufactured to meet various physical and chemical requirements for the current project. A summary of their typical physical and chemical properties is given in Table 7.1.

Type II portland cement was selected in the ESF invert for providing increased resistance to sulfate attack (Ref. 5.31). The need for resistance to sulfate attack was based on the SO_4^{2-} content of J-13 well water (Sec. 4.3.8) from the saturated zone below the repository horizon. Since sulfate resistance related to the mechanical properties of concrete is a long-term consideration, use of Type V portland cement may be more desirable since it has higher sulfate resistance. In addition, it also has the following advantages compared with Type II portland cement:

- higher C_2S (lower calcium/higher silica) content, which will produce lower pH value of cement paste solution, and will be closer to the host rock composition due to its higher silica content (see silica content in Table 4.1 in Sec. 4.1.1).
- less heat of generation and temperature increase (Ref. 5.30, p. 46).
- less amount of superplasticizers needed for workability when silica fume is used in the concrete due to its lower C_3A content (Ref. 5.32).

Table 7.1 Typical chemical and compound composition of portland cements (Ref. 5.6).

Types of portland cement	Chemical compositions, %						Compound composition, %			
	SiO_2	Al_2O_3	Fe_2O_3	CaO	MgO	SO_3	C_3S	C_2S	C_3A	C_4AF
Type I	20.9	5.2	2.3	64.4	2.8	2.9	55	19	10	7
Type II	21.7	4.7	3.6	63.6	2.9	2.4	51	24	6	11
Type III	21.3	5.1	2.3	64.9	3.0	3.1	56	19	10	7
Type IV	24.3	4.3	4.1	62.3	1.8	1.9	28	49	4	12
Type V	25.0	3.4	2.8	64.4	1.9	1.6	38	43	4	9

7.4.2 Aggregate

The importance of using the right type and quality of aggregates cannot be overemphasized since the fine and coarse aggregates generally occupy 60% to 75% of the concrete volume (70% to 80% by weight) and strongly influence the concrete's freshly mixed and hardened properties,

mixing proportions, and economy. Fine aggregates generally consist of natural sand or crushed stone with most particles smaller than 0.2 in. (5 mm). Coarse aggregates consist of one or a combination of gravels or crushed aggregate with particles predominantly larger than 0.2 in. (5 mm) and generally between 3/8 and 1.5 in. (19 and 38 mm) (Ref. 5.6).

Aggregates should be hard and strong and free of undesirable impurities. Soft, porous rock can limit strength and wear resistance; it may also break down during mixing and adversely affect workability by increasing the amount of fines. Aggregates should be free of impurities: silt, clay, dirt, or organic matter. No particular rock or mineralogical type, in itself, is required for aggregate, as long as it conforms to the requirement of ASTM C33. Therefore, if they are available, aggregates from local sources should be preferable, which would usually save a large amount of cost in transportation and time. There exists another important factor in selecting local aggregates for use in the emplacement drifts, i.e., the concrete should resemble the host rock of the repository as closely as possible to ensure the chemical compatibility between aggregates and host rock from the long-term performance viewpoint. Therefore, it is desirable to use tuff aggregates (Sec. 4.3.1) from TBM muck after proper processing (i.e., crushing, washing, screening, etc.) has been performed.

Aggregates are usually inert. There is, however, one situation which needs extra attention: the alkali-aggregate reaction, now called alkali-silica reaction. In this reaction, certain minerals and rocks react with the alkalis in cement causing an internal expansion in concrete and causing deterioration. As discussed in Section 7.3.4.4, the minerals usually associated with this reaction are opal, chalcedony, tridymite, cristobalite and certain zeolites; the rocks are glassy or cryptocrystalline rhyolites, dacites and andesites (including volcanic tuffs composed of these rocks) and cherts, both chalcedonic and opaline.

It is desirable to try to avoid the susceptible aggregate. However, that is usually not a practical or an economical solution. If a reactive aggregate must be used, there are several measures that can be taken to minimize the reaction. Most important is to use a low-alkali cement. A limit of 0.6 percent total alkali is commonly specified (Refs. 5.20, 5.21, 5.22; ACI 201.2R); 0.4 percent if a highly reactive aggregate is used (Ref. 5.20). Use of pozzolans such as silica fume, fly ash, and ground granulated blast-furnace slag also can significantly reduce the alkali-silica reactivity. Limiting the supply of water needed to cause the reaction with a low w/c ratio may also help reduce this alkali-aggregate reaction.

The fine aggregate should meet the gradation requirement given in ASTM C33. Its fineness modulus shall not be less than 2.3 nor more than 3.1 (ASTM C33). The maximum size of the coarse aggregate influences the paste requirements of the concrete and the optimum grading of the coarse aggregate. Usually, coarse aggregates should be graded up to the largest size practical under the job conditions. Larger sizes also minimize the water requirement and thus allow the cement content to be reduced. For high compressive strength concrete, 6000 psi (41.4 MPa) plus

with a cement content exceeding 600 lb per cubic yard (356 kg/m^3), the optimum maximum size is about 3/4 in. (19 mm) (Ref. 5.6). Since steel fiber is to be used for the current concrete mix design, the maximum size of coarse aggregate should not be greater than 3/4 in. (19 mm) (ACI 544.3R). To avoid pumpability problem for cast-in-place construction, the maximum size of coarse aggregate may need to be 1/2 in. (13 mm) or less.

7.4.3 Pozzolanic Materials

Based on PA's recommendation, a pH of about 10 or lower (Sec. 4.3.9) is desirable to ensure the long-term geochemical stability of the repository system (Ref. 5.5). In order to mitigate the potential problems associated with high pH solutions above 13 (could reach up to 13.5) with ordinary portland cement (see Section 7.2.1), cement for concrete can be engineered by adding pozzolans in the form of silica fume, fly ash or blast furnace slag, to reduce the pH values. There are certain advantages and disadvantages to these pozzolans. However, silica fume has the most advantages compared with the other ones, and it is selected to be used in the concrete for the emplacement drift ground support. The most important advantages of silica fume are listed as follows:

- Has the highest silica (SiO_2) content (93%) compared with that of fly ash (50%) and blast furnace slag (35%) (see Table 8.1 of Ref. 5.33).
- Has the finest and purest material among the three pozzolans (Ref. 5.6).
- Is very effective in lowering the permeability of concrete with enhanced durability (ACI 211.1; Ref. 5.6).
- Provides excellent sulfate resistance to concrete, better than fly ash or blast furnace slag (Ref. 5.6).

Since the application of silica fume is a very important factor in the cementitious material to be used for emplacement drift ground support, the major features are discussed in the following sections.

7.4.3.1 General

Consolidated silica fume, also referred to as silica fume or micro-silica, is a by-product of the manufacture of silicon, ferrosilicon, or the like, from quartz and carbon in electric arc furnaces (Ref. 5.33). The silicon dioxide (SiO_2) content can vary from 70 to 96%, increasing in percentage as the amount of silicon increases in the ferro silicon metal manufactured. Table 7.2 shows typical chemical compositions resulting from the production of various types of silicon alloys (Ref. 5.34). Tables 7.3 and 7.4 show the required and optional chemical compositions for

silica fume based on ASTM C 1240. Silica fume is a superfine material with a particle size of the order of 0.1 micron and a surface area of over 15,000 m²/kg (a hundred times greater than cement or fly ash). Its relative density (specific gravity) is similar to that of fly ash at about 2.3 but, owing to its extreme fineness, it has a very low bulk density of only 200 to 250 kg/m³ in its loose form. For this reason, it is usually handled either in a densified form with bulk density of 400 to 500 kg/m³ or as a 50/50 slurry with either water or a superplasticizing admixture (Refs. 5.33 and 5.35).

Table 7.2 Chemical Composition of Silica Fume From Different Alloy Types (wt%)
(Ref. 5.34)

Chemical Component	Silicon Metal	Ferro-Silicon Metal (75%)	Ferro-Silicon Metal (50%)
SiO ₂	95	90.5	84
C	1.3	1.5	1.9
Fe ₂ O ₃	0.3	1.6	2.4
Al ₂ O ₃	0.7	0.9	2.5
Na ₂ O	0.3	0.6	0.7
K ₂ O	0.3	0.7	1.3
MgO	0.2	1.4	2.0
SO ₃	0.8	0.9	1.1
CaO	0.3	0.5	0.8
LOI*	1.5	1.6	3.3

* LOI: Loss of Ignition

Table 7.3 Chemical Requirements for Silica Fume Based on ASTM C 1240

SiO ₂ , min, %	85.0
Moisture Content, max, %	3.0
Loss on Ignition, max, %	6.0

Table 7.4 Optional Requirements^A for Silica Fume Based on ASTM C 1240

Available Alkalies as Na ₂ O, max, % ^B	1.50
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^A Applicable only when specifically required by purchaser.

^B Applicable for silica fume to be used for concrete containing reactive aggregates with cement required to meet a limitation on alkali content.

7.4.3.2 Effect of Silica Fume on Compressive Strength and Young's Modulus

The actual strength level attainable is dependent upon many other factors (notably coarse aggregate characteristics), but in many instances silica fume permits the easy attainment of compressive strength in excess of 100 MPa (14500 psi) when, for highly workable concrete, 80 MPa (11600psi) might be difficult to attain without it (Ref. 5.33).

The actual action of silica fume in increasing the strength of concrete appears to be partly chemical and partly physical. Chemically, due to its superfine and highly reactive form, its pozzolanic reaction with the free calcium hydroxide released by hydrating cement is very effective. The physical effect of densification, and of improving the structure of the cement paste at its interface with the coarse aggregate, has been considered to be of similar magnitude to the chemical effect (Ref. 5.33).

The effect of condensed silica fume on the compressive strength of concrete can be seen from Figure 7.3 (Ref. 5.36). It clearly shows that for the same w/c ratio, the higher the silica fume content, the higher will be the compressive strength. It also indicates that to achieve 28-day 70 MPa (10150 psi) compressive strength for a 100 mm cube, the w/c ratio required is about 0.35 if no silica fume is used; however, with 8% silica fume, the w/c needed is about 0.50, and with 16% silica fume content the w/c requirement increases to about 0.65. It also indicates that higher compressive strength can be achieved with higher silica fume content at relatively higher w/c ratios.

Comprehensive investigations in Norway have shown that there are no significant differences between the Young's modulus (E) (or modulus of elasticity) of concrete with and without silica fume. However, it is known that "E" values do not continue to increase with increasing compressive strength of concrete. Therefore, very high-strength concretes tend to be more brittle. This is equally true of high-strength silica fume concrete. Data on Young's modulus of portland cement/blast furnace slag (BFS)/silica fume concrete indicate that, regardless of the various percentages of silica fume contents and W/(C+BFS) ratios, there is no significant difference between the "E" values obtained at 28 days. There was some indication that the highest values were obtained for concrete containing 10% silica fume (Ref. 5.36). The study by Buil and Acker (Ref. 5.23) indicated that for concrete containing 25% silica fume, the compressive strength increase is 43% whereas the Young's modulus increase is only 7% compared with that without silica fume.

7.4.3.3 Effect of Silica Fume on Pore-Solution Chemistry of Cement Pastes

Analysis of the pore solution in portland cement paste is one of the methods for investigating the mechanism of hydration and the possible long-term effects of portland cement containing admixtures and additives.

