Appendix B
Commentary on General NPH Design and Evaluation Criteria

B.1 NPH Design and Evaluation Philosophy

The natural phenomena hazard (NPH) design and evaluation criteria presented in this document (DOE-STD-1020) implement the requirements of DOE Order 420.1, "Facility Safety" (Ref. B-1) and the associated Implementation Guides: "Implementation Guide for the Mitigation of Natural Phenomena Hazards for DOE Nuclear Facilities and Non-nuclear Facilities" (Ref. B-2), "Implementation Guide for Nonreactor Nuclear Safety Design Criteria and Explosives Safety Criteria" (Ref. B-3), and "Implementation Guide for Use with DOE Orders 420.1 and 440.1 Fire Safety Program" (Ref. B-4) which are intended to assure acceptable performance of DOE facilities in the event of earthquake, wind/tornado, and flood hazards. As discussed in Chapter 1, performance is measured by target performance goals expressed as an annual probability of exceedance of acceptable behavior limits (i.e., behavior limits beyond which damage/failure is unacceptable). DOE Order 420.1 and the associated Implementation Guides establish a graded approach for NPH requirements by defining performance categories (numbered 0 through 4) each with a qualitative performance goal for behavior (i.e., maintain structural integrity, maintain ability to function, maintain confinement of hazardous materials) and a qualitative target probabilistic performance goal. DOE-STD-1020 provides four sets of NPH design and evaluation criteria (explicit criteria are not needed for Performance Category 0). These criteria range from those provided by model building codes for Performance Category 1 to those approaching nuclear power plant criteria for Performance Category 4.

DOE-STD-1020 employs the graded approach by following the philosophy of probabilistic performance goal-based design and evaluation criteria for natural phenomena hazards. Target performance goals range from low probability of NPH-induced damage/failure to very high confidence of extremely low probability of NPH-induced damage/failure. In this manner, structures, systems, and components (SSCs) are governed by NPH criteria which are appropriate for the potential impact on safety, mission, and cost of those SSCs. For example, a much higher likelihood of damage would be acceptable for an unoccupied storage building of low value than for a high-occupancy facility or a facility containing hazardous materials. SSCs containing hazardous materials which, in the event of damage, threaten public safety or the environment, and/or which have been determined to require special consideration, should have a very low probability of damage due to natural phenomena hazards (i.e., much lower probability of damage than would exist from the use of model building code design and evaluation procedures). For ordinary SSCs of relatively low

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cost, there is typically no need or requirement to add conservatism to the design beyond that of model building codes. For these SSCs, it is also typically not cost-effective to strengthen structures more than required by model building codes that consider extreme loads due to natural phenomena hazards.

Performance goals correspond to probabilities of structure or equipment damage due to natural phenomena hazards; they do not extend to consequences beyond structure or equipment damage. The annual probability of exceedance of SSC damage as a result of natural phenomena hazards (i.e., performance goal) is a combined function of the annual probability of exceedance of the event, factors of safety introduced by the design/evaluation procedures, and other sources of conservatism. These criteria specify hazard annual probabilities of exceedance, response evaluation methods, and permissible behavior criteria for each natural phenomena hazard and for each performance category such that desired performance goals are achieved for either design or evaluation. The ratio of the hazard annual probability of exceedance and the performance goal annual probability of exceedance is called the risk reduction ratio, R_R in DOE-STD-1020. This ratio establishes the level of conservatism to be employed in the design or evaluation process. For example, if the performance goal and hazard annual probabilities are the same (R_R = 1), the design or evaluation approach should introduce no conservatism. However, if conservative design or evaluation approaches are employed, the hazard annual probability of exceedance can be larger (i.e., more frequent) than the performance goal annual probability ($R_R > 1$). In the criteria presented herein, the hazard probability and the conservatism in the design/evaluation method are not the same for earthquake, wind, and flood hazards. However, the accumulated effect of each step in the design/evaluation process is to aim at the performance goal probability values which are applicable to each natural phenomena hazard separately.

Design and evaluation criteria are presented in Chapters 2, 3, and 4 for earthquake, wind, and flood hazards, respectively. These criteria are deterministic procedures that establish SSC loadings from probabilistic natural phenomena hazard curves; specify acceptable methods for evaluating SSC response to these loadings; provide acceptance criteria to judge whether computed SSC response is acceptable; and to provide detailing requirements such that behavior is as expected as illustrated in Figure B-1. These criteria are intended to apply equally for design of new facilities and for evaluation of existing facilities. In addition, the criteria are intended to cover buildings, equipment, distribution systems (piping, HVAC, electrical raceways, etc.), and other structures.

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DOE-STD-1020 primarily covers (1) methods of establishing load levels on SSCs from natural phenomena hazards and (2) methods of evaluating the behavior of structures and equipment to these load levels. These items are very important, and they are, typically, emphasized in design and evaluation criteria. However, there are other aspects of facility design that are equally important and that should be considered. These aspects include quality assurance considerations and attention to design details. Quality assurance requires peer review of design drawings and calculations; inspection of construction; and testing of material strengths, weld quality, etc. The peer reviewers should be qualified personnel who were not involved in the original design. Important design details include measures to assure ductile behavior and to provide redundant load paths, as well as proper anchorage of equipment and nonstructural building features. Although quality assurance and design details are not discussed in this report to the same extent as NPH load levels and NPH response evaluation and acceptance criteria, the importance of these parts of the design/evaluation process should not be underestimated. Quality assurance and peer review are briefly addressed in Section 1.4, in addition to discussions in the individual chapters on each natural phenomena hazard. Design detailing for earthquake and wind hazards is covered by separate manuals. Reference B-5 describes earthquake design considerations including detailing for ductility. Reference B-6 gives structural details for wind design.

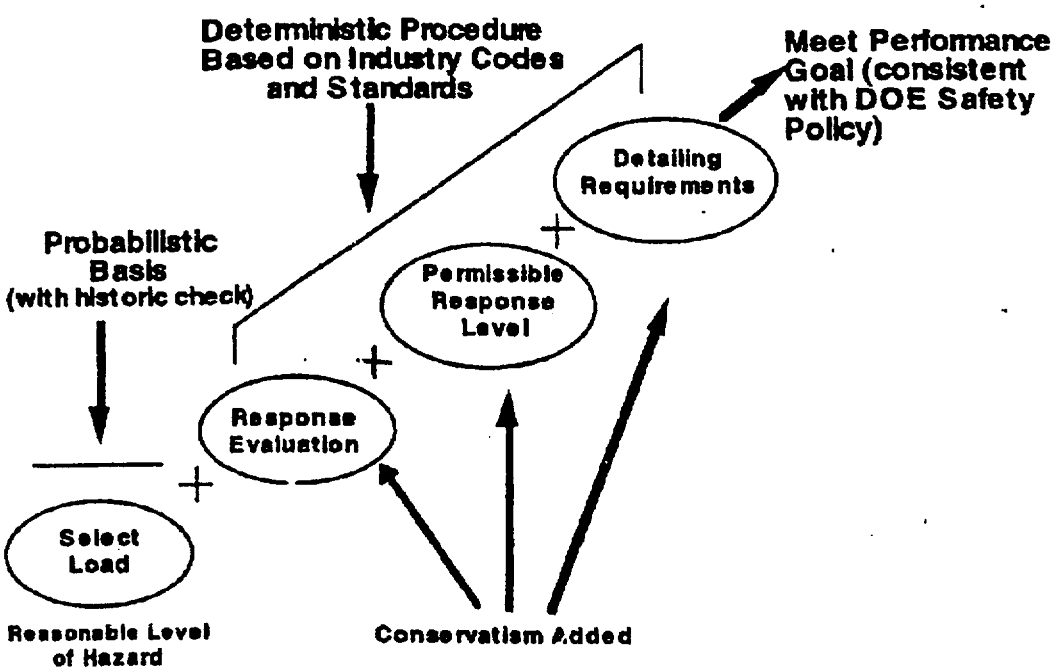

Figure B-1 DOE-STD-1020 Combines Various Methods to Achieve Performance Goals

B.2 Graded Approach, Performance Goals, and Performance Categories

As stated above, DOE Order 420.1 and the associated Implementation Guides establish a graded approach in which NPH requirements are provided for various performance categories each with a specified performance goal. The motivation for the graded approach is that it enables design or evaluation of DOE structures, systems, and components to be performed in a manner consistent with their importance to safety, importance to mission, and cost. There are only a few "reactor" facilities in the DOE complex and many facilities with a wide variety of risk potential, mission, and cost. Also, the graded approach enables cost-benefit studies and establishment of priorities for existing facilities. There are few new designs planned for the DOE complex and the evaluation of existing facilities requires cost benefit considerations and prioritizing upgrading and retrofit efforts. Finally, the graded approach is common practice by model building codes such as the Uniform Building Code (Ref. B-7), Department of Defense earthquake provisions (Ref. B-8), and even by the Nuclear Regulatory Commission which provides graded criteria from power plants to other licensed nuclear facilities.

The motivation for the use of probabilistic performance goals by the NPH Implementation Guide for DOE Order 420.1 and DOE-STD-1020 is that accomplish the graded approach using a quantified approach consistent with the variety of DOE facilities as well as meeting the risk-based DOE safety policy. Furthermore, the use of probabilistic performance goals enables the development of consistent criteria both for all natural phenomena hazards (i.e., earthquakes, winds, and floods) and for all DOE facilities which are located throughout the United States. The use of performance goal based criteria is becoming common practice as: it is embedded in recent versions of the Uniform Building Code and in the DOD seismic provisions for essential buildings; it has been used for DOE new production reactor NPH criteria; and it has been utilized in recent Nuclear Regulatory Commission applications such as for the advanced light water reactor program and for revisions to commercial reactor geological siting criteria in 10CFR100, Appendix A.

Five performance categories are specified in the Implementation Guide for DOE Order 420.1 for design/evaluation of DOE structures, systems, and components for natural phenomena hazards ranging from 0 through 4. Table B-1 presents both the qualitative and quantitative descriptions of the performance goals for each performance category. Both the qualitative description of acceptable NPH performance and the quantitative probability value for each performance category are equally significant in establishing these NPH

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design and evaluation criteria within a graded approach. SSCs are to be placed in categories in accordance with DOE-STD-1021-93 (Ref. B-9) Additional guidance on performance categorization is available in Reference B-10.

As mentioned previously, the quantitative performance goal probability values are applicable to each natural phenomena hazard (earthquake, wind, and flood) individually. The earthquake and flood design and evaluation criteria presented in this document are aimed at meeting the target performance goals given in Table B-1. The extreme wind and tornado design and evaluation criteria presented in this document are conservative compared to earthquake and flood criteria in that they are aimed at lower probability levels than the target performance goals in Table B-1. It is estimated that for extreme winds, the probabilities of exceeding acceptable behavior limits are less than one order of magnitude smaller than the performance goals in Table B-1. For tornado criteria, the probabilities of exceeding acceptable behavior limits are greater than one but less than two orders of magnitude smaller than the performance goals for Performance Categories 3 and 4. This additional conservatism in wind and tornado criteria for design and evaluation of DOE facilities is consistent with common practice in government and private industry. Furthermore, this additional conservatism can be accommodated in the design and evaluation of SSCs without significantly increasing costs. SSCs in Performance Categories 3 and 4 should be designed for tornadoes at certain sites around the country where tornado occurrences are high. The tornado hazard probability must be set lower than necessary to meet the performance goals in order for tornadoes rather than straight w. ᵗˢ or hurricanes to control the design criteria.