The influence of silica fume on the pH of the solution has been studied to assess the corrosion potential. As shown in Figure 7.4 (Ref. 5.36), the plain cement paste shows a pH of about 13.9. Addition of silica fume results in a progressive decrease in pH. A corresponding reduction in the total available alkali metal cations in solution is also observed. However, even at an addition of 10 to 20% silica fume the pH value does not drop below that of a saturated $\text{Ca}(\text{OH})_2$ solution which is approximately 12.5. Even at 30% silicate fume, the pH does not drop below 11.5, which is considered to be a threshold value for maintaining a good passivity of embedded steel in concrete (Refs. 5.35 and 5.36).

The pore solutions expressed from hydrated portland cement pastes containing up to 30% silica fume as replacement for cement were analyzed by Diamond (Ref. 5.37). The results indicated that the presence of silica fume enhances early concentrations of alkalies and hydroxyl ions, but after 1 day the effect is reversed, and the concentrations of these ions are progressively reduced to low values, sometimes of $\text{pH} < 12.5$. For the water/solid ratio of 0.5, the lowest calculated pH was 12.2. Alkali reduction seems more pronounced for pastes of higher water content, and low levels of silica fume replacement (5 or 10%) yield disproportionately large alkali removal effects.

7.4.3.4 Effect of Silica Fume on Concrete Creep and Durability

No published data are available on creep of concrete containing 5 to 15% of silica fume. However, Wolsiefer (Ref. 5.38) has reported limited data on concrete incorporating a proprietary mixture which is principally silica fume (20% by weight of cement). Creep measurements of this high-strength (111.4 MPa or 16153 psi compressive strength at 28 days) concrete were made after 7 days and every month after that up to 1 year. The creep results showed very low creep values (120×10^{-6} to 480×10^{-6} for 7 days to 12 months). Since the data are for very high-strength concrete, it should not be compared with those of compressive strength in the order of 40 to 60 MPa at 28 days. However, in another study (Ref. 5.23) in which creep tests have been performed on a concrete with 25% of cement replaced by silica fume and with compressive strength of 76 MPa, the results showed reduction in drying shrinkage, without any significant reduction in creep.

The addition of silica fume to concrete can provide a previously unattainable level of low permeability in addition to the chemical conversion of the most vulnerable calcium hydroxide into durable calcium silicates. It gives a physical uniformity of cement paste structure through avoiding bleeding effects and creating a smaller scale gel structure. In addition, any tendency of

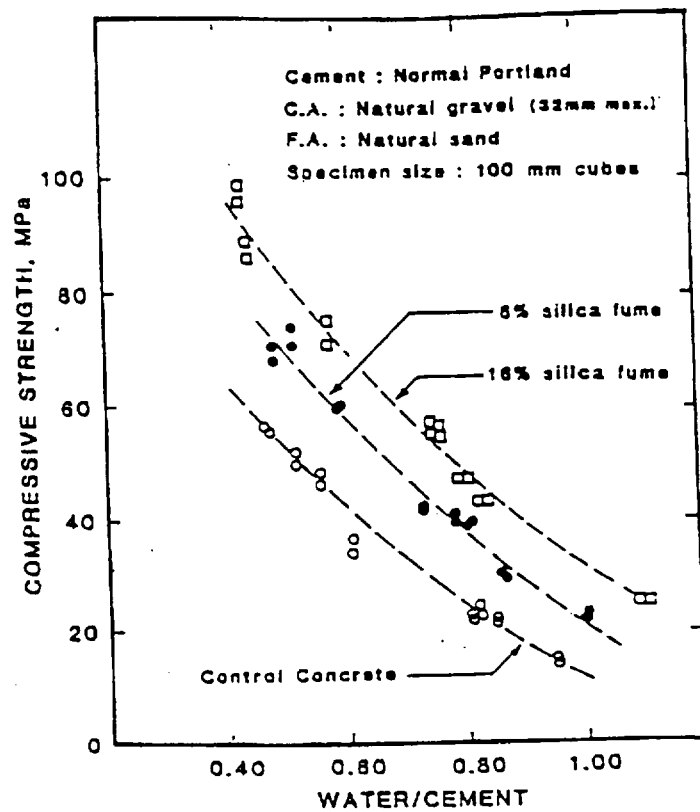


FIGURE 7.3 Relation between compressive strength of silica fume concrete and W/C ratio. (Ref. 5.36)

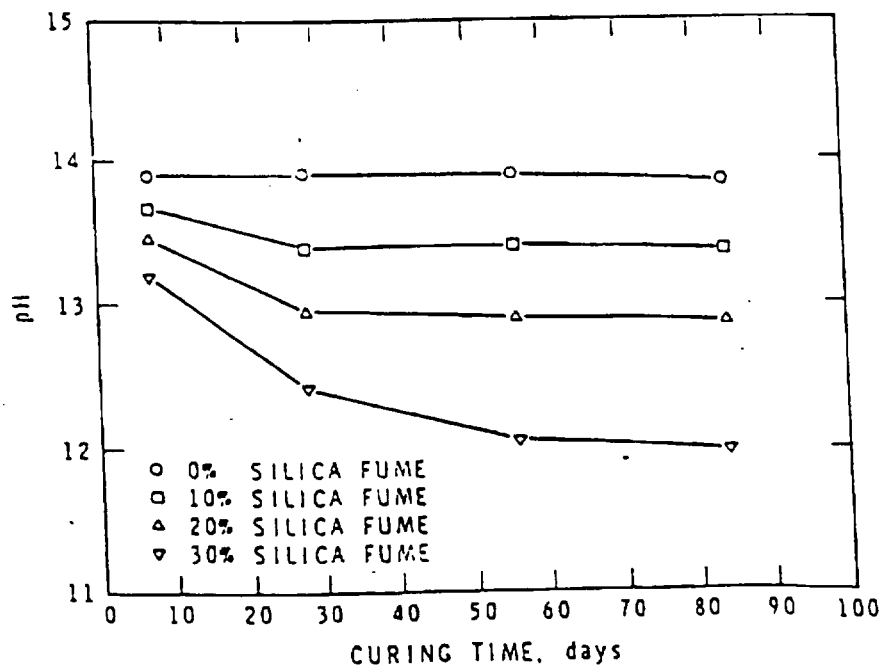


FIGURE 7.4 Influence of silica fume content on pH values of cement pastes cured for different periods. (Ref. 5.36)

the coarse aggregate to alkali-silicate reaction will be forestalled since the alkalis will be consumed in a non deleterious diffused reaction with the silica fume (Refs. 5.6, 5.33).

The combined effect of these factors is to provide a new degree of resistance to sulphates, chlorides and general aggressive chemicals (Ref. 5.33). In addition, silica fume has much lower electrical conductivity, which will provide greater resistance to steel corrosion (Refs. 5.33, 5.34).

Wolsiefer et al. (Ref. 5.39) have conducted a study on the performance of concretes incorporating various forms of silica fume. Several conclusions from the results of that investigation, which are important to the current study, are as follows:

- Regardless of the product form, the mechanical properties and durability characteristics of the silica fume concretes were comparable.
- There was no significant difference in the mechanical properties and durability characteristics of concretes incorporating different product forms of silica fume with SiO_2 ranging from 79 to 95 %. Lower SiO_2 levels did not require higher silica fume dosages for comparable performance.
- The Resistance to Chloride-Ion Penetration (RCP) test results indicated that the RCP values of all the silica fume concrete tests, (12 % SF and 0.30 W/C+SF), were below 300 coulombs, indicating very low permeability of the concretes.
- Decreasing the water-to-cementitious materials ratio and increasing the silica fume percentage decreased the RCP values.
- The drying shrinkage strains of the silica fume concretes made with the different product forms of the fumes were low, at about 500×10^{-6} after 448 days of air drying.
- Almost all silica fume concretes exhibited no carbonation after 28 days of moist curing and 448 days of air drying.

7.4.3.5 Effect of Elevated Temperature on Concrete Containing Silica Fume

The loss of concrete strength by heating to different temperatures is due to the differential thermal expansion between the cement paste and aggregate, and due to the resulting physical and chemical changes.

Theoretically speaking, the condensed silica fume toughens the portlandite (calcium hydroxide) in high-strength pastes, but after heating to 320 °C the condensed silica fume particles which have been converted to C-S-H lose their ability to toughen the portlandite because of the partial decomposition of C-S-H at 300 °C (Ref. 5.36).

Studies have been done on concrete prisms exposed to sustained temperatures ranging from 75 to 300 °C for a period of 60 days. Ultrasonic pulse velocity and weight loss determinations on test prisms before and after sustained heat exposure show that concrete incorporating 15 % condensed silica fume by weight of cement performs a little worse (about 0.5 to 2.0% more weight loss) as compared with ones without condensed silica fume. This was so for both non-air-entrained and air-entrained concrete (Ref. 5.36).

In some other studies on exposure of condensed silica fume concrete to high temperature and fire, several small cylinders of very low $W/(C+SF)$ (<0.20) and containing 20 % silica fume suddenly disintegrated at 300 °C (Ref. 5.36). However, this type of behavior is not entirely unusual for ultra-high strength and impermeable concrete because under slow heating, high vapor pressure can build up inside a specimen.

Since the temperature in the repository drift environment during the postclosure period is not expected to be greater than 200 °C (Sec. 4.3.4), the effect of elevated temperature on strength of concrete containing silica fume should be very minimal or insignificant. Nevertheless, it is desirable to conduct some mechanical tests on concrete containing silica fume exposed to temperatures of 200 °C (Sec. 4.3.4) to better understand its behavior.

7.4.4 Steel fiber

Although steel fiber reinforced concrete has been known for many years, its introduction to tunnel linings is relatively recent. However, due to its special features, the application of steel fiber in concrete tunnel linings has been increasing steadily. The most recent ones are the Jubilee Line Extension (JLE) in London (Ref. 5.40), in which precast concrete segments using steel fiber reinforcement were used as one of the four segment lining methods. Steel fiber reinforcement was used at the rate of 30 kg/m³ of concrete. To meet the requirement of a minimum compressive strength of 50 N/mm² (7250 psi) and a design life of 120 years, the designed concrete mix has a water-cement ratio of 0.4 (Sec. 4.3.10) and a slump of 70 mm (2.75 in.).

The use of steel fiber in the concrete lining for the emplacement drift is not for the structural reinforcement, rather, it is mainly for the supplementary role to inhibit cracking during handling, transportation and erection; and to increase ductility, the energy absorption capacity and the ultimate strain capacity of the concrete. Thus, it is desirable to use fibers (≥ 20 mm) at a low percentage ($\leq 3\%$) (Ref. 5.41).

The amount of steel fiber should be chosen so that the strength and toughness meets specification. The selection of 0.5% by volume of concrete is based on engineering judgment and experience from common practice.

The steel fibers with hooked ends are desirable because they can improve the bond and

anchorage in the concrete and increase the reinforcing efficiency and ductility. The steel fibers used in the Jubilee Line Extension are Dramix ZC 60/80 (Ref. 5.40) (see more information on Dramix steel fibers in Attachment II), which have hooked ends and are glued together in bundles with a water soluble adhesive. When the glue is dissolved by water in the mix, the fibers will be separated as individual fibers. The collating of fibers reduces balling considerably (Ref. 5.42). One important note about adding the steel fibers to the concrete mix is to use a dispensing system, which is necessary to have random but uniform distribution of steel fibers in the concrete. It should also be noted that, although the fibers are of steel, their behavior when attacked by corrosion is quite different and not as destructive as rebar corrosion. This is because the fibers in the concrete are non-continuous and discrete and are protected by the alkaline concrete matrix which provides a defence against the propagation of corrosion (Ref. 5.40).

The toughness index of fiber reinforced concrete is a measure of the amount of the energy required to deflect a 102-mm prism by a given amount (1.9 mm) compared to the energy required to bring the prism to the point of first crack. It is calculated as the area under the load-deflection curve up to a deflection of 1.9 mm divided by the area under the load-deflection curve of the prism up to the first crack. Toughness index values twice that of ordinary fiber reinforced concrete have been reported for some fiber reinforced silica fume concrete. In general, the toughness index of fiber reinforced silica fume concrete is higher than that for concrete without silica fume (Ref. 5.36).

7.4.5 Water-Reducing Admixtures

Water-reducing admixtures are used to reduce the quantity of mixing water required to produce concrete of a certain slump, reduce water-cement ratio, or increase slump. Water reducing admixtures conforming to ASTM C494 Type A or D usually reduce the water content by approximately 5% to 10% (Ref. 5.6).

Water reducers can be divided into three categories based on the general composition of their active ingredients. These are 1) lignosulfonates, 2) hydroxylated carboxylic acids, and 3) carbohydrates (Ref. 5.6). It should be pointed out that they are all organic materials. Most conventional water-reducing admixtures belong to the first two categories (Ref. 5.22). In the construction of the ESF, a commercial water reducer, Polyheed, was used in the precast concrete mix design with the typical dosage of 125 oz (3700 ml) for 752 lb (341 kg) of cement (about 17 oz/cwt or 1080 ml per 100 kgs of cement) (Ref. 5.43). Polyheed is an aqueous solution of a lignosulfonate and additional ingredients for improved performance and finishability. It is a mid-range (5-18%) water reducer suitable for a wide concrete slump range. In addition to having the strength performance comparable to chloride-bearing, water-reducing admixtures at all ages, it exhibits improved performance with a wide range of cements, fly ashes, silica fumes, granulated slags, and aggregates (including coarse and manufactured sand) (see detail in Attachment II).

High-range water reducers (also called superplasticizers) conforming to ASTM C494 Type F or G generally reduce water content by 12% to 30% (Ref. 5.6). Superplasticizers are often used to produce flowing concrete with slumps about 7-1/2 in. (19 cm) or more with no increase in water demand other than that contained in the admixture (ACI 211.1). By using superplasticizer, it is possible to increase concrete workability and not need any additional water.

Since a large amount of silica fume (greater than 10% of cement by weight) (see Sec. 7.5.2.2) is proposed to be used in the concrete, a great amount of high-range water reducer has to be used to achieve the required workability due to the very high specific area of the silica fume. The addition of steel fibers also reduces the workability, which can also be restored with addition of an appropriate amount of superplasticizer (ACI 544.1R; Ref. 5.42).

There are basically, three principal types of superplasticizers: 1) lignosulfonate-based, 2) polycondensate of formaldehyde and melamine sulfonate (often referred to simply as melamine sulfonate), and 3) polycondensate of formaldehyde and naphthalene sulfonate (often referred to as naphthalene sulfonate) (Refs. 5.6 and 5.35).

Among these three types of superplasticizers, lignosulfonate superplasticizers are generally used in conjunction with either melamine or naphthalene superplasticizers when desiring high strength concrete. Until recently, only one melamine superplasticizer was available (Ref. 5.35). Naphthalene superplasticizers, on the other hand, have been in use longer any of the others, and are available under a great number of brand names. They are generally available as either calcium salts, or more commonly, sodium salts. The particular advantages of naphthalene superplasticizers, apart from their being slightly less expensive than the other types, appears to be that they make it easier to control the rheological properties of high strength concrete because of their slight retarding action (Ref. 5.35). It is also indicated in one study that naphthalene superplasticizer admixtures do not seem to influence the properties of hardened concrete containing fly ash and exposed to high temperatures (Ref. 5.44).

There is no *a priori* way of determining the required superplasticizer dosage; it must be determined by some sort of trial and error procedure. If one should work with the lowest w/c ratio possible, then the highest superplasticizer dosage rate should be used. On the other hand, if the required strength with high w/c is desirable, the dosage of superplasticizer can be adjusted to get the desired workability. For example, Rheobuild 1000, a commercial superplasticizer, is an aqueous solution of sulfonated naphthalene superplasticizer. The recommended dosage is at a rate of 10 to 25 fl. ozs per 100 lbs (650 to 1600 ml per 100 kgs) of cementitious materials, depending on the application (see more information in Attachment II). For fiber-reinforced concrete, a slightly higher dosage of superplasticizer is needed to achieve the same workability when compared to a corresponding superplasticized concrete without fibers (Ref. 5.45).

For the current study, a moderate to large amount of silica fume (see Sec. 7.5.2.2) needs to be

used to reduce the pH value of the cement paste solution, which, in turn, indicates a need to use a great amount of superplasticizer to achieve the workability. However, since all the current commercial superplasticizers are organic materials, the addition of plasticizer or superplasticizer will probably introduce organic materials to the repository drift environment, which may not be acceptable from the post-closure performance viewpoint.

Because the postclosure emplacement drift will be very dry at temperatures above 100°C for an extended period of time, it is postulated that under this dry and elevated temperature environment, the organic admixtures in the concrete will be decomposed and volatilized as gases, mostly CO₂ gas. In order to confirm the above postulation, it is necessary to conduct some basic laboratory tests on cement-paste samples prepared with different types (2 or 3) of candidate commercial superplasticizers. The objectives of these tests are to investigate 1) evolution of the organic admixtures in the hydrated cement paste, and 2) evolution of the hydrated phases under dry and elevated temperature conditions. Most importantly, the result of these tests will provide very important information about whether the organic materials from the concrete can be reduced substantially, and acceptable to PA's requirement for long-term postclosure performance of the repository.

7.4.6 Silica Flour

Silica flour (basically pure quartz) may be used for alternative concrete mix designs. It is used mainly as a filler to provide better material gradation.

7.5 CONCRETE MIX DESIGN

7.5.1 Introduction

The objective of this section is to present a preliminary concrete mix design based on information presented in previous sections and a review of the literature on concrete formulations. It is not the intention to determine the final mix design at this stage. However, it provides a preliminary concrete mix design, which, though it cannot be used for construction, will serve as a basis for subsequent evaluations of concrete.

Repository Design is considering concrete for use as permanent ground support. Specifically, precast and cast-in-place concrete linings are being analyzed as options for ground support in the emplacement drifts. ESF experience with concrete is limited to the use of precast concrete for the invert segment. To date, performance of the invert segment under ambient conditions has been satisfactory. In addition, the concrete mix design that was used for the ESF inverts is also being used for a test concrete lining section for a full-scale heater test scheduled to commence near the end of this calendar year. The ESF concrete, containing Type II cement, and described in Reference 5.43 is used herein as a reference concrete or a starting point for the development of

a concrete mix specifically designed to fulfill repository needs.

For the repository, criteria for ground support concrete include preclosure strength and durability at sustained temperatures up to 200°C (Sec. 4.3.4) and a service life of 150 years. Another consideration is a criterion regarding the ultimate pH of the postclosure repository drift environment. It is assumed that by careful proportioning and specification of the composition of concrete used in the emplacement drift, an optimum pH can be achieved. However, factors other than concrete pH, such as waste package oxidation, radiation, and microbial activity, can affect the ultimate pH. Also, the effort to produce a concrete mix that minimizes pH has to be balanced with the need to provide a concrete that meets preclosure ground support criteria. Therefore, a postclosure criterion for concrete is to achieve as low a concrete pore solution pH as reasonably possible in order to minimize possible detrimental affects on waste isolation.

7.5.2 Selection and Proportioning of Components

Concrete mixtures should be kept as simple as possible, as an excessive number of components often make a concrete mixture difficult to control. In general, the proportioning of concrete mixtures consists of two interrelated steps: (1) selection of suitable components (cement, aggregate, water content, and admixtures) and (2) determination of the relative quantities ("proportioning") to produce concrete with the appropriate workability, strength, and durability.

7.5.2.1 Selection of Components

Cement

As discussed in Section 7.4.1, the basic principle in the concrete mix design is to select cement types to meet the design function and requirement. Type V portland cement is proposed for the following reasons:

- Higher silica content - Higher C_2S (lower calcium/higher silica) content compared with Type II portland cement, which will produce a lower pH value and a composition closer to the host rock composition.
- Minimizes use of organic admixtures (Sec. 4.3.5) - Due to its lower C_3A content compared with Type II portland cement, a lesser amount of superplasticizer is needed for workability when silica fume is used in the concrete (Ref. 5.32).
- Provides sulfate resistance - Type II portland cement is used in the ESF invert for providing increased resistance to sulfate attack (Ref. 5.31), which is a conservative design based on the SO_4^{2-} content of J-13 well water (Sec. 4.3.8) from the saturated zone below the repository horizon (Sec. 4.3.1). However, the sulfate concentration in the unsaturated zone

where the repository would be located is higher (see Table 9.1 of Ref. 5.46) giving a somewhat greater potential for corrosion. Sulfate resistant portland cement will be required even if the groundwater contains sulfate in low concentration. Particularly, if the concrete will be subject to wetting and drying cycles by the water movement (this may take place in the repository), evaporation of the water from concrete surfaces will lead to the deposition and accumulation of sulfates in the concrete at a concentration higher than that in the groundwater. Such processes can significantly accelerate sulfate attack. Type V cement provides a high level of sulfate resistance that is appropriate for the long service life required of repository concrete.

- Improved curing - Heat of generation and temperature are less than those from Type II portland cement.

Aggregate

Tuff aggregate (TSw2) from TBM muck is proposed, based on the recommendation of consultants to the PA group (Ref. 5.5). The tuff aggregate should enhance compatibility between concrete and the host rock (Sec. 4.3.1). If found acceptable, use could be made of stockpiled tuff from repository excavation. Because the tuff contains certain amounts of tridymite, cristobalite, and zeolite, requirements for concrete formulation and production, as is the case with any aggregate, will also depend on results of alkali-silica reactivity testing (Ref. 5.20). By using cement with a limit of 0.6 percent total alkali, the potential of alkali-silica reactions will be minimized.

Admixtures

Air-entraining admixtures are not suggested for use in the concrete since there is no freezing-thawing effect in the repository environment (Sec. 4.3.12). Such admixtures were not used in the ESF concrete (Reference 5.43).

Water-reducing admixtures could be required in the concrete mix design to reduce the water content. Typically, water reducers lower the water content by approximately 5% to 10% (Ref. 5.6). It was used in the precast concrete for the ESF invert. It is very reasonable to use it for the repository emplacement drift.

High-range water reducers (also called superplasticizers) are mainly used for increasing the workability due to the use of a large amount of silica fume in the concrete. Superplasticizers are also of use to restore workability, which is reduced by the addition of steel fibers.