Table B-1 Structure, System, or Component (SSC) NPH Performance Goals for Various Performance Categories

Performance Category	Performance Goal Description	NPH Performance Goal Annual Probability of Exceeding Acceptable Behavior Limits, P_F
0	No Safety, Mission, or Cost Considerations	No requirements
1	Maintain Occupant Safety	$\approx 10^{-3}$ of the onset of SSC(1) damage to the extent that occupants are endangered
2	Occupant Safety, Continued Operation with Minimum Interruption	$\approx 5 \times 10^{-4}$ of SSC damage to the extent that the component cannot perform its function
3	Occupant Safety, Continued Operation, Hazard Confinement	$\approx 10^{-4}$ of SSC damage to the extent that the component cannot perform its function
4	Occupant Safety, Continued Operation, Confidence of Hazard Confinement	$\approx 10^{-5}$ of SSC damage to the extent that the component cannot perform its function

(1) These performance goals are for each natural phenomena hazard (earthquake, wind, and flood).
(2) SSC refers to structure, distribution system, or component (equipment).

The design and evaluation criteria for SSCs in Performance Categories 0, 1, and 2 are similar to those given in model building codes. Performance Category 0 recognizes that for certain lightweight equipment items, furniture, etc., and for other special circumstances where there is little or no potential impact on safety, mission, or cost, design or evaluation for natural phenomena hazards may not be needed. Assignment of an SSC to Performance Category 0 is intended to be consistent with, and not take exception to, model building code NPH provisions. Performance Category 1 criteria include no extra conservatism against natural phenomena hazards beyond that in model building codes that include earthquake, wind, and flood considerations. Performance Category 2 criteria are intended to maintain the capacity to function and to keep the SSC operational in the event of natural phenomena hazards. Model building codes would treat hospitals, fire and police stations, and other emergency-handling facilities in a similar manner to DOE-STD-1020 Performance Category 2 NPH design and evaluation criteria.

Performance Category 3 and 4 SSCs handle significant amounts of hazardous materials or have significant programmatic impact. Damage to these SSCs could potentially endanger worker and public safety and the environment or interrupt a significant mission. As a result, it is very important for these SSCs to continue to function in the event of natural phenomena hazards, such that the hazardous materials may be controlled and confined. For these categories, there must be a very small likelihood of damage due to natural phenomena hazards. DOE-STD-1020 NPH criteria for Performance Category 3 and higher SSCs are more conservative than requirements found in model building codes and are similar to DOD criteria for high risk buildings and NRC criteria for various applications as illustrated in Table B-2. Table B-2 illustrates how DOE-STD-1020 criteria for the performance categories defined in DOE Order 420.1 and the associated Implementation Guides compare with NPH criteria from other sources.

Table B-2 Comparison of Performance Categories from Various Sources

Source	SSC Categorization			
DOE-STD-1020 - DOE Natural Phenomena Hazard Criteria	1	2	3	4
Uniform Building Code	General Facilities	Essential Facilities	-	-
DOD Tri-Service Manual for Seismic Design of Essential Buildings	-	-	High Risk	-
Nuclear Regulatory Commission	-		Evaluation of NRC Fuel Facilities	Evaluation of Existing Reactors

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The design and evaluation criteria presented in this document for SSCs subjected to natural phenomena hazards have been specified to meet the performance goals presented in Table B-1. The basis for selecting these performance goals and the associated annual probabilities of exceedance are described briefly in the remainder of this section.

For *Performance Category 1 SSCs*, the primary concern is preventing major structural damage or collapse that would endanger personnel. A performance goal annual probability of exceedance of about 10^{-3} of the onset of significant damage is appropriate for this category. This performance is considered to be consistent with model building codes (Refs. B-7, B-11, B-12, and B-13), at least for earthquake and wind considerations. The primary concern of model building codes is preventing major structural failure and maintaining life safety under major or severe earthquakes or winds. Repair or replacement of the SSC or the ability of the SSC to continue to function after the occurrence of the hazard is not considered.

Performance Category 2 SSCs are of greater importance due to mission-dependent considerations. In addition, these SSCs may pose a greater danger to on-site personnel than Performance Category 1 SSCs because of operations or materials involved. The performance goal is to maintain both capacity to function and occupant safety. Performance Category 2 SSCs should allow relatively minor structural damage in the event of natural phenomena hazards. This is damage that results in minimal interruption to operations and that can be easily and readily repaired following the event. A reasonable performance goal is judged to be an annual probability of exceedance of between 10^{-3} and 10^{-4} of structure or equipment damage, with the SSC being able to function with minimal interruption. This performance goal is slightly more severe than that corresponding to the design criteria for essential facilities (e.g., hospitals, fire and police stations, centers for emergency operations) in accordance with model building codes (e.g., Ref. B-7).

Performance Category 3 and higher SSCs pose a potential hazard to public safety and the environment because radioactive or toxic materials are present. Design considerations for these categories are to limit SSC damage so that hazardous materials can be controlled and confined, occupants are protected, and functioning of the SSC is not interrupted. The performance goal for Performance Category 3 and higher SSCs is to limit damage such that DOE safety policy is achieved. For these categories, damage must typically be limited in confinement barriers (e.g., buildings, glove boxes, storage cannisters, vaults), ventilation systems and filtering, and monitoring and control equipment in the event of an occurrence of severe earthquakes, winds, or floods. In addition, SSCs can be placed in Performance Categories 3 or 4 if improved performance is needed due to cost or mission requirements.

For Performance Category 3 SSCs, an appropriate performance goal has been set at an annual probability of exceedance of about 10^{-4} of damage beyond which hazardous material confinement and safety-related functions are impaired. For Performance Category 4 SSCs, a reasonable performance goal is an annual probability of exceedance of about 10^{-5} of damage beyond which hazardous material confinement and safety-related functions are impaired. These performance goals approaches and approximates, respectively, at least for earthquake considerations, the performance goal for seismic-induced core damage associated with design of commercial nuclear power plants (Refs. B-14, B-15, B-16, and B-17). Annual frequencies of seismic core damage from published probabilistic risk assessments (PRA) of recent commercial nuclear plants have been summarized in Reference B-18. This report indicates that mean seismic core damage frequencies ranged from 4×10^{-6}/year to 1×10^{-4}/year based on consideration of 12 plants. For 10 of the 12 plants, the annual seismic core damage frequency was greater than 1×10^{-5}. Hence, the Performance Category 4 performance goals given in the NPH Implementation Guide for DOE Order 420.1 are consistent with Reference B-18 information.

B.3 Evaluation of Existing Facilities

New SSCs can be designed by these criteria, but existing SSCs may not meet these NPH provisions. For example, most facilities built a number of years ago in the eastern United States were designed without consideration of potential earthquake hazard. It is, therefore, likely that some older DOE facilities do not meet the earthquake criteria presented in this document.

For existing SSCs, an assessment must be made for the as-is condition. This assessment includes reviewing drawings and conducting site visits to determine deviations from the drawings and any in-service deterioration. In-place strength of the materials can be used when available. Corrosive action and other aging processes should be considered. Evaluation of existing SSCs is similar to evaluations performed of new designs except that a single as-is configuration is evaluated instead of several configurations in an iterative manner, as required in the design process. Evaluations should be conducted in order of priority, with highest priority given to those areas identified as weak links by preliminary investigations and to areas that are most important to personnel safety and operations with hazardous materials. Prioritization criteria for evaluation and upgrade of existing DOE facilities are currently being developed.

If an existing SSC does not meet the natural phenomena hazard design/evaluation criteria, several options (such as those illustrated by the flow diagram in Figure B-2) need to be considered. Potential options for existing SSCs include:

1. Conduct a more rigorous evaluation of SSC behavior to reduce conservatism which may have been introduced by simple techniques used for initial SSC evaluation. Alternatively, a probabilistic assessment of the SSC might be undertaken in order to demonstrate that the performance goals for the SSC can be met.
2. The SSC may be strengthened to provide resistance to natural phenomena hazard effects that meets the NPH criteria.
3. The usage of the SSC may be changed so that it falls within a lower performance category and consequently, less stringent requirements.

If SSC evaluation uncovers deficiencies or weaknesses that can be easily remedied, these should be upgraded without considering the other options. It is often more cost-effective to implement simple SSC upgrades than to expend effort on further analytical studies. Note that the actions in Table B-2 need not necessarily be accomplished in the order shown.

Evaluations of existing SSCs must follow or, at least, be measured against the NPH criteria provided in this document. For SSCs not meeting these criteria and which cannot be easily remedied, budgets and schedule for required strengthening must be established on a prioritized basis. As mentioned previously, prioritization criteria for evaluation and upgrade of existing DOE facilities are currently being developed. Priorities should be established on the basis of performance category, cost of strengthening, and margin between as-is SSC capacity and the capacity required by the criteria. For SSCs which are close to meeting criteria, it is probably not cost effective to strengthen the SSC in order to obtain a small reduction in risk. As a result, some relief in the criteria is allowed for evaluation of existing SSCs. It is permissible to perform such evaluations using natural phenomena hazard exceedance probability of twice the value specified for new design. For example, if the natural phenomena hazard annual probability of exceedance for the SSC under consideration was 10^{-4}, it would be acceptable to reconsider the SSC at hazard annual probability of exceedance of $2x10^{-4}$. This would have the effect of slightly reducing the seismic, wind, and flood loads in the SSC evaluation. This amount of relief is within the tolerance of meeting the target performance goals and is only a minor adjustment of the corresponding NPH design and evaluation criteria.

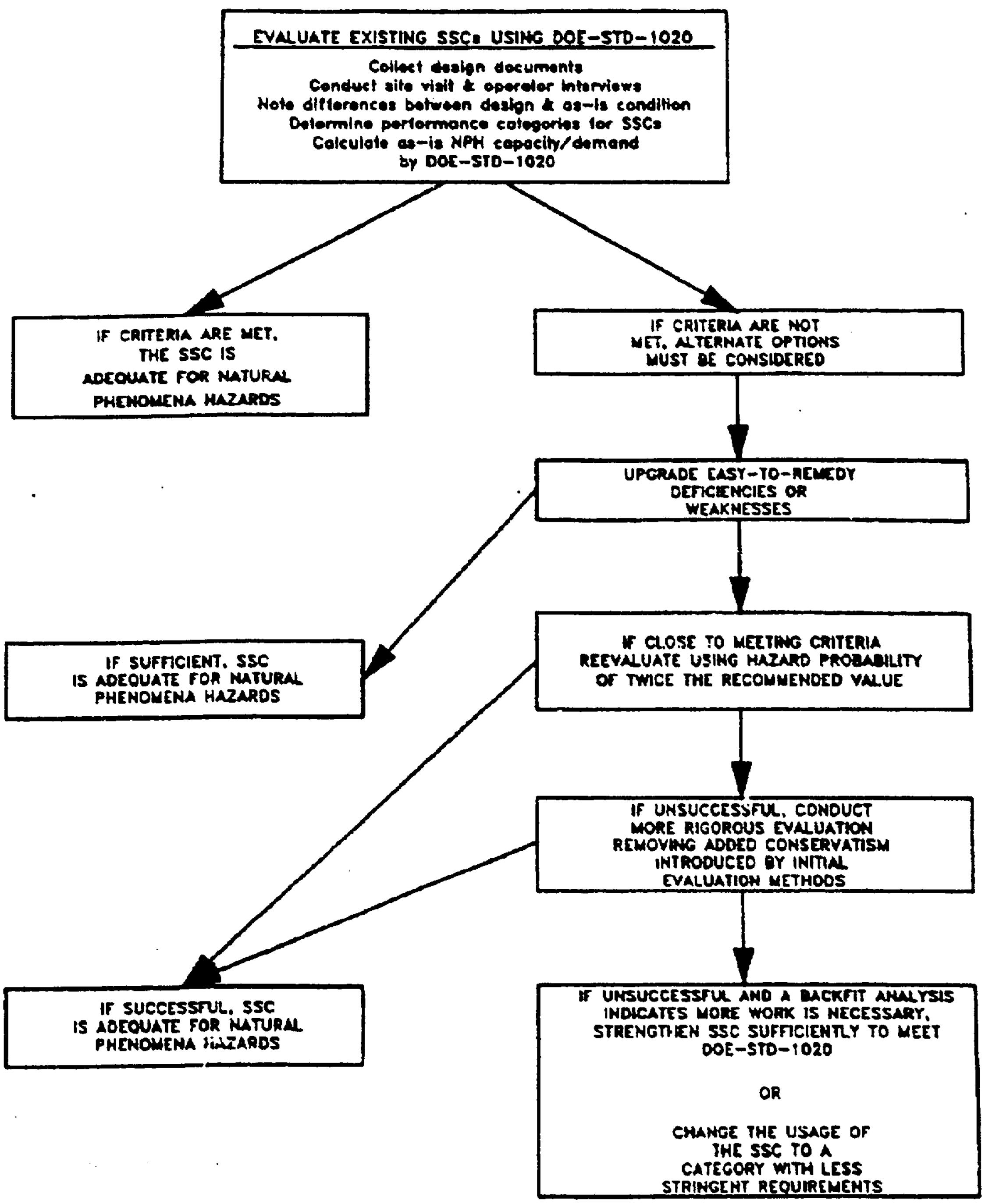

Figure B-2 Evaluation Approach for an Existing SSC

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B.4 References

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B-15. *Zion Probabilistic Safety Study*, Commonwealth Edison Company, Chicago, Illinois, 1981.

B-16. *Indian Point Probabilistic Safety Study*, Power Authority of the State of New York, 1982.

B-17. *Severe Accident Risk Assessment, Limerick Generating Station*, Philadelphia Electric Company, April 1983.

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Appendix C
Commentary on Earthquake Design and Evaluation Criteria

C.1 Introduction

Earthquake design and evaluation criteria for DOE structures, systems, and components are presented in Chapter 2 of this standard. Commentary on the DOE earthquake design and evaluation provisions is given in this appendix. Specifically, the basic approach employed is discussed in Section C.2 along with meeting of target performance goals, seismic loading is addressed in Section C.3, evaluation of seismic response is discussed in Section C.4, capacities and good seismic design practice are discussed in Section C.5, special considerations for systems and components and for existing facilities are covered in Sections C.6 and C.7, respectively, and quality assurance and peer review are addressed in Section C.8. Alternate seismic mitigation measures are discussed in Section C.9.

These seismic criteria use the target performance goals of the NPH Implementation Guide for DOE Order 420.1 (Ref. C-67) to assure safe and reliable performance of DOE facilities during future potential earthquakes. Design of structures, systems, and components to withstand earthquake ground motion without significant damage or loss of function depends on the following considerations:

1. The SSC must have sufficient strength and stiffness to resist the lateral loads induced by earthquake ground shaking. If an SSC is designed for insufficient lateral forces or if deflections are unacceptably large, damage can result, even to well-detailed SSCs.
2. Failures in low ductility modes (e.g., shear behavior) or due to instability that tend to be abrupt and potentially catastrophic must be avoided. SSCs must be detailed in a manner to achieve ductile behavior such that they have greater energy absorption capacity than the energy content of earthquakes.
3. Building structures and equipment which are base supported tend to be more susceptible to earthquake damage (because of inverted pendulum behavior) than distributed systems which are supported by hangers with ductile connections (because of pendulum restoring forces).
4. The behavior of an SSC as it responds to earthquake ground motion must be fully understood by the designer such that a "weak link" that could produce an unexpected failure is not overlooked. Also, the designer must consider both relative displacement and inertia (acceleration) induced seismic failure modes.
5. SSCs must be constructed in the manner specified by the designer. Materials must be of high quality and as strong as specified by the designer. Construction must be of high quality and must conform to the design drawings.

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By the NPH Implementation Guide for DOE Order 420.1 (Ref. C-67) and this standard, probabilistic performance goals are used as a target for formulating deterministic seismic design criteria. Table C-1 defines seismic performance goals for structures, systems, or components (SSCs) assigned to Performance Categories 1 through 4. SSCs are to be assigned to performance categories in accordance with DOE-STD-1021-93 (Ref. C-26). The seismic performance goals are defined in terms of a permissible annual probability of unacceptable performance P_F (i.e., a permissible failure frequency limit). Seismic induced unacceptable performance should have an annual probability less than or approximately equal to these goals.

Table C-1 Structure, System, or Component (SSC) Seismic Performance Goals for Various Performance Categories

Performance Category	Performance Goal Description	Seismic Performance Goal Annual Probability of Exceeding Acceptable Behavior Limits, P_F
1	Maintain Occupant Safety	$\approx 10^{-3}$ of the onset of SSC[1] damage to the extent that occupants are endangered
2	Occupant Safety, Continued Operation with Minimum Interruption	$\approx 5 \times 10^{-4}$ of SSC damage to the extent that the component cannot perform its function
3	Occupant Safety, Continued Operation, Hazard Confinement	$\approx 10^{-4}$ of SSC damage to the extent that the component cannot perform its function
4	Occupant Safety, Continued Operation, Confidence of Hazard Confinement	$\approx 10^{-5}$ of SSC damage to the extent that the component cannot perform its function

(1) SSC refers to structure, distribution system, or component (equipment).

The performance goals shown in Table C-1 include both quantitative probability values and qualitative descriptions of acceptable performance. The qualitative descriptions of expected performance following design/evaluation levels of earthquake ground motions are expanded in Table C-2. These descriptions of acceptable performance are specifically tailored to the needs in many DOE facilities.

The performance goals described above are achieved through the use of DOE seismic design and evaluation provisions which include: (1) lateral force provisions; (2) story drift/damage control provisions; (3) detailing for ductility provisions; and (4) quality assurance provisions. These provisions are comprised of the following four elements taken together: (1) seismic loading; (2) response evaluation methods; (3) permissible response levels; and (4) ductile detailing requirements. Acceptable performance (i.e., achieving performance goals) can only be reached by consistent specification of all design criteria elements as shown in Figure C-1.

Table C-2 Qualitative Seismic Performance Goals

PC	Occupancy Safety	Concrete Barrier	Metal Liner	Component Functionality	Visible Damage
1	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Confinement not required.	Confinement not required.	Component will remain anchored, but no assurance it will remain functional or easily repairable.	Building distortion will be limited but visible to the naked eye.
2	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Concrete walls will remain standing but may be extensively cracked; they may not maintain pressure differential with normal HVAC. Cracks will still provide a tortuous path for material release. Don't expect largest cracks greater than 1/2 inch.	May not remain leak tight because of excessive distortion of structure.	Component will remain anchored and majority will remain functional after earthquake. Any damaged equipment will be easily repaired.	Building distortion will be limited but visible to the naked eye.
3	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Concrete walls cracked; but small enough to maintain pressure differential with normal HVAC. Don't expect largest cracks greater than 1/8 inch.	Metal liner will remain leak tight.	Component anchored and functional.	Possibly visible local damage but permanent distortion will not be immediately apparent to the naked eye.
4	No structural collapse, failure of contents not serious enough to cause severe injury or death, or prevent evacuation	Concrete walls cracked; but small enough to maintain pressure differential with normal HVAC. Don't expect largest cracks greater than 1/8 inch.	Metal liner will remain leak tight.	Component anchored and functional.	Possibly visible local damage but permanent distortion will not be immediately apparent to the naked eye.

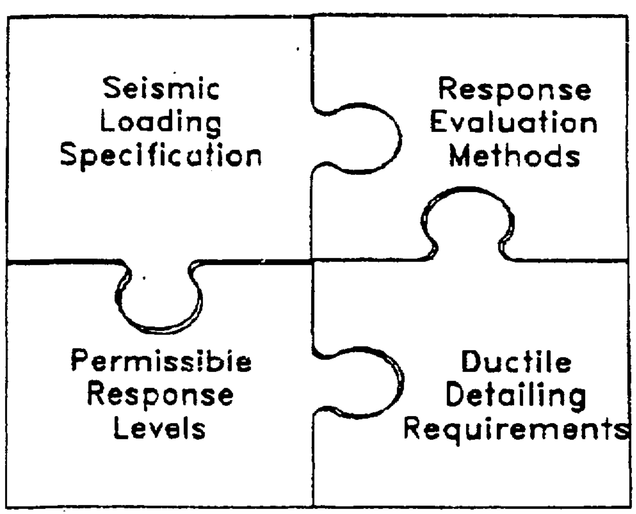


Figure C-1 Consistent Specification of All Seismic Design/Evaluation Criteria Elements

C.2 Basic Approach for Earthquake Design and Evaluation and Meeting Target Performance Goals

C.2.1 Overall Approach for DOE Seismic Criteria

Structure/component performance is a function of: (1) the likelihood of hazard occurrence and (2) the strength of the structure or equipment item. Consequently, seismic performance depends not only on the earthquake probability used to specify design seismic loading, but also on the degree of conservatism used in the design process as illustrated in Figure C-2. For instance, if one wishes to achieve less than about 10^{-4} annual probability of onset of loss of function, this goal can be achieved by using conservative design or evaluation approaches for a natural phenomena hazard that has a more frequent annual probability of exceedance (such as 10^{-3}), or it can be achieved by using median-centered design or evaluation approaches (i.e., approaches that have no intentional conservative or unconservative bias) coupled with a 10^{-4} hazard definition. At least for the earthquake hazard, the former alternate has been the most traditional. Conservative design or evaluation approaches are well-established, extensively documented, and commonly practiced. Median design or evaluation approaches are currently controversial, not well understood, and seldom practiced. Conservative design and evaluation approaches are utilized for both conventional facilities (similar to DOE Performance Category 1) and for nuclear power plants (similar to DOE Performance Category 4). For consistency with these other uses, the approach in this standard specifies the use of conservative design and evaluation procedures coupled with a hazard definition consistent with these procedures.