Silica Fume

Silica fume is proposed to be used in the concrete mix mainly to reduce the pH values of cement paste pore solution and to increase the strength of concrete as well. In addition, silica fume increases the resistance to corrosion and consequently the potential durability and longevity of the concrete by increasing strength, density (reduces permeability), and preventing alkali-reactivity. It gives a physical uniformity of cement paste structure by avoiding bleeding effects and creating a small scale gel structure. The combined effect of these factors is to provide increased resistance to sulphates, chlorides and general aggressive chemicals (Ref. 5.33). Any concern about increased carbonation and drying shrinkage due to the use of silica fume can be mitigated with proper and longer moist-curing periods (Ref. 5.6).

Silica Flour

Silica flour (basically pure quartz) may be used for alternative concrete mix design (see Option 2 in the next section). It is used mainly as a filler to provide better material gradation.

Steel Fibers

Steel fibers may be used in the concrete mix. The use of steel fiber in a concrete lining inhibits cracking during handling, transportation and installation, by increasing ductility, the energy absorption capacity and the ultimate strain capacity of the concrete. In this case it is desirable to use fibers (≥ 20 mm) at a low percentage ($\leq 3\%$) (Ref. 5.41). Although steel fiber reinforced concrete has been known for many years, its introduction for tunnelling industry is relatively recent and only limited to a few installations. For the Jubilee Line Extension project in London, steel-fiber reinforced concrete was used for some sections with the dosage of 30 kg/m^3 of concrete (Ref. 5.40).

As discussed in Section 7.4.4, the use of steel fiber in the concrete lining for the emplacement drifts is not for structural reinforcement. Steel fiber is used instead of steel rebar in the concrete to reduce spalling and corrosion problems. Since steel fibers are distributed homogeneously in the concrete and are present close to the surface, they ensure excellent reinforcement against spalling. The fibers in the concrete are non-continuous and discrete and are protected by the alkaline concrete matrix, which provides a defense against the propagation of corrosion (Ref. 5.40).

The specification for the steel fiber proposed to be used in the concrete for the current study is as follows:

Type: Type I, deformed fibers (ASTM A 820)

Dimensions: 50/0.6, 60/0.8, or 30/.50 (for cast-in-place) for length/diameter in mm

Steel fiber content: 0.5 % by volume of concrete

7.5.2.2 Proportioning of Components

Because the use of silica fume aids in reducing the potential pH of the concrete system, two concrete mix designs are considered to help in understanding how to achieve this goal. Option 1, containing a moderate amount of silica, has a normal proportion of concrete mix with 10 to 20 percent of silica fume by weight. Option 2 contains a relatively high amount of silica, in the form of silica fume and silica flour, and requires more than normal amounts of superplasticizer.

In common practice (ACI 211.1), the amount of silica fume, as a percentage of cement by weight, is usually not greater than 15 percent, except in cases where it is desired to enhance certain properties, such as increase in strength. To increase strength, the quantity of silica fume is increased. However, concrete containing relatively large proportions of silica fume may become sticky and unworkable, requiring the addition of organic superplasticizers to increase workability. At present, all the commercial superplasticizers are organic materials, which are to be avoided due to postclosure, waste isolation concerns.

Based on the current evaluation of cementitious material from Performance Assessment (Sec. 4.3.9; Ref. 5.5), a target pH value of about 10 or lower is desirable from the postclosure performance viewpoint. It should be noted that this value is a preliminary recommendation. Moreover, a concrete with a cement paste pH value of 10 is difficult to achieve by the addition of silica fume alone. Even with 30 percent silica fume, the pH value of the cement pore solution will not drop below 12 (Refs. 5.36, 5.37). However, there are other actions that may reduce pH values in the cement paste solution during the postclosure period: microbial activity, the oxidation of carbon steel from the waste package, radiation effect on water chemistry, and the carbonation effect. Although there are some uncertainties regarding these in reducing pH, it appears reasonable to take them into consideration. In other words, even though the starting pH value of the cement paste solution may be above 10, the long-term pH value during the postclosure period might drop below 10 due to those potential actions mentioned above.

Option 1 (Moderate Silica)

In this option, a preliminary proportion or composition of concrete mixture by using silica fume of 15 percent (which is the average of the range considered) by weight of cement is developed. The concrete mixture proportion is derived by following the procedures in ACI 211.1 "Standard Practice for Selecting Proportions for Normal, Heavyweight, and Mass Concrete". Since the assessment on potential postclosure performance impacts from cementitious materials has not been completed by the Performance Assessment group and the final ingredients have not been determined, the concrete mix proportions presented here are preliminary in nature. The absolute volume method (see ACI 211.1 Chapter 6) is used for the derivation of the mix design since it is more accurate than the weight method (Sec. 4.3.11) (Refs. 5.6, 5.22).

The proportions of concrete mix design are derived based on the following conditions:

- minimum compressive strength at 28 days (f'_c): 5000 psi (34.5 MPa) (Sec. 4.3.14)
- Slump: 3 to 4 in. (7.6 to 10.2 cm)
- Silica fume: 10 to 20 % by weight of cement (15% used in calculation), with specific gravity of 2.2 (Sec. 4.3.15)
- Air-entraining admixtures: not used, since there is no freezing-thawing effect in the repository environment (Sec. 4.3.12)
- Cement: ASTM C150 Type V portland cement with specific gravity of 3.15 (Sec. 4.3.15)
- Coarse aggregate: 3/4" (19 mm) maximum size of tuff rock from selected TBM muck (Ref. 5.50), with an oven-dry specific gravity of 2.68, absorption of 0.5%, and an oven-dry rodded unit weight of 100 lb per cu ft (1.60 g/cm³). The laboratory sample for trial batch has a moisture content of 1%. (Sec. 4.3.16). It should be noted that the 3/4" (19 mm) maximum size is mainly for precast construction. For cast-in-place concrete, a smaller size such as 1/2 in. (12.7 mm) may be required.
- Fine aggregate: Same source as coarse aggregate with an oven-dry specific gravity of 2.64, an absorption of 0.7%, and a fineness modulus of 2.8. The laboratory sample for trial batch has a moisture content of 3%. (Sec. 4.3.16)
- Target water/cement ratio: 0.4 (Sec. 4.3.10)
- Water-reducing admixture: 10 oz/cwt (650 ml/100 kg) of cementitious material (typical value, see Attachment II for Polyheed)
- Superplasticizer: 25 oz/cwt (1600 ml/100 kg) of cementitious material (typical value, see Attachment II for Rheobuild 1000)
- Steel fiber: 0.5% of concrete by volume (Sec. 7.5.2.1) with specific gravity of 7.850 (Sec. 4.3.15)

It should be noted that unit weights; absorption level of aggregate; specific gravities of cement, silica fume and steel fiber; and fineness modulus of fine aggregate are values assumed for normal practice. These parameters should be determined before starting the mix design process. Table 7.5 shows the preliminary composition of concrete mixture for repository drifts based on the above assumptions (the step-by-step calculation is shown in Attachment I). It was calculated

Table 7.5 Preliminary Composition of Precast Concrete for Repository Emplacement Drifts
(Option 1)

Cement: Type V portland cement, 670 lb/cu yd (398 kg/m³)

Water (to be added): 270 lb/cu yd (160 kg/m³)

Aggregate

Coarse Aggregate (1% MC): 3/4" (19 mm) max size, tuff aggregate, 1690 lb/cu yd (1003 kg/m³)

Fine Aggregate (3% MC): same source as coarse aggregate, 1310 lb/cu yd (777 kg/m³)

Admixtures

Silica Fume: 100 lb/cu yd (59 kg/m³)

Water-Reducing Admixture: Polyheed or similar type, 10 oz/cwt (650 ml/100 kg) (or 5 lb/cu yd) of cementitious material

Superplasticizer: Rheobuild or similar type, 25 oz/cwt (1600 ml/100 kg) (or 12 lb/cu yd) of cementitious material

Steel Fiber

Dramix ZC or similar, 0.5 % by volume of concrete, i.e., 66 lb/cu yd (39 kg/m³)

Note: For cast-in-place mix design, it may need to have the maximum coarse aggregate size at ½ in. (12.7 mm) and adjust the volume of coarse and fine aggregate to increase the workability.

based on assumed conditions. Some assumptions may change depending on the test results, local conditions, special specifications or requirements. Consequently, the composition of concrete and the type of ingredients may also change. In addition, methods for proportioning and evaluating concrete mixtures containing those supplementary admixtures, such as silica fume, water-reducing admixture (e.g., Polyheed), and superplasticizer (e.g., Rheobuild), must be based on trial mixtures using a range of ingredient proportions. By evaluating their effects on strength, water requirement, time of set, and other important properties, the optimum amount of admixtures can be determined. In particular, the appropriate dosage of silica fume needs further investigation, because one of the major reasons to add silica fume in the concrete mix is to lower the pH value of the cement pore solution. However, the real mechanisms and reaction rates of many factors that potentially affect the pH values in the cement paste solution during the postclosure period are not fully understood.

Based on 15 percent of silica fume in the cement content and the low water/cement ratio (e.g. < 0.6), the 28-day compressive strength should be greater than 10000 psi (69.0 MPa) (see Fig. 7.3), or 7000 psi (48.3 MPa) with 30 percent reduction to account for the elevated temperature condition, i.e., less than 200 °C (Sec. 4.3.4), which meets the strength requirement. Notice that due to the strength development (increase) after 28 days, the ultimate compressive strength of concrete before the heat is applied will be greater than that at 28 days, the above mentioned 7000 psi (48.3 MPa) is a conservative estimate. The permeability of the concrete based on this mix design is expected to be very low, probably less than 500 coulombs when measured in accordance with the procedure of AASHTO T277.

Option 2 (High Silica)

In this option, a special design of concrete mix with less cement content, a larger amount of silica fume and silica flour, and a higher dosage of superplasticizer is proposed, which may yield a starting pH value below 10 (Sec. 4.3.9). The potential problem with this mix design is that it uses a relatively large amount of superplasticizer (i.e., organic material), which may not be acceptable due to waste isolation concerns.

A concrete with a cement content of 97 kg/m³ (i.e., 164 lb/cu yd) has been developed at AECL-WL (Canada) (Ref. 5.5). This Low-Heat-High-Performance Concrete (LHHPC) was specifically developed for high mass structures that are being proposed for use in the geological isolation and disposal of radioactive wastes in Canada. The mix proportion of this concrete is presented in Table 7.6 (Ref. 5.6). This new material has an unconfined compressive strength of 100 MPa (14500 psi) at 90 days after casting, and exhibits a maximum temperature increase of only 15 °C during hydration. It also possesses the additional advantages of high-performance concrete such as low permeability and low pH in the range of 7.8 to 9.5, which may be attributed to the low cement content. Moreover, the cement content was decreased in the mix by partial replacement of the cement with inert silica flours and silica fume. The silica flour is basically quartz crystal,

which is used as a filler between the aggregate and is not reactive. The silica fume reacts with portlandite. Consequently the content of $\text{Ca}(\text{OH})_2$ in the hardened material will decrease, and the pH value of the cement paste solution is reduced.