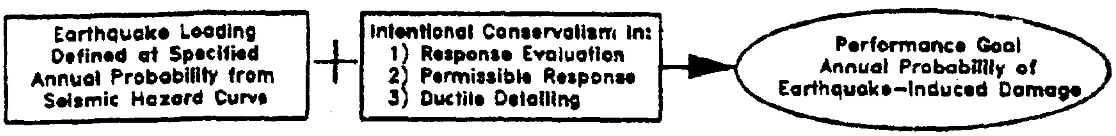


Figure C-2 Performance Goal Achievement

The performance goals for Performance Category 1 SSCs are consistent with goals of model building codes for normal facilities; the performance goals for Performance Category 2 SSCs are slightly more conservative than the goals of model building codes for important or essential facilities. For seismic design and evaluation, model building codes utilize equivalent static force methods except for very unusual or irregular facilities, for which a dynamic analysis method is employed. The performance goals for Performance Category 3 SSC's are consistent with DOE essential facilities and Pu handling facilities. The performance goals for Performance Category 4 SSC's approach those used for nuclear power plants. For these reasons, this standard specifies seismic design and evaluation criteria for PC-1 and PC-2 SSC's corresponding closely to model building codes and seismic design and evaluation criteria for both PC-3 and PC-4 SSC's based on dynamic analysis methods consistent with those used for similar nuclear facilities.

By this standard, the DBE is defined at specified hazard probability P_H and the SSC is designed or evaluated for this DBE using an adequately conservative deterministic acceptance criteria. To be adequately conservative, the acceptance criteria must introduce an additional reduction in the risk of unacceptable performance below the annual risk of exceeding the DBE. The ratio of the seismic hazard exceedance probability, P_H to the performance goal probability P_F is defined herein as the risk reduction ratio R_R, given by:

$$R_R = \frac{P_H}{P_F} \qquad \text{(C-1)}$$

The required degree of conservatism in the deterministic acceptance criteria is a function of the specified risk reduction ratio. Table C-3 provides a set of seismic hazard exceedance probabilities, P_H and risk reduction ratios, R_R for Performance Categories 1 through 4 required to achieve the seismic performance goals specified in Table C-1. Note that Table C-3 follows the philosophy of:

1) gradual reduction in hazard annual exceedance probability
2) gradual increase in conservatism of evaluation procedure as one goes from Performance Category 1 to Performance Category 4 (PC 1 to PC 4).

Table C-3 Seismic Performance Goals & Specified Seismic Hazard Probabilities

Performance Category	Target Seismic Performance Goal, P_F	Seismic Hazard Exceedance Probability, P_H	Risk Reduction Ratio, R_R
1	1×10^{-3}	2×10^{-3}	2
2	5×10^{-4}	1×10^{-3}	2
3	1×10^{-4}	5×10^{-4} (1×10^{-3})[1]	5 (10)[1]
4	1×10^{-5}	1×10^{-4} (2×10^{-4})[1]	10 (20)[1]

[1] For sites such as LLNL, SNL-Livermore, SLAC, LBL, and ETEC which are near tectonic plate boundaries.

Different structures, systems, or components may have different specified performance goal probabilities, P_F. It is required that for each structure, system, or component, either: (1) the performance goal category; or (2) the hazard probability (P_H) or the DBE together with the appropriate R_R factor will be specified in a design specification or implementation document that invokes these criteria. As shown in Table 2-3, the recommended hazard exceedance probabilities and performance goal exceedance probabilities are different. These differences indicate that conservatism must be introduced in the seismic behavior evaluation approach to achieve the required risk reduction ratio, R_R. In earthquake evaluation, there are many places where conservatism can be introduced, including:

1. Maximum design/evaluation ground acceleration and velocity.
2. Response spectra amplification.
3. Damping.
4. Analysis methods.
5. Specification of material strengths.
6. Estimation of structural capacity.
7. Load or scale factors.
8. Importance factors.
9. Limits on inelastic behavior.
10. Soil-structure interaction (except for frequency shifting due to SSI).
11. Effective peak ground motion.
12. Effects of a large foundation or foundation embedment.

For the earthquake evaluation criteria in this standard, conservatism is intentionally introduced and controlled by specifying (1) hazard exceedance probabilities, (2) load or scale factors, (3) importance factors, (4) limits on inelastic behavior, and (5) conservatively specified material strengths and structural capacities. Load and importance factors have been retained for the evaluation of Performance Category 2 and lower SSCs because the UBC approach (which includes these factors) is followed for these categories. Importance factors are not used for Performance Category 3 and higher SSCs. However, a seismic scale factor SF is used to provide the difference in risk reduction ratio R_R between Performance Categories 3 and 4. Material strengths and structural capacities specified for Performance Category 3 and higher SSCs correspond to ultimate strength code-type provisions (i.e., ACI 318-89 for reinforced concrete, LRFD, or AISC Chapter N for steel). Material strengths and structural capacities specified for Performance Category 2 and lower SSCs correspond to either ultimate strength or allowable stress code-type provisions. It is recognized that such provisions introduce conservatism. In addition, significant additional conservatism can be introduced if considerations of effective peak ground motion, soil-structure interaction, and effects of large foundation or foundation embedment are ignored.

The differences in seismic evaluation criteria among categories in terms of load and importance factors, limits on inelastic behavior, and other factors by this standard are summarized below:

1.	PC 1 and PC 2	From PC 1 to PC 2, seismic hazard exceedance probability is lowered and importance factor is increased. All other factors are held the same.
2.	PC 2 and PC 3	From PC 2 to PC 3, load and importance factors are eliminated, damping is generally increased, and limits on inelastic behavior are significantly reduced. All other factors are essentially the same, although static force evaluation methods are allowed for PC 2 SSCs and dynamic analysis is required for PC 3 SSCs.
3.	PC 3 and PC 4	From PC 3 to PC 4, seismic hazard exceedance probability is lowered and a seismic scale factor is used. All other factors are held the same.

The basic intention of the deterministic seismic evaluation and acceptance criteria presented in Chapter 2 is to achieve less than a 10% probability of unacceptable performance for a structure, system, or component (SSC) subjected to a Scaled Design/Evaluation Basis Earthquake (SDBE) defined by:

$$SDBE = (1.5SF)(DBE) \tag{C-2}$$

where SF is the appropriate seismic scale factor (SF is 1.0 for PC 3 and 1.25 for PC 4). The seismic evaluation and acceptance criteria presented in this standard has intentional and controlled conservatism such that the required risk reduction ratios, R_R, and target performance goals are achieved. The amount of intentional conservatism has been evaluated in Reference C-20 as that there should be less than 10% probability of unacceptable performance at input ground motion defined by a scale factor of 1.5SF times the DBE. Equation C-2 is useful for developing alternative evaluation and acceptance criteria which are also based on the target performance goals.

It is permissible to substitute alternate acceptance criteria for those criteria defined in Chapter 2 so long as these alternate criteria will also reasonably achieve less than about a 10% probability of unacceptable performance for the combination of the SDBE defined by Equation C-2 with the best-estimate of the concurrent non-seismic loads. This relief is permitted to enable one to define more sophisticated alternate acceptance criteria than those presented in Chapter 2 when one has a sufficient basis to develop and defend this alternate criteria.

C.2.2 Influence of Seismic Scale Factor

The target performance goals of the Implementation Guide for DOE Order 420.1 are the basis of the seismic design and evaluation criteria presented in this standard. It is known that for PC 1 and PC 2, target performance goals, P_F, of 1×10^{-3} and 5×10^{-4}, respectively, are met relatively closely. However, for PC 3 and PC 4, target performance goals, P_F, of $1x10^{-4}$ and $1x10^{-5}$, respectively, are met in a more approximate manner as illustrated in this section. The variability in performance goal achievement can be most significantly attributed to the uncertainty in the slopes of seismic hazard curves from which DBE ground motion is determined. Seismic hazard curve slope does not have a significant effect on performance for PC 1 and PC 2 because P_F and P_H do not differ greatly (i.e. $R_R = P_H/P_F = 2$).

Over any ten-fold difference in exceedance probabilities, seismic hazard curves may be approximated by:

$$H(a) = K a^{-k_H} \tag{C-3}$$

where H(a) is the annual probability of exceedance of ground motion level "a," K is a constant, and k_H is a slope parameter. Slope coefficient, A_R is the ratio of the increase in ground motion corresponding to a ten-fold reduction in exceedance probability. A_R is related to k_H by:

$$k_H = \frac{1}{\log(A_R)} \tag{C-4}$$

The Basis for Seismic Provisions of DOE-STD-1020 (Ref. C-20) presents estimates of seismic hazard curve slope ratios A_R for typical U.S. sites over the annual probability range of 10^{-3} to 10^{-5}. For eastern U.S. sites, A_R typically falls within the range of 2 to 4 although A_R values as large as 6 have been estimated. For California and other high seismic sites near tectonic plate boundaries with seismicity dominated by close active faults with high recurrence rates, A_R typically ranges from 1.5 to 2.25. For other western sites with seismicity not dominated by close active faults with high recurrence rates such as INEL, LANL, and Hanford, A_R typically ranges from 1.75 to 3.0. Therefore, seismic design/evaluation criteria should be applicable over the range of A_R from 1.5 to 6 with emphasis on the range from 2 to 4.

DOE seismic design and evaluation criteria presented in Chapter 2 is independent of A_R and, thus, does not reflect its effect on meeting target goals. The performance of structures, systems, and components in terms of annual probability of exceeding acceptable behavior limits can be evaluated by convolution of seismic hazard and seismic fragility curves. Seismic fragility curves describe the probability of unacceptable performance versus ground motion level. The fragility curve is defined as being lognormally distributed and is expressed in terms of two parameters: a median capacity level, C_{50}, and a logarithmic standard deviation, β. β expresses the uncertainty in the capacity level and generally lies within the range of 0.3 to 0.6. For DBE ground motion specified at annual probability, P_H, it is shown in Ref. C-20 that the risk reduction ratio, R_R, between the annual probability of exceeding the DBE and the annual probability of unacceptable performance is given by:

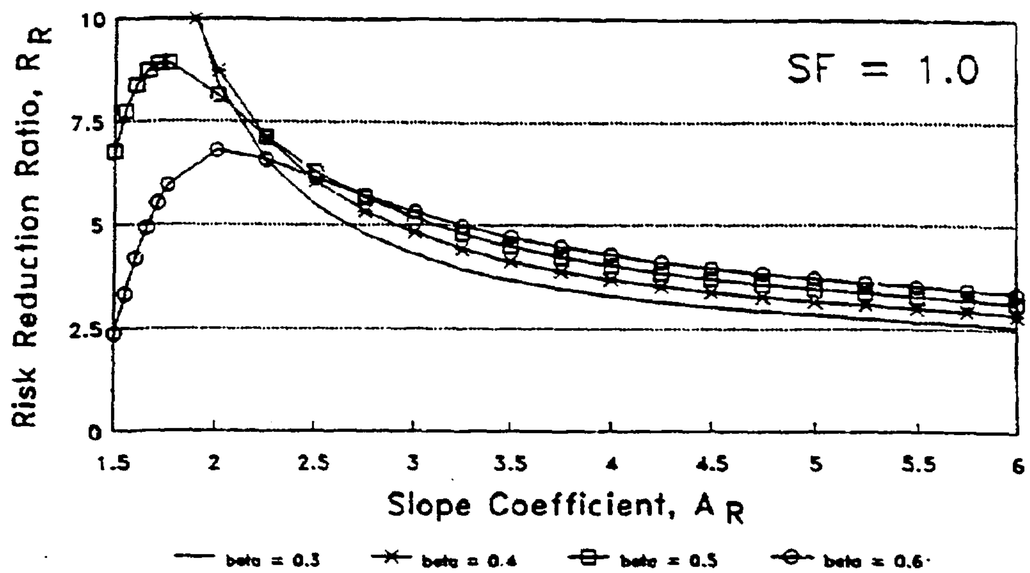
$$R_R = (C_{50}/\mathrm{DBE})^{k_H} e^{-\frac{1}{2}(k_H\beta)^2} \tag{C-5}$$

where C_{50} and β define the seismic fragility curve and DBE and k_H define the seismic hazard curve.