By comparing the mix proportion and the characteristics of LHHPC and those from Option 1, it appears that it may be desirable to make some adjustments to the LHHPC's proportion as follows:

- The pH value does not need to be so low; it may be increased a little by increasing the cement content from 97 to about 200 kg/m^3 (340 lb/cu yd).
- Since the cement content is approximately doubled, the amount of silica fume and silica flour needs to be reduced, probably to about half of the original values.
- The amount of superplasticizers also needs to be reduced due to the reduction of silica fume.
- The amount of coarse aggregate in LHHPC is about 4% greater than that from Option 1; it needs to be reduced to about the same amount as that of Option 1. The amount of fine aggregate needs to be adjusted depending on the volume of the other components.
- The water content in LHHPC is much less than that from Option 1. It is probably due to the small amount of cement used. Since the cement content will be increased, the water content should also be increased to ensure the cement hydration and workability of the concrete.
- The steel fiber will be added to the concrete mix. Its amount may need to be adjusted in this new mix design.

Table 7.6 Mix Proportions in LHHPC* (Ref. 5.5), kg/m^3 (lb/cu yd)

Materials	Low Heat High-Performance Concrete (LHHPC)
Portland Type V Cement (Canadian Type 50)	97.02 (164)
Silica Fume	97.02 (164)
Silica Flour	193.85 (329)
Fine Aggregate	894.74 (1517)

Coarse Aggregate	1039.59 (1762)
Superplasticizer	10.32 (17)
Water	91.86 (156)

* LHHPC is protected under US Patent 08/384,645.

- Tuff aggregate instead of granite aggregate will be used. It is assumed that there will be no compatibility problem between tuff aggregate and cement based on the new mix design.

As stated earlier, the Option 2 mix design is merely conceptual without the exact estimate of concrete components. It provides an alternative mix to accommodate the lower pH values required of the cement paste solution during the postclosure period. Since there are many uncertainties involved in the evolution of concrete during the long-term postclosure performance, it is beyond the scope of this study to investigate this issue. It should also be noted that since there is change in the composition compared with that of LHHPC, different physical and mechanical properties may develop. Further testing is needed to determine the strength, durability of the concrete and pH value of the cement paste solution.

7.5.2.3 Estimate of Quantities of Concrete Components Per Linear Meter of Lining

An estimate of quantities of concrete components per linear meter for the Option 1 mix design is calculated in this subsection. Figure 7.5 shows the cross-sectional view of the typical emplacement drift in the current repository design. The excavation diameter of the drift is 5.5 meters and the concrete lining is 200 mm thick (Sec. 4.3.13).

The cross-sectional area of the concrete in Figure 7.5 can be divided into two parts: the lining and the invert.

- (1) Area of Lining (excluding the invert), A_1

$$A_1 = \pi/4 (5.5^2 - 5.1^2) (360-84)/360 = 2.55 \text{ m}^2$$

- (2) Area of Invert, A_2

$$A_2 = \text{Circular Sector}_{ABC} - \text{Triangle}_{ADE} - \text{Area}_{GHI}$$

$$\text{Circular Sector}_{ABC} = \pi/4 (5.5^2) (84/360) = 5.54 \text{ m}^2$$

$$AF = 2750 - 542 - 313 = 1895 \text{ mm} = 1.895 \text{ m}$$

$$\text{Triangle}_{ADE} = (2.55) (\sin 42^\circ) (1.895) = 3.23 \text{ m}^2$$

$$\text{Area}_{GHI} = 1/2(1.50 + 2.222) (0.542) = 1.01 \text{ m}^2,$$

$$\text{Thus, } A_2 = 5.54 - 3.23 - 1.01 = 1.30 \text{ m}^2$$

The total area of the concrete is $2.55 + 1.30 = 3.85 \text{ m}^2$. The volume per linear meter of concrete liner is therefore 3.85 m^3 or 5.04 cu yd ($1 \text{ m}^3 = 1.31 \text{ cu yd}$). It should be noted that the calculated concrete volume is only an estimate based on configuration in Figure 7.5. It also does not account for allowance for overbreak and void filling.

The estimated quantities of concrete components per cubic yard are shown in Table 7.5. For 5.04 cu yd (3.85 m^3) of concrete per linear meter of liner, the corresponding quantities of concrete components are shown in Table 7.7

7.5.3 Curing

Curing is the maintenance of a satisfactory moisture content and temperature in concrete during some definite period immediately following placing and finishing, so that desired properties may develop. Curing has a strong influence on the properties of hardened concrete such as durability, strength, watertightness, abrasion resistance, volume stability, and resistance to freezing and thawing and deicer salts (Ref. 5.6). Concrete must be properly cured if its optimum properties are to be developed. Concrete structures rarely fail because the specified strength is not attained, but inadequate strength at the time the forms are stripped may cause a problem.

With proper curing, concrete will become stronger and more durable. The improvement is rapid at early stages but continues more slowly thereafter. The most effective curing method for concrete depends on circumstances. For most jobs, normal curing (i.e., moist curing) is adequate, but in some cases, special care is needed.

Due to the relatively long time required to produce mature concrete, the possibility of accelerating strength development by steam curing needs to be investigated. Since concrete in the emplacement drift will be subjected to high temperature (higher than 100°C) for a long period, it may be desirable to apply steam curing to the concrete to precondition it to the high-temperature environment (Ref. 5.5). However, such curing is feasible only for precast segments.

According to ACI, steam curing procedures may vary, but in general, it depends on the time allotted for each of the following operations:

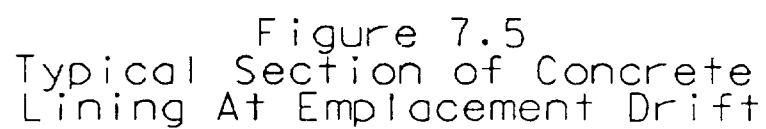


Table 7.7 Estimate of Quantities of Concrete Components per Linear Meter of Lining*
(Option 1)

Cement: Type V portland cement, 3380 lb (1530 kg)

Water (to be added): 1360 lb (617 kg)

Aggregate

Coarse Aggregate (1% MC): 3/4" (19 mm) max size, tuff aggregate, 8520 lb (3864 kg)

Fine Aggregate (3% MC): same source as coarse aggregate, 6600 lb (2990 kg)

Admixtures

Silica Fume: 500 lb (227 kg)

Water-Reducing Admixture: Polyheed or similar type, 25 lb (11 kg)

Superplasticizer: Rheobuild or similar type, 60 lb (27 kg)

Steel Fiber

Dramix ZC or similar, 0.5 % by volume of concrete, i.e., 330 lb (150 kg)

* based on 5.04 cu yd or 3.85 m³ per linear meter of lining

- Presteaming period
- Temperature rise period
- Maximum temperature period.

The ACI recommends the heating of concrete should not begin sooner than 1 to 7 hr after casting while some investigations suggest that longer presteaming times may increase long-term strength (Ref. 5.14). The temperature rise period is the time required to reach the maximum cure temperature from ambient conditions. The ACI recommendation is that the rate of heating should not exceed a temperature increase of 20 °F/hr (11 °C/hr) (Ref. 5.14, p. 2-16). The maximum temperature period is the time concrete is exposed to the highest curing temperature. Generally, this time is 24 hr or less, but it is increased to as much as 3 days for some applications. Maximum temperature is usually in the range of 130° to 180°F (54 to 82 °C). Higher temperatures are not recommended. Maximum temperatures should not exceed 150 °F (66 °C) if steam curing exceeds 24 hr. For longer curing times, a maximum temperature of 130 °F (54 °C) may yield high strength at later stages than curing at 160 °F (71 °C) for the same length of time (Ref. 5.14).

Since steam curing is recommended by the cement experts (Ref. 5.5) for the concretes for the current project, it is necessary to further investigate its effects on the concrete strength and durability by conducting steam curing tests on the concrete specimens prepared based on the proposed mix design. Another technology recommended by the cement experts is to apply pressure during the first several hours after mixing (Ref. 5.5), which can be a very effective way of reducing its permeability by compacting the concrete. Notice that this method is only applicable to precast concrete.

7.5.4 Workability

Workability is the property of freshly mixed concrete or mortar which determines the ease and homogeneity with which it can be mixed, placed, compacted, and finished. For hardened concrete to be of an acceptable quality, the fresh concrete must satisfy the following requirements (Ref. 5.22):

1. It must be easily mixed and transported.
2. It must be uniform throughout a given batch and between batches.
3. It should have flow properties such that it is capable of filling completely the forms for which it was designed.
4. It must have the ability to be compacted fully without an excessive amount of energy being applied.
5. It must not segregate during placing and consolidation.
6. It must be capable of being finished, either against the forms or by means of trowelling or other surface treatment.

Workability is very important in controlling the quality and construction speed for cast-in-place concrete used for ground support in the drifts, due to long haul distances and limited working space in the drifts. Cast-in-place concrete must be relatively fluid, and it must gain strength rapidly under ambient conditions. Moreover, silica fume and steel fiber, which usually reduce the workability, will be added to the concrete. Certain types of dispensing equipment for steel fiber and on-site vibrations will be necessary to achieve the desired workability using the cast-in-place method. This is one of the reasons that the precast concrete method is the technique preferred by the cement experts (Ref. 5.5).

A large number of workability tests have been proposed over the years, almost all of them completely empirical. Only a few of these tests have been incorporated into standards (Ref. 5.22). Among them, the slump test is by far the oldest and most widely used test of workability. For given proportions of cement and aggregates, the higher the slump, the wetter the mix. The test apparatus and test procedure of the slump test are described in ASTM C143.

For the selected concrete mix design, enough concrete must be mixed for a slump test. If the results of the slump test are not acceptable, then appropriate adjustment needs to be made to the batch samples.

7.6 CHARACTERISTICS OF STEEL

Steel, in addition to concrete, is an important construction material for repository ground support (Section 4.3.5). In this section, the possible long-term effects on steel properties of such factors as elevated sustained temperature, radiation, and geochemical processes are discussed.

7.6.1 Mechanical Properties

7.6.1.1 Strength and Modulus of Elasticity

The yield point of structural steel (carbon or low-alloy) generally decreases linearly from its value at 20 °C to about 80 percent of that value at 430 °C, and to about 70 percent at 540 °C (Ref. 5.28). By interpolation, the value at 200 °C (Sec. 4.3.4) is about 91 percent of that at 20 °C. The modulus of elasticity of structural steel decreases from an initial value of 200 GPa (29,000 ksi) at about 20 °C to about 172 GPa (25,000 ksi) at 480 °C (Ref. 5.28, p. 9-67), or 86 percent of the room-temperature value. By interpolation, it will be 189 GPa (27,400 ksi) at 200 °C (Sec. 4.3.4), which is about 5% decrease in comparison with that at 20 °C.

These results are similar to those reported by the American Institute of Steel Construction (Ref. 5.47). The AISC notes that the yield strength of elevated-temperature carbon steel at 430 °C is approximately 77 percent of room-temperature strength; at 540 °C, yield strength is 63 percent of room temperature strength. In contrast, a report on the elevated-temperature properties of ferritic

steels (Ref. 5.48, p. 617) states that carbon steels are used extensively in pressure vessels up to about 370 °C, and given the yield and ultimate strength of carbon steels at the maximum service temperature (370 °C), they can be used essentially as they would for design of components at room temperature. Creep is not observed in these steels until temperatures are above 370 °C (Ref. 5.48, p. 622).