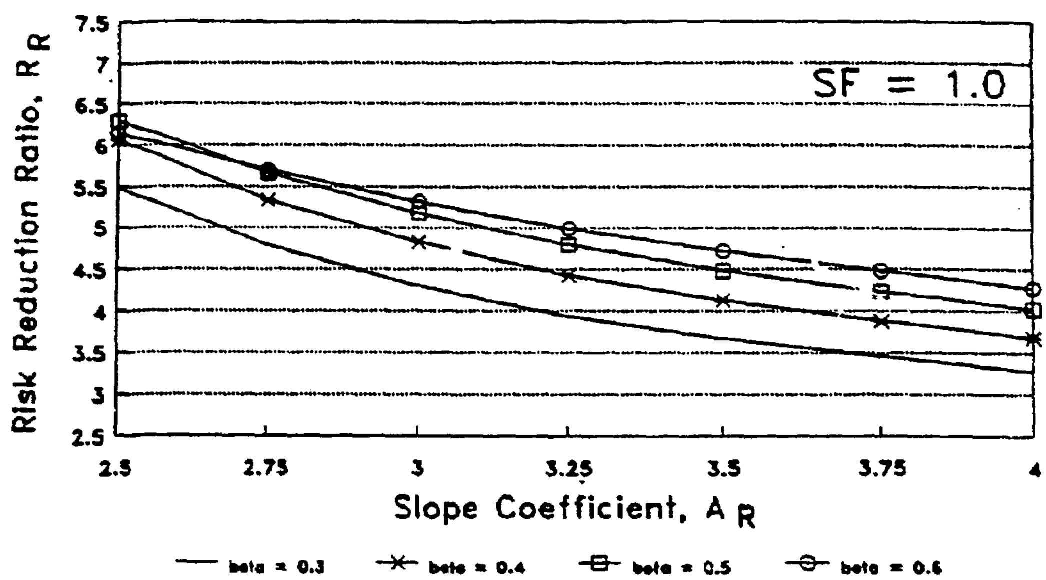
Using the basic criterion of DOE-STD-1020 that target performance goals are achieved when the minimum required 10% probability of failure capacity, C_{10} is equal to 1.5 times the seismic scale factor, SF, times the DBE ground motion, Equation (C-5) may be rewritten as:

$$R_R = (1.5\mathrm{SF})^{k_H} e^{\left[1.282k_H\beta-\frac{1}{2}(k_H\beta)^2\right]} \tag{C-6}$$

Equation (C-6) demonstrates the risk reduction ratio achieved by DOE seismic criteria as a function of hazard curve slope, uncertainty, β, and seismic scale factor, SF. Note from Table C-3 that for Performance Category 4 (not near tectonic plate boundaries), the hazard probability is 1×10^{-4} and the performance goal is 1×10^{-5} such that the target risk reduction ratio, R_R is 10 and for Performance Category 3, the hazard probability is 5×10^{-4} and the performance goal is 1×10^{-4} such that the target risk reduction ratio, R_R is 5. The actual risk reduction ratios from Equation (C-6) versus slope coefficient A_R are plotted in Figures C-3 and C-4 for Performance Categories 3 and 4, respectively. In these figures, SF of 1.0 is used for PC 3 and SF of 1.25 is used for PC 4 and the range of β from 0.3 to 0.6 has been considered. For the hazard curves considered by DOE-STD-1024-92 (Ref. C-13), A_R values average about 3.2 in the probability range associated with PC 3 and about 2.4 in the probability range associated with PC 4. More recent seismic hazard studies (Ref. C-6) gives A_R values which average about 3.8 in the probability range associated with PC 3 and about 3.0 in the probability range associated with PC 4. As a result, Figure C-3 includes a blown-up view for the 2.5 to 4 A_R range and Figure C-4 includes a blown-up view for the 2 to 3 A_R range.

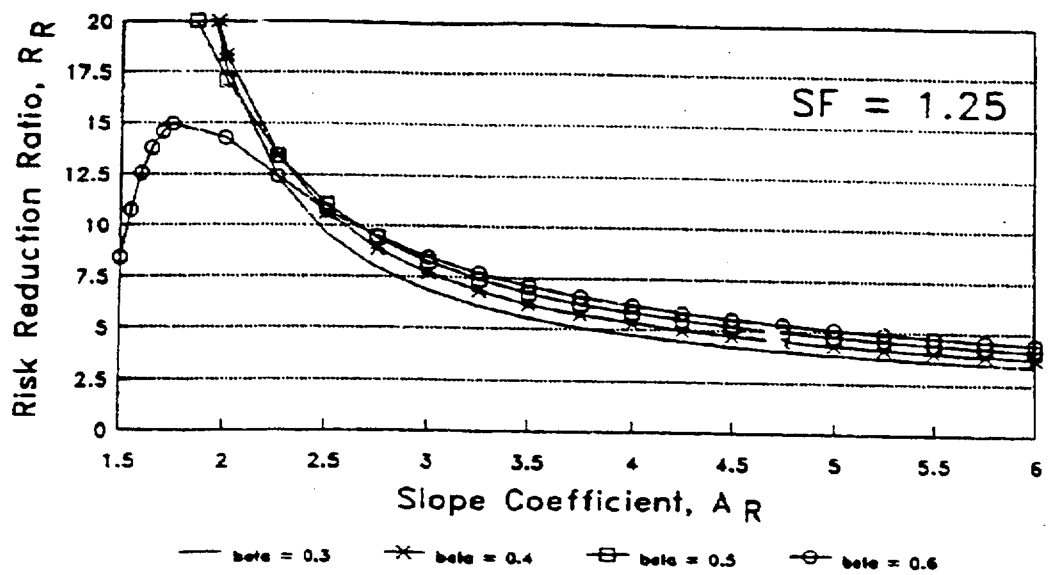


a) A_R from 1.5 to 6

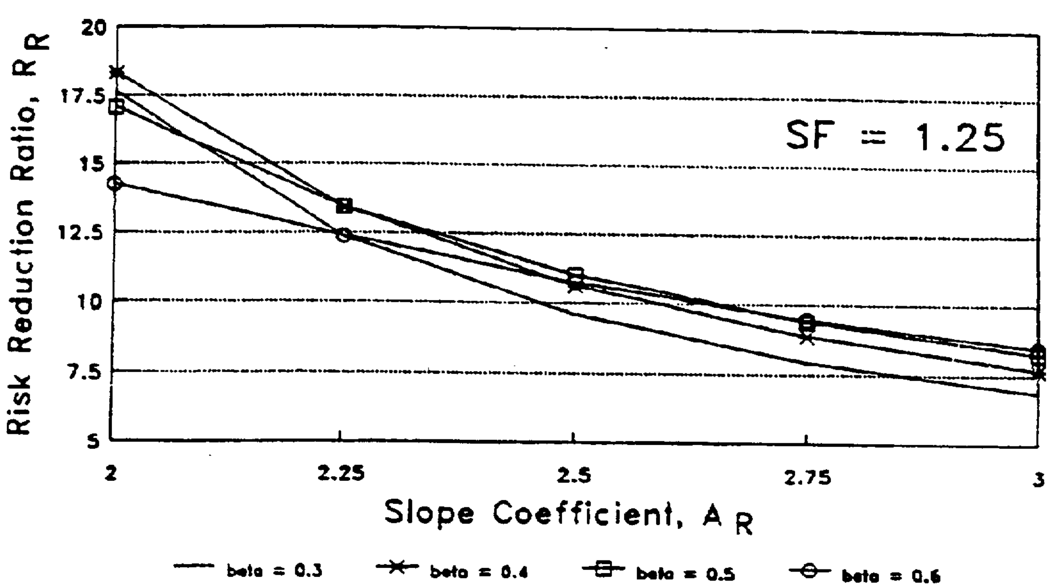


b) A_R from 2.5 to 4

Figure C-3 Value of R_R vs A_R for SF = 1.0 (PC 3)



a) A_R from 1.5 to 6



b) A_R from 2 to 3

Figure C-4 Value of R_R vs A_R for SF = 1.25 (PC 4)

Figure C-3 demonstrates that for SF = 1.0, risk reduction ratios between about 3 and 10 are achieved over the A_R range from 2 to 6. These risk reduction ratios support achieving performance goals between about $2x10^{-4}$ to $5x10^{-5}$. In the primary region of interest of A_R between 2.5 and 4, risk reduction ratios from 4 to 6 are achieved as compared to the target level of 5 for PC 3 and sites not near tectonic plate boundaries. Figure C-4 demonstrates that for SF = 1.25, risk reduction ratios between about 3 and 20 are achieved over the A_R range from 2 to 6. These risk reduction ratios support achieving performance goals between about $3x10^{-5}$ to $5x10^{-6}$. In the primary region of interest of A_R between 2 and 3, risk reduction ratios from about 8 to 17 are achieved as compared to the target level of 10 for PC 4 and sites not near tectonic plate boundaries.

The risk reduction ratio achieved may be improved by using a variable formulation of SF which is a function of A_R. In order to justify use of the variable scale factor approach, the site specific hazard curve must have a rigorous pedigree. Reference C-20 demonstrates that the SF factors shown in Figure C-5 give the best fit of R_R over the A_R range of primary interest from about 2 to about 6. The use of the scale factors given in Figure C-5 combined with Equation C-6 improves the R_R values compared to target values as shown in Figures C-6 and C-7 for PC 3 (R_R = 5) and PC 4 (R_R = 10), respectively. Figures C-6 and C-7 demonstrate that when the variable scale factors from Figure C-5 are used, risk reduction factors achieved are within about 10% of the target values of 5 and 10, respectively. As a result, target performance goals would be met within about the same 10%.