Based on the above information, carbon steels at 200 °C (Sec. 4.3.4) may experience modest, but insignificant, decreases in strength (about 9 percent) and modulus of elasticity (about 5 percent) in comparison to these same parameters at 20 °C.

7.6.1.2 Toughness and Ductility

Toughness is the ability of a metal to absorb energy and deform plastically before fracturing. A measure of toughness is notch toughness, which is measured (in joules) by impact testing. Toughness generally decreases as the strength, hardness, and carbon content of steel are increased (Ref. 5.48, p. 739). At 200 °C (Sec. 4.3.4) the notch toughness of a steel with 0.11-percent carbon is about six times that of a steel with 0.80-percent carbon. The 0.80-percent carbon steel exhibits the least ductility of the carbon steels and has the highest transition temperature from brittle to ductile behavior. For maximum toughness and ductility, the carbon content should be kept as low as possible, consistent with strength (Ref. 5.48, p. 739). The brittle-ductile transition temperature and the carbon content are the principal factors in determining the appropriate toughness and ductility for steel. Based on this information, steel conforming to ASTM A36, which is that used for ESF ground support (Ref. 5.49), should perform satisfactorily in the anticipated repository environment.

7.6.2 Thermal Properties

Properties needed to characterize the thermal and thermal/mechanical behavior of steel include thermal expansion, thermal conductivity, and specific heat. The basic heat transfer parameter, thermal diffusivity, is calculated by dividing thermal conductivity by the product of specific heat and density (Ref. 5.27).

7.6.2.1 Thermal Expansion Coefficient

Structural steels (i.e., carbon steels) have a coefficient of thermal expansion that varies from about $11.5 \times 10^{-6}/^{\circ}\text{C}$ at 20 °C to $13.8 \times 10^{-6}/^{\circ}\text{C}$ at 200 °C (Ref. 5.48, Figure 58, p. 652) (Sec. 4.3.4). The thermal expansion coefficient for tuff for near-field considerations is shown to vary from about $5.07 \times 10^{-6}/^{\circ}\text{C}$ at 25 °C to $8.97 \times 10^{-6}/^{\circ}\text{C}$ at 200 °C (Secs. 4.3.4, 4.3.7). These data show differences in expansion coefficients between tuff, (or concrete with a tuff aggregate), and steel decrease from about $6.5 \times 10^{-6}/^{\circ}\text{C}$ at 25 °C, to about $5 \times 10^{-6}/^{\circ}\text{C}$ at 200 °C (Sec. 4.3.4). Because differences in expansion coefficients are relatively small and the rate of repository

heating is expected to be slow, the effects of differential expansion are anticipated to be minor.

For steel-fiber reinforced concrete, the effect due to different thermal expansion coefficients is insignificant because steel fibers are discretely and uniformly distributed in the concrete.

7.6.2.2 Thermal Conductivity and Specific Heat

The average thermal conductivity of the carbon steel (grade 1025) for temperature 0 to 200 °C (Sec. 4.3.4) is 50.6 7 W/m·K ranging from 51.9 to 49.0 W/m·K (Ref. 5.48, p. 197), with higher values for lower temperatures. The average specific heat of the carbon steel (grade 1025) for temperature 50 (Sec. 4.3.3) to 200 °C (Sec. 4.3.4) is 502.5 J/kg·K ranging from 486 to 519 J/kg·K with higher values for higher temperatures (Ref. 5.48, p. 198). It should be noted that the grade and specification of carbon steel has not been determined. Grade 1025 carbon steel is used here only for illustrating the general trend of thermal conductivity and specific heat. As stated in the section for concrete, these two thermal properties are fundamental input parameters for calculating the temperature rise of heated steel.

7.6.3 Durability

7.6.3.1 Corrosion

Steel sets are susceptible to corrosion under certain environmental conditions. When oxygen is present, rusting can occur. Rusting is an abiotic electrochemical process that requires a flow of electrical current for the chemical corrosion reaction to proceed. For electrochemical (galvanic) corrosion to occur, two dissimilar metals must come into electrical contact in the presence of moisture (Ref. 5.15, p. C-3).

Corrosion occurs in localized anodic areas. Local anodic and cathodic areas are caused by several conditions, including different impurity levels in the steel, different amounts of residual strain, or different concentrations of oxygen or electrolyte in contact with the metal (Ref. 5.15, p. C-3).

It was indicated in M&O ACD report (Ref. 5.15, p. C-4) that steel corrosion in mines is often caused by the sulfuric acid generated by the oxidation of ore-bearing and pyritic sulfide phases. These sulfuric acid solutions are extremely corrosive to steel. The corrosion potential is enhanced if soluble copper is present in the acid solutions (as copper plates out on the steel), causing the dissolution of iron (for example, corroded train rails in underground copper mines). This type of aggressive corrosion will not occur in the waste emplacement drifts due to the absence of sulfide phases in the host formation and copper plate on the steel sets. Moreover, the production of bacterial sulfide is almost impossible except for a short time after waste emplacement, which is explained below (see biological effects in Sec. 7.6.3.2).

There are many ways to prevent steel corrosion. A common practice is to apply protective coatings. Metal coatings, such as zinc galvanizing, are often used to protect steel from corrosion. However, zinc may react with concrete materials to produce hydrogen gas, which is not desirable in the drift. Moreover, the long-term behavior of this coating under high temperature is not known. It is not advisable to apply protective coatings to the steel sets in the emplacement drifts. Based on the observation of steel sets in the ESF, no real corrosion occurred, but some local oxidation was noticed, which might have occurred before the steel sets were installed. Since the repository emplacement drift environment is not conducive to corrosion due to its high temperature, minimum presence of water, and very low amount of chloride, the effect of corrosion on the steel strength should be insignificant during the service life of 150 years (Sec. 4.2.1).

7.6.3.2 Biological Effects

In aqueous, oxygen-free, reduced environments, the lifetime of steel and iron material is diminished by sulfur-reducing bacteria (SRB). Although the deleterious effects of SRB have been demonstrated in both laboratory and natural settings, special conditions that do not exist at the Yucca Mountain repository are required for SRB to corrode steel. In addition to the anaerobic, aqueous environment, these conditions require the availability of sulfate, an electron acceptor, and a carbon source (Ref. 5.15, p. C-10).

Steel biodegradation occurs when SRB consumes hydrogen during sulfate reduction. Iron immersed in water releases Fe^{++} cations, and the metal surface becomes negatively charged by the remaining electrons. The dissolving process continues only if the electrons are removed, for example by an oxidizing agent. Under aerobic conditions, oxygen acts as an electron acceptor and rust is formed. Under anaerobic conditions, the electrons left on the metal surface reduce protons, from the dissociation of water to hydrogen, which forms a protective layer over the submerged iron surface. SRB oxidize the elemental hydrogen with sulfate as the electron acceptor. Removal of the hydrogen protons by SRB disrupts the natural equilibrium and causes cathodic depolarization of the iron surface (Ref. 5.15, p. C-10). The presence of liquid-phase water is critical to this process. The combined radiation and heat will pose lethal challenges to all bacteria. Elevated temperatures in the emplacement drifts will ultimately eliminate liquid water, eliminating a water source for the bacteria. Even the most thermophilic bacteria cannot withstand 160°C temperatures. Temperatures exceeding 80°C will develop in the drifts within nominally 25 years after waste emplacement, preventing any bacterial activity for the remainder of the waste retrieval period (Ref. 5.15, p. C-21).

The presence of oxygen, excessively high temperatures, and inadequate sources of sulfate and carbon create an environment hostile to SRB. In the absence of the specialized environment required for SRB metabolism, corrosion of steel by SRB is not expected to occur.

7.6.3.3 Effects of Radiation

Radiation hazards from the waste packages will come from different radiation types including alpha-particles, beta-particles, and neutrons and photons (gamma- and x-rays). The primary radiation from the waste package is neutron and gamma because the alpha and beta radiation are stopped by the disposal container (Ref. 5.15, p. C-9). Neutrons and gamma radiation can produce ionization when they pass through materials because the energy of these particles can eject electrons from the elements they contact. Organic compounds, such as lubricants and electrical insulation, will suffer fragmentation that results in the formation of different material, and integrated circuits in computer systems can be damaged. Metals such as steel are less affected by such irradiation (Ref. 5.15, p. C-9). Generally, the only type of radiation emanating from the high-level waste packages that may affect steel is the neutron field. The remainder, namely beta, gamma, and alpha radiation, have no known significant effect upon the structural properties of steel (Ref. 5.15, p. C-9).

Irradiation is described in terms of (1) the flux of neutrons striking the material, measured as the number of neutrons per square meter per second ($n/[m^2 \cdot s]$) and (2) the fluence, which is flux integrated over time or the number of neutrons per square meter (n/m^2). Based on a 100-year service life for the underground facilities, the neutron fluence for a waste package has been conservatively estimated to not exceed $2.2 \times 10^{20} \text{ n/m}^2$ (Ref. 5.15, p. C-9). This value is about $3.3 \times 10^{20} \text{ n/m}^2$ for a 150-year service life (Section 4.2.1), which includes retrievability preparation time and closure operations. Because the neutron fluence over the service life of the underground facilities is less than $2.2 \times 10^{20} \text{ n/m}^2$ (considering a 100-year service life), the expected increase in the ductile-brittle transition temperature is not more than 0.6 K (Ref. 5.15, p. C-10) or 0.9 K for a 150-year service life, which indicates the effect of neutron irradiation on steel is minimal.

In summary, the neutron radiation field expected from any single waste package is not expected to exceed $3.3 \times 10^{20} \text{ n/m}^2$ for 150 years based upon estimated values from bare fuel assemblies. This value is very conservative because in practice the radiation field would be expected to be several orders of magnitude lower due to shielding from the waste package walls, decay of radioactivity, and geometric divergence. Therefore, radiation effects are believed to be insignificant and not expected to degrade the steel properties (Ref. 5.15, p. C-10).

8. CONCLUSIONS

The following conclusions are made based on the evaluation of the materials for emplacement drift ground support. Due to the preliminary nature of this analysis, all input data, excluding codes and standards, shall be treated as unqualified/unconfirmed, and therefore, the outputs are also unqualified. The output/conclusions from this analysis cannot be used as input into documents supporting procurement, fabrication, or construction.

8.1 POSTCLOSURE EFFECTS OF MATERIALS

8.1.1 Concrete

The postclosure evolution of cementitious materials in the repository emplacement drifts has potential detrimental effects on waste isolation:

- 1) Concrete pore solutions with high pH have the potential to increase radionuclide solubility.
- 2) Dissolved constituents of concrete formed at high pH could reduce sorption of radionuclides by natural barrier components such as zeolites.
- 3) Water from dehydration reactions within the cement could be produced in the near-field environment and possibly increase the relative humidity within the tunnel.
- 4) The porosity and permeability of the host rock could be changed by increases in silica solubility.
- 5) The introduction of organic materials such as superplasticizers in concrete in the emplacement drifts may have impacts on performance through changes in the concentrations of organic acids and organic colloids which can increase waste package corrosion, radionuclide solubility and transport, and silicate mineral dissolution. In addition, organics may promote microbial activity which is another factor leading to waste package corrosion.

8.1.2 Steel

From the viewpoint of PA, there is little concern about the introduction of steel sets in the repository (see discussion in Sec. 7.2.3). A large number of waste packages, of which the outer barrier material is carbon steel, will be emplaced in the emplacement drifts. The carbon steel in the waste packages is very similar to the material for the steel sets. In the postclosure period, however, especially after the temperature cools off and relative humidity is increased, the potential for steel corrosion could probably be higher. Since there are many uncertainties and variables involved in the mechanism and prediction of corrosion during the postclosure period, it is beyond the scope of this study to investigate this issue.

8.2 CHARACTERISTICS OF CONCRETE

8.2.1 Strength

- For temperatures up to 200°C (392°F) (Sec. 4.3.4), which is the temperature limit in the repository emplacement drifts, the compressive strength of concrete is assumed to be reduced up to 30% compared with the ambient temperature strength.
- Most of the strength loss of concrete takes place within the first few months of testing, which indicates that long-term exposure of concrete within a waste emplacement drift does not result in sustained degradation during the design life, which suggests that a relatively short-term test program (e.g. less than 2 years) may provide sufficient basis to determine concrete behavior for the duration of the repository preclosure period.
- For the same water/cement (w/c) ratio, the higher the silica fume content, the higher will be the compressive strength. For the same content of silica fume, strength increases as w/c ratio decreases. The impact of cyclic varying temperature in unventilated emplacement drifts should be insignificant since the temperature increase is expected to be slow and uniform with the exception of a potential blast cooling event. Even though this result is conservative for repository conditions, repeated cycles of cooling and heating should be avoided.

8.2.2 Deformability

- For temperature of 200°C (or 392°F) (Sec. 4.3.4), which is the temperature limit in the repository emplacement drift, and prolonged periods of heating above 100°C, it is reasonably conservative to assume that the modulus of elasticity of concrete may be reduced about 50 percent compared with the ambient temperature modulus of elasticity.
- Long-term exposure to elevated temperatures produces losses in concrete modulus larger than those determined from relatively short-term tests. This difference is more noticeable at 450 °F, where short-term test results indicated moduli losses on the order of 35 percent, whereas long-term test results show moduli losses almost twice as large as this.
- Poisson's ratio shows no well defined relationship with respect to time and temperature. For concrete containing silica fume, Young's modulus does not continue to increase with increasing compressive strength. It was indicated in one study that for concrete containing 25% silica fume, the compressive strength increase is 43% whereas the Young's modulus increase is only 7% compared with that without silica fume.

8.2.3 Creep and Shrinkage

- The effects of creep may be harmful but, on the whole, creep, unlike shrinkage, is beneficial in relieving stress concentrations. It has contributed to the success of concrete as a structural member, which is very important for the structural design of the concrete

linings for the emplacement drift exposed to high temperatures for a prolonged period.

- In order to minimize the drying shrinkage, the water content of concrete needs to be kept as low as possible. This can be achieved by keeping the total coarse aggregate content of the concrete as high as possible, using low slumps, and placing methods that minimizing water requirements. Silica fume has no effect on basic creep, but significantly reduces the drying creep, or shrinkage. Steam curing will also reduce drying shrinkage.
- Empirical equations for predicting the creep strain under elevated temperature have been derived by PCA (see Equations 7-2 and 7-3 in Sec. 7.3.1.3). It may be reasonable to apply this equation in the structural analysis of concrete linings for emplacement drift stability when no test data are available for the repository site. However, it needs to be emphasized that creep tests of the proposed concrete mix should be conducted and further work to verify the applicability of this equation should also be done. If any change is found to be necessary, then the modified or newly developed equation for predicting the creep should be used.

8.2.4 Thermal Properties

- The coefficient of thermal expansion of concrete is nearly directly proportional to that of the contained aggregate, with the main controlling factor being the proportion of quartz present. Rocks with a high quartz content have the lowest coefficient. An average value of coefficient of thermal expansion of concrete is $9.9 \times 10^{-6}/^{\circ}\text{C}$. In general, the coefficient of thermal expansion tends to increase with temperature.
- The thermal conductivity of concrete is independent of temperature within the normal climatic range. In general, the thermal conductivity of normal weight concrete ranges from 1.5 to 3.5 W/m·K with an average of 2.5 W/m·K. Conductivity of concrete decreases with increasing temperature. For temperatures between 100 °C and 400 °C, thermal conductivity decreases linearly with temperature.
- The specific heat of concrete is strongly dependent on porosity (w/c ratio), water content, and temperature. Typical values of concretes with normal-weight aggregates have specific heats ranging from 840 to 1170 J/kg·K with average of 1005 J/kg·K.

8.2.5 Radiation and Biological Effects

- The radiation exposure that results in significant loss in concrete compressive strength is much larger than the reasonably predicted radiation exposures for concrete within the waste emplacement drifts during the retrieval period. Gamma and radiation field strengths for an integrated exposure for 150 years, assuming no decay, is about four orders of

magnitude below an approximate threshold value, above which measurable degradation of concrete occurs. Neutron radiation field strength is much less than that from gamma radiation.

- The combined radiation and heat will pose lethal challenges to all bacteria. The heat will dry the drifts, eliminating a water source for the bacteria. Even the most thermophilic bacteria cannot withstand 160°C temperatures. Temperatures exceeding 80°C will develop in the drifts within nominally 25 years after waste emplacement, preventing any bacterial activity for the remainder of the 150-year operation life.

8.2.6 Durability

- The single parameter that has the largest influence on durability is the water/cement (w/c) ratio. As the w/c ratio is decreased, the porosity of the paste is decreased and the concrete becomes more impermeable, which also increases the strength.
- The major effects of carbonation are: 1) increased shrinkage upon drying and 2) lower alkalinity of concrete. Although it is not desirable to have shrinkage by carbonation, it is beneficial to have a low pH value for reducing the radionuclide solubility, which is desirable from the long-term geochemical viewpoint during the postclosure period.
- Resistance to sulfate attack is one of the major concerns about the durability of concrete. Since sulfate resistance related to the mechanical properties of concrete is a long-term consideration, use of Type V portland cement appears to be more desirable. The resistance is enhanced for concrete with higher cement content and for cement low in C₃A content. Silica fume provides excellent sulfate resistance to concrete, better than fly ash or ground slag. Another way of reducing sulfate attack that can be used for precast concrete is high-pressure steam curing.
- Alkali-silica reaction can affect the performance of concrete. It is desirable to try to avoid using the susceptible aggregate. However, it is usually not a practical or an economical solution. If a reactive aggregate must be used, several measures can be taken to minimize the reaction. Most important is to use a low-alkali cement. A limit of 0.6 percent total alkali in the cement is commonly specified. Use of pozzolans, such as silica fume, fly ash, and ground granulated blast-furnace slag, also can significantly reduce the alkali-silica reaction.
- Abrasion resistance is closely related to the compressive strength of concrete. A low water-cement ratio and adequate curing are necessary for abrasion resistance. The type of aggregate and surface finish or treatment used also have a strong influence on abrasion resistance. The addition of steel fiber to the concrete will increase the ductility and the

energy absorption capacity to inhibit the cracking caused by the handling and erection of the concrete segments.

8.3 EVALUATION OF CONCRETE MATERIALS AND MIX DESIGN

Based on the evaluations of suitable components of concrete such as cement types, aggregates, pozzolanic materials, steel fibers, and water-reducing admixtures in Section 7.4, a preliminary concrete mix design is proposed in Section 7.5. This concrete mix design is developed based on the experience in ESF concrete invert construction, the strength and durability requirements for the preclosure period, and the performance considerations during the postclosure period.

Because the use of silica fume aids in reducing the potential pH of the concrete system, two concrete mix designs are presented to help in understanding how to achieve this goal. Option 1, containing a moderate amount of silica, is a concrete mix of commonly used proportions of materials with 10 to 20% of silica fume by weight. Option 2 contains a relatively high amount of silica, in the form of silica fume and silica flour, and requires more than normal amounts of superplasticizer.

In Option 1 (Moderate Silica), a preliminary proportion of concrete components is developed using 15% silica fume by weight of cement. The concrete mixture proportion is derived by following the procedures in ACI 211.1. Table 7.5 shows the preliminary composition of the concrete mixture for repository drifts based on the assumptions listed in Section 7.5.2.2. Since the assessment of potential postclosure performance impacts from cementitious materials has not been completed by the Performance Assessment group and the final components have not been determined, the concrete mix proportion presented here is preliminary in nature. It was calculated based on assumed conditions. Some assumptions may change depending on test results, local conditions, special specifications or requirements. Consequently, the composition of concrete and the type of ingredients may also change. In addition, methods for proportioning and evaluating concrete mixtures containing those supplementary admixtures, such as silica fume, water-reducing admixture (e.g., Polyheed), and superplasticizer (e.g., Rheobuild), must be based on trial mixtures using a range of ingredient proportions. By evaluating their effects on strength, water requirement, time of set, and other important properties, the optimum amount of admixtures can be determined. It is also important to note that it is necessary to use a dispensing system in adding the steel fibers to the concrete mix to have random but uniform distribution of steel fibers in the concrete.

It should be mentioned that although a target pH value of about 10 is desirable from the postclosure performance viewpoint, this value is a preliminary recommendation. Although the starting pH value of the cement paste solution may not reach such a low value, the long-term pH value during the postclosure period may drop below 10 due to the potential impacts such as microbial activity, the oxidation of carbon steel from the waste package, radiation effect on water

chemistry, and the carbonation effect. Even though there are some uncertainties about these impacts, it appears reasonable to take them into consideration. Further investigation on how these potential impacts affect the pH value of the cement paste solution in the postclosure period seems to be necessary to confirm this viewpoint.

In Option 2 (High Silica), a special design of concrete mix with less cement content, a larger amount of silica fume and silica flour, and a higher dosage of superplasticizer is proposed, which may yield a starting pH value below 10. The potential problem with this mix design is that it uses a relatively large amount of superplasticizer (i.e., organic material), which may not be acceptable due to waste isolation concerns. In this mix design, a detailed proportioning of concrete components (i.e., quantities of each component) is not presented due to many uncertainties involved in the evolution of concrete during the postclosure period. Rather, a conceptual proposal is presented in comparison with a mix design developed by AECL-WL (Canada).

Finally, an estimate of quantities of concrete components per linear meter of concrete lining for Option 1 mix design is calculated in Section 7.5.2.3 based on the cross-sectional view of a typical emplacement drift in the current repository design. Table 7.7 shows the estimate for a drift diameter of 5.5 meters and the concrete lining of 200 mm thickness.

8.4 CHARACTERISTICS OF STEEL

8.4.1 Strength and Modulus of Elasticity

- Up to about 370 °C, the yield and ultimate strength of carbon steels can be used essentially as they would for design of components at room temperature. Creep is not observed in these steels until temperatures are above 370 °C. The modulus of elasticity of structural steel decreases from an initial value of 200 GPa (29,000 ksi) at about 20 °C to about 172 GPa (25,000 ksi) at 480 °C, or 189 GPa (27,400 ksi) at 200 °C (Sec. 4.3.4), which is about a 5% decrease in comparison to that at 20 °C.
- For the repository emplacement drift environment, which has a limiting temperature of 200 °C (Sec. 4.3.4), carbon steels may experience modest, but insignificant, decreases in strength (up to about 9 percent) and modulus of elasticity (up to about 5 percent) in comparison to these same parameters at 20 °C.