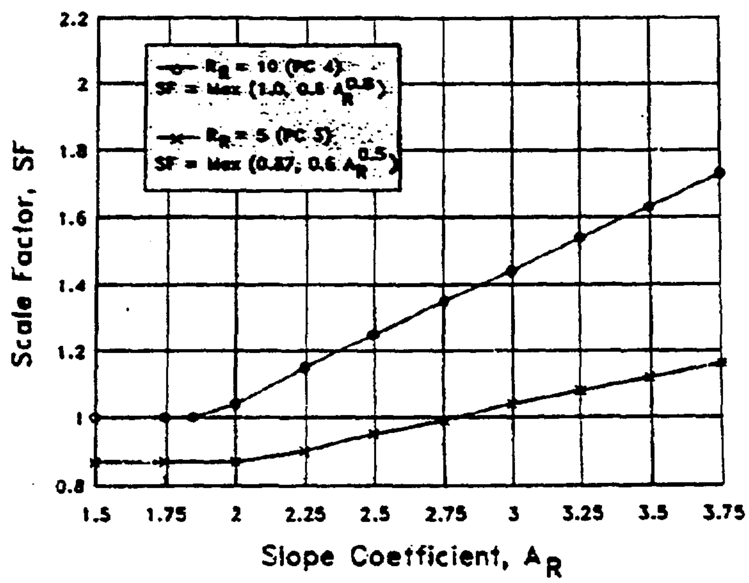


Figure C-5 Variable II Seismic Scale Factor for PC 3 and PC 4

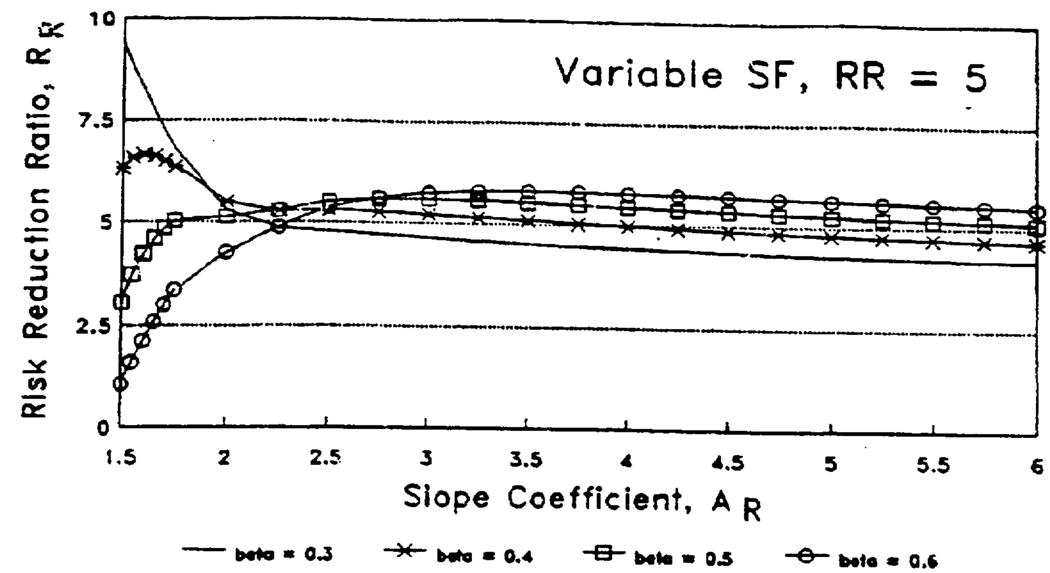


Figure C-6 Value of R_R vs A_R for Variable SF (Fig. C-5 for PC 3)

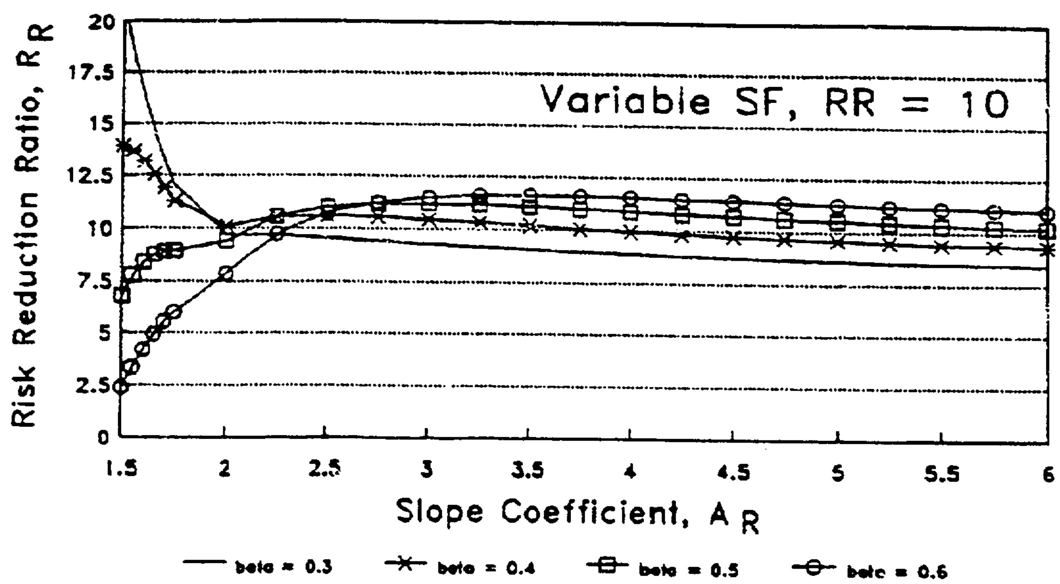


Figure C-7 Value of R_R vs A_R for Variable SF (Fig. C-5 for PC 4)

For sites near tectonic plate boundaries for which A_R is in the range of about 1.5 to 2.25, such as LLNL, SNL-Livermore, SLAC, LBL, and ETEC. Figures C-3a and C-4a demonstrate that larger risk reduction ratios are achieved than the target levels of 5 for PC 3 and 10 for PC 4, respectively. Therefore, it is acceptable to use twice the hazard probabilities for these sites combined with the appropriate constant scale factors. Hence, for sites near tectonic plate boundaries, target performance goals may be adequately achieved with hazard probabilities and seismic scale factors of 1×10^{-3} and 1.0 for PC 3 and 2×10^{-4} and 1.25 for PC 4.

C.3 Seismic Design/Evaluation Input

The seismic performance goals presented in Tables C-1 and C-2 are achieved by defining the seismic hazard in terms of a site-specified design response spectrum (called herein, the Design/Evaluation Basis Earthquake, (DBE)). Either a site-specific design response spectrum specifically developed for the site, or a generic design response spectrum that is appropriate or conservative for the site may be used as the site-specified design response spectrum. Probabilistic seismic hazard estimates are used to establish the DBE. These hazard curves define the amplitude of the ground motion as a function of the annual probability of exceedance P_H of the specified seismic hazard.

For each performance category, an annual exceedance probability for the DBE, P_H is specified from which the maximum ground acceleration (or velocity) may be determined from probabilistic seismic hazard curves. Evaluating maximum ground acceleration from a specified annual probability of exceedance is illustrated in Figure C-8. Earthquake input excitation to be used for design and evaluation by these provisions is defined by a median amplification smoothed and broadened design/evaluation response spectrum shape such as that shown in Figure C-8 anchored to this maximum ground acceleration. Note that the three spectra presented in Figure C-8 are identical; the top spectrum has spectral acceleration plotted against natural frequency on a log scale, the middle spectrum is on what is termed a tripartite plot where spectral velocities and displacements as well as accelerations are shown, and the bottom spectrum has spectral acceleration plotted against natural period on a linear scale.

It should be understood that the spectra shown in Figure C-8 represent inertial effects. They do not include relative or differential support motions of structures, equipment, or distribution systems supported at two or more points typically referred to as seismic anchor motion (SAM). While SAM is not usually applicable to building design, it might have a significant effect on seismic adequacy of equipment or distribution systems.

Seismic design/evaluation criteria based on target probabilistic performance goals requires that Design/Evaluation Basis Earthquake (DBE) motions be based on probabilistic seismic hazard assessments. In accordance with DOE Order 420.1 and the associated NPH Implementation Guide (Refs. C-27 and C-67), it is not required that a site-specific probabilistic seismic hazard assessment be conducted if the site includes only Performance Category 2 and lower SSCs. If such an assessment has not been performed, it is acceptable to determine seismic loads (as summarized in Section C.3.2.2) from the larger of those determined in accordance with the UBC (Ref. C-2) and with UCRL-53582, Rev. 1 (Ref. C-14). Design/evaluation earthquake ground motion determined from a recent site-specific probabilistic seismic hazard assessment is considered to be preferable to the UBC for determining ZC. Therefore, the DBE response spectrum for Performance Category 2 and lower may be developed from a new probabilistic seismic hazard assessment following the guidance given herein for Performance Category 3 and higher. However, when design/evaluation earthquake ground motion is based on recent site-specific geotechnical studies and the resulting seismic loads are less than that determined by the UBC, the differences must be justified and approval of seismic loads must be obtained from DOE.

For design or evaluation of SSCs in Performance Category 3 and higher, it is strongly recommended that a modern site-specific seismic hazard assessment be performed to provide the basis for DBE ground motion levels and response spectra. DOE Order 420.1 and the associated NPH Implementation Guide (Refs. C-27 and C-67), require that the need for updating the site seismic hazard assessment be reviewed at least every 10 years. The DOE seismic working group interim standard, DOE-STD-1024-92 (Ref. C-13), indicates that the approach used for the seismic hazard assessments summarized in UCRL-53582 (Ref. C-14), which are more than 10 years old, are out of date relative to the current state of the art. However, in accordance with DOE-STD-1024-92, it is permissible to establish DBE ground motion levels and response spectra for Performance Categories 3 and 4 based on UCRL-53582 in the interim until a modern site-specific seismic hazard assessment becomes available. DBE ground motion levels for Performance Categories 3 and 4 based on UCRL-53582 are also provided in Section C.3.2.2.

Minimum values of the DBE are provided in Section 2.3 to assure a minimum level of seismic design at all DOE sites. Such a minimum level of seismic design is believed to be necessary due to the considerable uncertainty about future earthquake potential in the lower seismicity regions of the United States where most DOE sites are located.

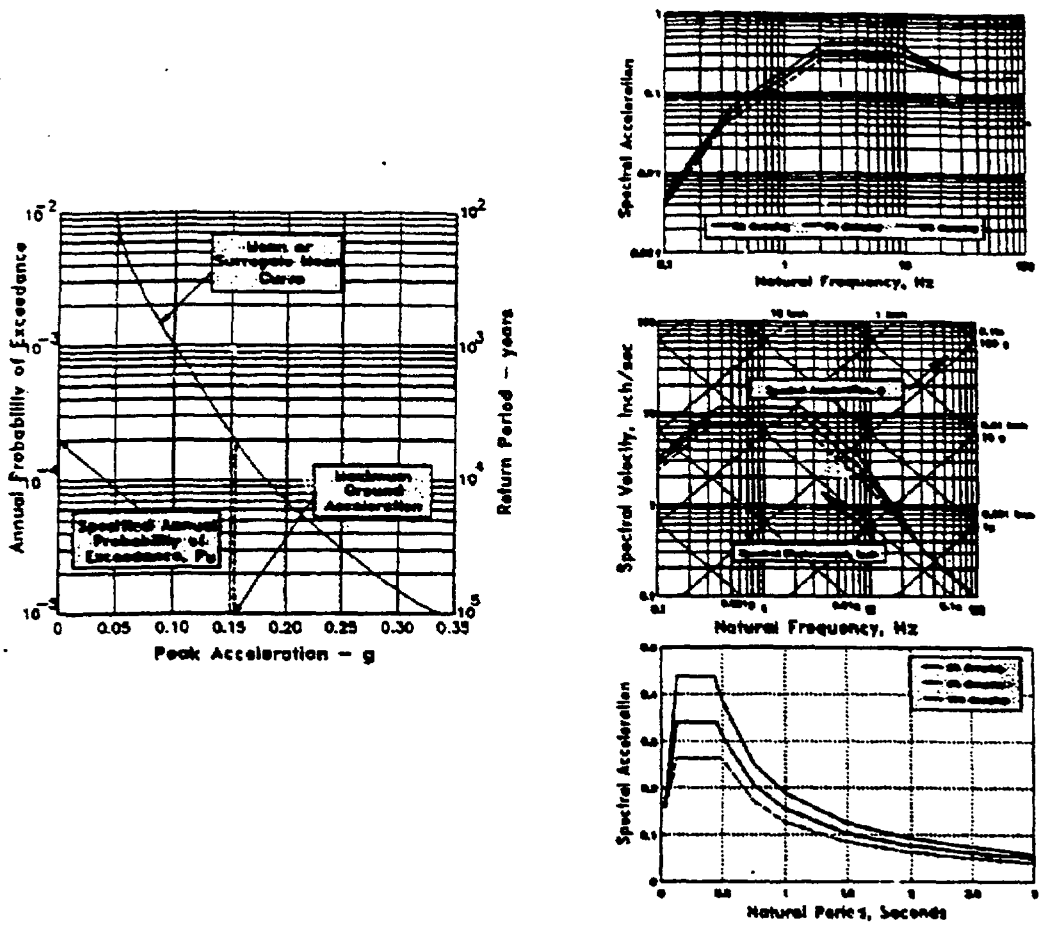


Figure C-8 Earthquake Input Excitation is Defined by Maximum Ground Acceleration Anchoring Site-Specific Response Spectra

C.3.1 Earthquake Hazard Annual Exceedance Probabilities

Historically, non-Federal Government General Use and Essential or Low Hazard facilities located in California, Nevada, and Washington have been designed for the seismic hazard defined in the Uniform Building Code. Other regions of the U.S. have used the UBC seismic hazard definition, other building code requirements, or have ignored seismic design. Past UBC seismic provisions (1985 and earlier) are based upon the largest earthquake intensity that has occurred in a given region during about the past 200 years. These provisions do not consider the probability of occurrence of such an earthquake and thus do not make any explicit use of a probabilistic seismic hazard analysis. However, within the last 15 years there have been developments in building codes in which the seismic hazard provisions are based upon a consistent annual probability of exceedance for all regions of the U.S. In 1978, ATC-3 provided probabilistic-based seismic hazard provisions (Ref. C-1). From the ATC-3 provisions, changes to the UBC (Ref. C-2) and the development of the National Earthquake Hazards Reduction Program (NEHRP, Ref. C-3) have resulted. A probabilistic-based seismic zone map was incorporated into the UBC beginning with the 1988 edition. Canada and the U.S. Department of Defense have adopted this approach (Refs. C-4 and C-5). The suggested annual frequency of exceedance for the design seismic hazard level differs somewhat between proposed codes, but all lie in the range of 10^{-2} to 10^{-3}. For instance, UBC (Ref. C-2), ATC-3 (Ref. C-1), and NEHRP (Ref. C-3) have suggested that the design seismic hazard level should have about a 10 percent frequency of exceedance level in 50 years which corresponds to an annual exceedance frequency of about $2x10^{-3}$. The Canadian building code used $1x10^{-2}$ as the annual exceedance level for their design seismic hazard definition. The Department of Defense (DOD) tri-services seismic design provisions for essential buildings (Ref. C-5) suggests a dual level for the design seismic hazard. Facilities should remain essentially elastic for seismic hazard with about a 50 percent frequency of exceedance in 50 years or about a $1x10^{-2}$ annual exceedance frequency, and they should not fail for a seismic hazard which has about a 10 percent frequency of exceedance in 100 years or about $1x10^{-3}$ annual exceedance frequency.

On the other hand, nuclear power plants are designed so that safety systems do not fail if subjected to a safe shutdown earthquake (SSE). The SSE generally represents the expected ground motion at the site either from the largest historic earthquake within the tectonic province within which the site is located or from an assessment of the maximum earthquake potential of the appropriate tectonic structure or capable fault closest to the site. The key point is that this is a deterministic definition of the design SSE. Recent probabilistic hazard studies (e.g., Ref. C-6) have indicated that for nuclear plants in the eastern U.S., the design SSE level generally corresponds to an estimated annual frequency of exceedance of between $0.1x10^{-4}$ and $10x10^{-4}$ as is illustrated in Figure C-9. The probability level of SSE design spectra (between 5 and 10 hz) at the 69 eastern U.S. nuclear power plants considered by Ref. C-6 fall within the above stated range. Figure C-9 also demonstrates that for 2/3 of these plants the SSE spectra corresponds to probabilities between about $0.4x10^{-4}$ and $2.5x10^{-4}$. Hence, the specified hazard probability level of $1x10^{-4}$ in this standard is consistent with SSE levels.

These seismic hazard definitions specified in this standard are appropriate as long as the seismic design or evaluation of the SSCs for these earthquake levels is conservatively performed. The level of conservatism of the evaluation for these hazards should increase as one goes from Performance Category 1 to 4 SSCs. The conservatism associated with Performance Categories 1 and 2 should be consistent with that contained in the UBC (Ref. C-2), ATC-3 (Ref. C-1), or NEHRP (Ref. C-3) for normal or essential facilities, respectively. The level of conservatism in the seismic evaluation for Performance Category 4 SSCs should approach that used for nuclear power plants when the seismic hazard is designated as shown above. The criteria contained herein follow the philosophy of a gradual reduction in the annual exceedance probability of the hazard coupled with a gradual increase in the conservatism of the evaluation procedures and acceptance criteria as one goes from Performance Category 1 to Performance Category 4.

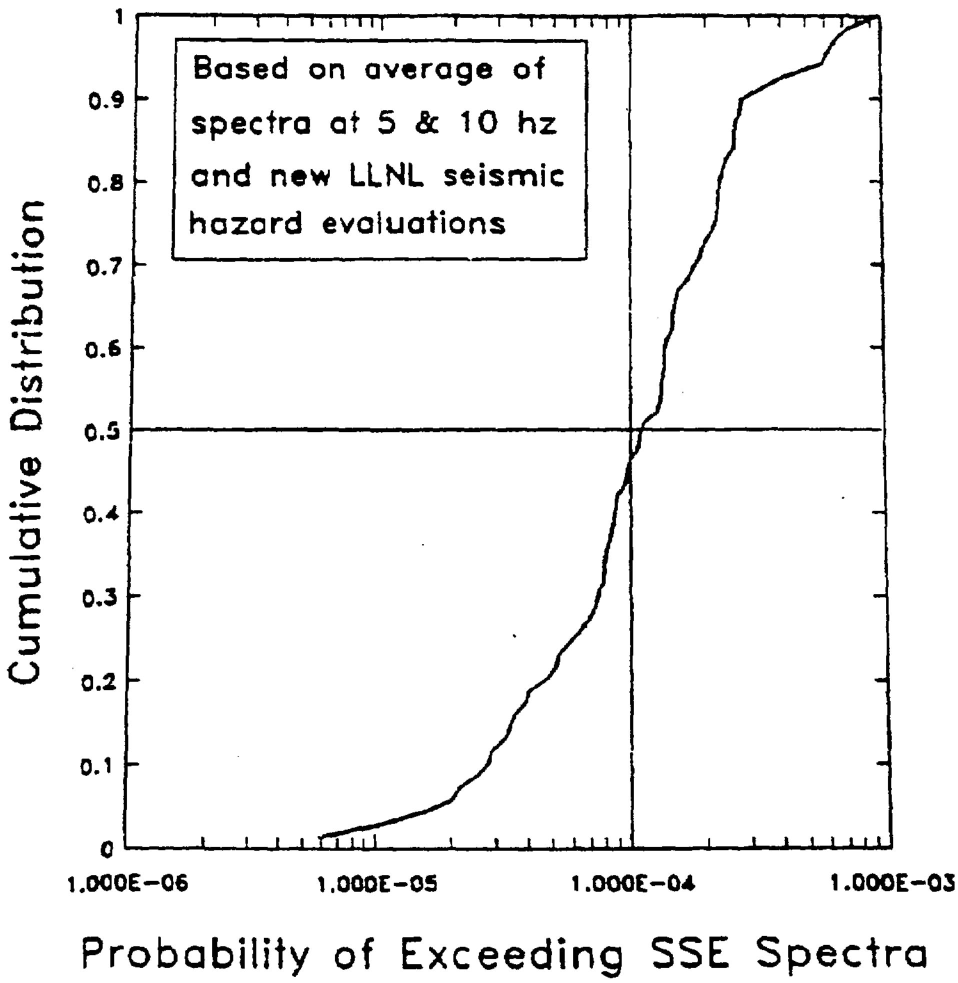


Figure C-9 Probability of Exceeding SSE Response Spectra

40632

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UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	) Docket No. 72-22
	) ASLPB No. 97-732-02-ISFSI
PRIVATE FUEL STORAGE L.L.C.	) DEPOSITION OF:
(Private Fuel Storage Facility)	) **DR. WALTER J. ARABASZ**
	) **(Utah Contention L, Part B)**

Wednesday, October 31, 2001 - 9:20 a.m.

Location: PARSONS, BEHLE & LATIMER
201 S. Main, Suite 1800
Salt Lake City, UT 84111

Reporter: Vicky McDaniel
Notary Public in and for the State of Utah

requirements for seismic design of new nuclear power plants, correct?

A. Correct.

Q. And they have adopted a PSHA, probability seismic hazard analysis, approach for new nuclear power plants, correct?

A. As an allowable option, in my understanding.

Q. And the use of a PSHA -- well, strike that. Are you aware generally that the NRC is moving towards risk-informed regulation?

A. Yes, I am.

Q. And use of a PSHA would be in accordance with the NRC's movement toward a risk-informed regulation?

A. Correct.

Q. Isn't one of the advantages of using a PSHA analysis for earthquakes as opposed to a deterministic analysis that you're better able to incorporate risk and uncertainty into your analysis?

A. Correct.

Q. How would you generally describe these advantages in practical terms? Why -- I take it you would favor the use of a PSHA generally as opposed to a deterministic method?