8.4.2 Thermal Properties

- Structural steels have a coefficient of thermal expansion that varies from about $11.5 \times 10^{-6}/^{\circ}\text{C}$ at 20 °C to $13.8 \times 10^{-6}/^{\circ}\text{C}$ at 200 °C (Sec. 4.3.4). The thermal expansion coefficient for tuff for near-field considerations is shown to vary from about $5.07 \times 10^{-6}/^{\circ}\text{C}$ at 25 °C to $8.97 \times 10^{-6}/^{\circ}\text{C}$ at 200 °C (Sec. 4.3.4). These data show differences in expansion

coefficients between tuff (or concrete with a tuff aggregate) and steel decrease from about $6.5 \times 10^{-6}/^{\circ}\text{C}$ at 25°C , to about $5 \times 10^{-6}/^{\circ}\text{C}$ at 200°C (Sec. 4.3.4). Because differences in expansion coefficients are relatively small and the rate of repository heating is expected to be slow, the effects of differential expansion are anticipated to be minor. For steel-fiber reinforced concrete, the effect due to different thermal expansion is insignificant because steel fibers are discretely and uniformly distributed in the concrete.

- The average thermal conductivity of the carbon steel for temperature 0 to 200°C (Sec. 4.3.4) is $50.67 \text{ W/m}\cdot\text{K}$ ranging from 51.9 to $49.0 \text{ W/m}\cdot\text{K}$, with higher values for lower temperatures. The average specific heat of the carbon steel for temperature 50 (Sec. 4.3.3) to 200°C (Sec. 4.3.4) is $502.5 \text{ J/kg}\cdot\text{K}$ ranging from 486 to $519 \text{ J/kg}\cdot\text{K}$ with higher values for higher temperatures.

8.4.3 Radiation Effect

- Generally, the only type of radiation emanating from the high-level waste packages that may affect steel is the neutron field. The neutron radiation field expected from any single waste package is not expected to exceed $3.3 \times 10^{20} \text{ n/m}^2$ for 150 years based upon estimated values from bare fuel assemblies. The expected increase in the ductile-brittle transition temperature is not more than 0.9 K for a 150-year service life, which indicates the effect of neutron radiation on steel is minimal. It should also be noted that $3.3 \times 10^{20} \text{ n/m}^2$ is very conservative because in practice the radiation field would be expected to be several orders of magnitude lower due to shielding from the waste package walls, decay of radioactivity, and geometric divergence. Therefore, radiation effects are believed to be insignificant and not expected to degrade the steel properties.

8.4.4 Durability

- Steel sets are susceptible to corrosion under certain conditions. Since the repository emplacement drift environment is not conducive to corrosion due to its high temperature, minimum presence of water, and very low amount of chloride, the effect of corrosion on the steel strength is very insignificant during the service life of 150 years.
- Steel biodegradation occurs when sulfate reducing bacteria (SRB) consumes hydrogen during sulfate reduction. The presence of oxygen, excessively high temperatures, and inadequate sources of sulfate and carbon create an environment hostile to SRB. In the absence of the specialized environment required for SRB metabolism, corrosion of steel by SRB is not expected to occur.
- As discussed in the section for concrete, the combined radiation and heat will pose lethal challenges to all bacteria. Elevated temperatures in the emplacement drifts will ultimately

eliminate liquid water, eliminating a water source for the bacteria. Even the most thermophilic bacteria cannot withstand 160°C temperatures. Temperatures exceeding 80°C will develop in the drifts within nominally 25 years after waste emplacement, preventing any bacterial activity for the remainder of the 150-year service life.

9. ATTACHMENTS

ATTACHMENTS	DESCRIPTION
I	Step-by-Step Calculation of Concrete Mix Design - Option 1
II	Manufacturer Information: A. Steel Fiber - Dramix B. Water-Reducing Admixture - Polyheed C. Superplasticizer - Rheobuild 1000

Attachment I : Step-by-step Calculation of Concrete Mix Design - Option 1

In this section, only the proportioning procedure based on ACI Standard Practice ACI 211.1 will be described. The selection of adequate materials is already discussed in Section 7.5.2.1.

Step 1: Choice of Slump

The desired slump is 3 to 4 in. (7.6 to 10.2 cm)

Step 2: Coarse Aggregate Size

The tuff aggregate from the TBM muck meeting ASTM C 33 will be used. The maximum size will be 3/4 in. (19 mm). Basically, for high compressive strength concrete (6000 psi plus) (41.4 MPa plus) with a cement content exceeding 600 lb/cu yd (356 kg/m³), the optimum maximum size is about 3/4 in. (19 mm) (Ref. 5.6). For steel fiber reinforced concrete, maximum aggregate size in excess of 3/4 in. (19 mm) are not suitable (ACI 544.3R-88).

Step 3: Water-Cement Ratio (w/c)

Target water-cement ratio of 0.4 is selected based on two factors: 1) In Table 6.3.4(a) of ACI 211.1, w/c ratio is 0.48 for f'_c of 5000 psi (34.5 MPa), 2) For the durability of the tunnel lining, w/c of less than 0.4 is indicated. It is reasonable to assume 0.4 for the current study.

Step 4: Water Content

From Table 6.3.3 of ACI 211.1, the required water content for a slump of 3 to 4 in. (7.6 to 10.2 cm) and maximum aggregate size of 3/4 in. (19 mm) is 340 lb/cu yd (201 kg/m³). However, since water-reducing admixture is to be used, a 5% reduction of water required is assumed, so the required water content is $340 (0.95) = 323$ lb/cu yd (192 kg/m³).

Step 5: Cement and Silica Fume

Since silica fume, a highly pozzolanic material, will be added to the cement, the w/c ratio needs to be replaced with $w/(c+p)$, where p stands for pozzolanic material, and in this case, it is silica fume. Based on page 211.1-11 of ACI 211.1, the equation for converting a target water-cement ratio w/c to a weight ratio of water to cement plus pozzolanic materials $w/(c+p)$ by the absolute volume method is as follows:

$$\begin{aligned} &w/(c+p) \text{ weight ratio,} \\ &\text{absolute volume equivalency} = 3.15(w/c)/[3.15(1-F_v) + G_p(F_v)] \end{aligned} \quad (1)$$

Where G_p is the sp. gravity of pozzolan and F_v is the pozzolan percentage by absolute volume of the total volume of cement plus pozzolan. If only the desired pozzolan percentage by weight F_w is known, it can be converted to F_v as follows:

$$F_v = 1/[1 + (G_p/3.15) (1/F_w - 1)] \quad (2)$$

For $p/c = 0.15$, $F_w = p/(c+p) = 0.15/(1+0.15) = 0.13$, so substituting $F_w = 0.13$ into equation (2), $F_v = 0.176$. Then substituting $F_v = 0.176$ into equation (1), $w/(c+p)$ is calculated to be 0.422, i.e., the target weight ratio to maintain an absolute volume equivalency is $w/(c+p) = 0.422$.

For mixing water of 323 lb/cu yd (192 kg/m³) (see Step 4), the required weight of cementitious material, i.e., cement plus silica fume, is $323/0.422 = 765$ lb/cu yd (454 kg/m³). The weight of silica fume is $765 (0.13) = 99$ lb/cu yd (59 kg/m³) and the weight of cement is $765 - 99 = 666$ lb/cu yd (395 kg/m³).

Step 6: Coarse Aggregate Content

From Table 6.3.6 of ACI 211.1, the bulk volume of coarse aggregate with maximum size of 3/4 in. (19 mm) recommended when using a fine aggregate with fineness modulus of 2.80 is 0.62. Since it weighs 100 lb/cu ft, the oven-dry weight of coarse aggregate for a cubic yard of concrete is $100(27)(0.62) = 1674$ lb/cu yd (993 kg/m³).

Step 7: Fine Aggregate Content

The volume of fine aggregate is determined by subtracting the absolute volume of the known ingredients from 27 cu ft (i.e., 1 cu yd) (0.76 m³). In general, the liquid admixture volume is insignificant and is not included in the calculation. The volume of each ingredient is calculated as follows:

$$\begin{aligned} \text{Water} &= 323/62.4 = 5.18 \text{ cu ft (0.15 m}^3\text{)} \\ \text{Coarse aggregate} &= 1674/(2.68 \times 62.4) = 10.01 \text{ cu ft (0.28 m}^3\text{)} \\ \text{Silica fume} &= 99/(2.2 \times 62.4) = 0.72 \text{ cu ft (0.02 m}^3\text{)} \\ \text{Cement} &= 666/(3.15 \times 62.4) = 3.39 \text{ cu ft. (0.10 m}^3\text{)} \end{aligned}$$

The volume of fine aggregate is $27 - 5.18 - 10.01 - 0.72 - 3.39 = 7.70$ cu ft (0.22 m³). The weight of fine aggregate is $7.70 \times 2.64 \times 62.4 = 1268$ lb/cu yd (752 kg/m³).

Step 8: Moisture Corrections

With the aggregate moisture content indicated in the assumptions (see Sec. 7.5.2.2), the trial batch aggregate proportions become:

Coarse aggregate (1%) = $1674 \times 1.01 = 1691 \text{ lb/cu yd}$ (1003 kg/m^3)
Fine aggregate (3%) = $1268 \times 1.03 = 1306 \text{ lb/cu yd}$ (775 kg/m^3).

Step 9: Water Content Correction

Absorbed water does not become part of the mixing water and needs to be excluded. So, surface moisture contributed by the coarse aggregate amounts to $1\% - 0.5\% = 0.5\%$, or $1674 \times 0.005 = 8 \text{ lb}$ (3.6 kg), that contributed by fine aggregate amounts to $3\% - 0.7\% = 2.3\%$, or $1268 \times 0.023 = 29 \text{ lb}$ (13.2 kg). The water contributed by the water-reducing admixture (WRA) and superplasticizer also need to be adjusted.

WRA: $(10 \times 765/100)/16 = 4.8$, say 5 lb/cu yd (3 kg/m^3)
Superplasticizer: $(25 \times 765/100)/16 = 12.0 \text{ lb/cu yd}$ (7 kg/m^3).
(1 lb = 16 oz)

Thus, the total water that needs to be added is $323 - 8 - 29 - 5 - 12 = 269 \text{ lb/cu yd}$ (160 kg/m^3)

Step 10: Proportion of Concrete Mixture

From the above calculations, the preliminary proportions of the concrete mixture based on the listed assumptions are:

Cement	666 (rounded to 670*)
Silica fume	99 (rounded to 100*)
Coarse aggregate (1%)	1691 (rounded to 1690*)
Fine aggregate (3%)	1306 (rounded to 1310*)
Water-reducing admixture	5
Superplasticizer	12
<u>Water (to be added)</u>	<u>269 (rounded to 270*)</u>
Subtotal	4048 lb/cu yd (2402 kg/m^3)

Note: the rounded-off values are shown in Table 7.5.

The steel fiber to be added to the concrete mix is assumed to be 0.5% of concrete by volume, i.e., its weight is $(7.850)(62.4)(27)(0.005) = 66.1$, say, 66 lb/cu yd (39 kg/m^3).

Title: Materials for Emplacement Drift Ground Support

Attachment II

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Page: II-1 of II-12

Attachment II: Manufacturer Information

A. Steel Fiber - Dramix