A. I recall in my last deposition saying that yes, because of my involvement in the evolution of PSHA that I understand its benefits and agree to them.

Q. Therefore, as far as this contention is concerned, the issue as far as you're concerned is what the level of the return period should be for picking the design level for ISFSIs with respect to a PSHA analysis?

A. Yes. I think simply put, it would be pinning down what are to be the applicable regulations and standards.

Q. Insofar as use of the probability seismic hazard analysis approach would be?

A. Correct.

Q. Now, you referred to the rulemaking plan, and that is referenced in item 1 under Part B of Utah L.

A. Correct.

Q. And the rulemaking plan that you're referencing there is set forth in a SECY paper 98-126 dated June 4, 1998?

A. Correct.

Q. And what is your understanding of divisions of the June 1998 rulemaking plan in terms of what are provided for?

A. That's in -- first, that the staff presented

A. I don't take issue with that, no.

Q. And do you take issue with the second sentence in that first statement or bullet where it says, "In its Statement of Consideration accompanying the rulemaking for 10 CFR Part 72, the NRC recognized the reduced radiological hazards associated with dry cask storage facilities and stated that the seismic design-basis ground motions for these facilities may not be as high as for commercial nuclear power plants"? Do you agree with that statement, that the design-basis ground motions for ISFSIs may not be as high as those for commercial nuclear power plants, given their reduced hazards?

A. It seems logical. I don't take great issue with it, no.

Q. And generally do you agree with the graded approach in terms of seismic design requirements for facilities linked to their use or potential hazards?

A. It seems rational and needed, yes.

Q. So therefore I take it that with Basis 3 -- going back to graduated approaches, in fact doesn't the Uniform Building Code, International Building Code provide for graduated approaches for seismic design requirements for structures?

A. Yes, they do.

Q. And similarly DOE 1020 provides for graduated design requirements for structures?

A. Correct.

Q. And so your area of this disagreement with the staff I take it concerns the second statement that appears on Exhibit 3?

A. That's correct, the second and third.

Q. Second and third, okay. And there the staff claims that the reference probability for nuclear power plants as set forth in Reg Guide 1.165 of $1E^{-5}$ is expressed as the median annual probability of exceedance, and they claim that is the same as the -- as a mean annual probability of exceedance of $1E^{-4}$.

A. That's correct.

Q. And you take issue with that statement as it's applied in the context here with respect to Private Fuel Storage facility?

A. Yes.

Q. If I understand your position correctly, it's that the statement that a median of $1E^{-5}$ is the same as a mean annual probability of exceedance of $1E^{-4}$ is based on plants and experience in the central and eastern United States?

A. That's correct.

Q. And it's your position that for plants in

A. Performance category 3, yes.

Q. And it had a performance objective of what?

A. $1\text{x}10^{-4}$.

Q. And what do you understand that performance objective to mean in practical terms?

A. It's the annual probability of exceedance relating to some limits of acceptable behavior. I think that's the type of wording that DOE uses to define a seismic performance goal so that the annual probability of not exceeding some defined consequence, some adverse consequence would be $1\text{x}10^{-4}$.

Q. And then you have performance category 4 facilities, and they have a 10^{-4} probability exceedance hazard?

A. I believe that's correct, yes.

Q. And the 10^{-5} objective performance; is that correct?

A. That's correct, to the best of my memory, yes.

Q. As we talked about before, 10^{-4} corresponds to nuclear -- excuse me -- performance category 4 corresponds to nuclear power plants?

A. Yes.

Q. And it would be appropriate in terms of DOE Standard 1020 for ISFSIs to be under performance category 3?

A. That's my general understanding.

Q. And you would agree with that classification for ISFSIs under DOE Standard 1020?

A. I personally would, yes.

Q. So therefore under the 1994 version of the DOE Standard 1020, that would provide for ISFSIs such as the PSFS a use of a mean exceedance hazard of $5x10^{-4}$ for design, correct?

A. Correct.

Q. With the objective goal of some consequence not exceeding 10^{-4}?

A. Correct.

Q. And if that approach were adopted, you would find that approach acceptable?

A. I have to -- let's see. I guess I'm speaking as an advisor to the state and as an expert. Everything in my understanding would say yes, this is a rational approach.

Q. From DOE Standard 1020, do you know how this difference between the probability exceedance hazard, for example, at $5x10^{-4}$, and the ultimate objective criteria for performance category 3 facilities is achieved?

A. The document I think implicitly includes

Yucca Mountain Site Characterization Project

TOPICAL REPORT YMP/TR-003-NP

PRECLOSURE SEISMIC DESIGN METHODOLOGY FOR A GEOLOGIC REPOSITORY AT YUCCA MOUNTAIN

Revision 2

August 1997

U.S. Department of Energy
Office of Civilian Radioactive Waste Management
North Las Vegas, NV 89036

Topical Report : Preclosure Seismic Design Methodology for a Geologic Repository at Yucca Mountain

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1.2 CONTENT OF THE SEISMIC TOPICAL REPORTS

This topical report is the second in a series of three reports that the DOE has planned that together will describe the preclosure seismic design process. The relationship of the three topical reports is illustrated in Figure 1-1. Topical Report I, *Methodology to Assess Fault Displacement and Vibratory Ground Motion Hazards at Yucca Mountain* (DOE 1994a), describes the DOE methodology for assessing vibratory ground motion and fault displacement hazards. Topical Report II (this report) describes the DOE preclosure seismic design methodology and design acceptance criteria and establishes seismic hazard levels that are appropriate for design. The DOE anticipates that a third report, currently scheduled for fiscal year 1998, will describe the results of the assessment of the vibratory ground motion and fault displacement hazards at Yucca Mountain and the determination of the appropriate design bases for these hazards.

The content of the three seismic reports is described in more detail in the following paragraphs.

Topical Report I--Topical Report I describes the DOE methodology for probabilistic assessment of vibratory ground motion and fault displacement hazards. The methodology involves a series of workshops structured so that multiple experts can interact to evaluate hypotheses and models using the Yucca Mountain site and area geological, geophysical, and seismological data sets. The data sets will be made available to all participant experts uniformly. Importantly, the methodology requires that the experts specifically evaluate all hypotheses and models that have credible support in the data. The product of the methodology is multiple interpretations by the experts of seismic sources, source properties, and evaluations of ground motion, all of which include specific expressions of uncertainty. The methodology does not involve expert opinion, which implies judgments unconstrained by data or normal scientific rigor, but instead employs normal earth science procedures and practice, and carries the usual past practice one step further by requiring uncertainty in the interpretations to be specifically expressed. Moreover, it forces a consistent level of scientific rigor, a comprehensive and consistent consideration of data, and documentation of all interpretations.

Additional information on the methodology is contained in *Probabilistic Analyses of Ground Motion and Fault Displacement at Yucca Mountain*, Yucca Mountain Study Plan 8.3.1.17.3.6 (DOE 1995a).

Topical Report I does not provide the values of vibratory ground motion and fault displacement hazards for design of the facility SSCs; it describes only the methodology for hazard assessment. The application of this methodology at the Yucca Mountain site will yield hazard estimates that will, together with planned deterministic evaluations, comprise the information base considered in determining preclosure design basis vibratory ground motion and fault displacement values. The hazard estimates will also be used in the assessment of postclosure waste containment and isolation performance.

Topical Report II--Topical Report II (this report) describes the design methodology and criteria that the DOE intends to implement to provide reasonable assurance that vibratory ground motions and fault displacements will not compromise the preclosure safety functions of SSCs important to safety. The seismic design methodology and criteria implement the requirements of 10 CFR 60, including the requirement in the recent ruling (61 FR 64257) to identify Category-1 and -2 design basis events. This report establishes hazard probability levels that are appropriate for determining the two levels of design basis vibratory ground motions and the two levels of design basis fault displacements. Acceptance criteria for both surface and underground facilities are provided for vibratory ground motion and fault displacement design. In addition, the report provides criteria for fault avoidance, which is the DOE preferred approach to mitigating fault displacement hazards. Seismic design considerations for waste packages, which will function on the surface and underground and which have a number of unique performance requirements, are discussed. NRC guidance documents for the seismic design of nuclear power reactors that can appropriately be applied to preclosure seismic design of the repository are identified.

Topical Report III--A third seismic topical report is planned for completion in fiscal year 1998. The DOE intends to conduct and document the probabilistic seismic hazard assessment during fiscal year 1997 using the methodology of Topical Report I. Using the results of the hazard assessment, preclosure seismic design inputs will be developed and documented in a Seismic Design Report, which is scheduled for the second quarter of fiscal year 1998. The third topical report would document the results of both of these efforts for formal NRC staff review.

It is expected that seismic design inputs will be determined from controlling earthquakes identified from a disaggregation of the probabilistic seismic hazard results and from a consideration of deterministic hazard assessments. Disaggregation of the hazard results will be carried out for hazard exceedance probability levels established in Topical Report II and for ground motion frequencies of interest. Different earthquakes may control the hazard in different frequency ranges. Ground motions from the controlling earthquakes will be evaluated deterministically.

In addition to conducting the probabilistic hazard assessment, the DOE intends to perform deterministic evaluations of Type I faults and candidate Type I faults that lie within 5 km of the Yucca Mountain site, including estimations of maximum earthquake magnitudes for the faults. The DOE intends to evaluate where the hazards from these deterministic evaluations fall within the probabilistic results. This comparison will provide a check on the reasonableness of the vibratory ground motion and fault displacement design bases.

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3.0 DESIGN OF STRUCTURES, SYSTEMS, AND COMPONENTS FOR VIBRATORY GROUND MOTION

This section presents and rationalizes the reference exceedance probabilities that the DOE plans to use in identifying Frequency-Category-1 and -2 design basis vibratory ground motions. It then discusses the design acceptance criteria that the DOE plans to apply in the preclosure seismic design of structures, systems and components (SSCs) that are important to safety. Design acceptance criteria are discussed specifically for SSCs on the ground surface, for underground openings, and for other underground SSCs.

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3.1 HAZARD LEVELS FOR DESIGN BASIS GROUND MOTIONS

In accordance with the recent 10 CFR 60 rulemaking discussed in Section 2.1.1, the DOE will identify SSCs that are important to (radiological) safety. The DOE procedure for identifying these SSCs is summarized in Appendix B. The classification process involves the identification of Frequency-Category-1 and Frequency-Category-2 design basis events and event-initiated accident scenarios and the calculation of corresponding exposures to workers and the public. The calculated exposures are compared to regulatory limits, and any SSC that must continue to function after a design basis event to ensure the exposure limits are not exceeded is classified as important to safety. No SSCs have yet been classified. Note that SSCs may be important to safety for both Frequency-Category-1 and Frequency-Category-2 design basis events. Where this occurs, the most stringent (i.e., Frequency-Category-2) design basis will apply.

The regulatory definitions of Category-1 and -2 design basis events are qualitative descriptions of the likelihood of occurrence before permanent closure of the geologic repository operations area. For use in SSC classification, which requires knowledge of the design basis events and calculation of radiation exposures, these definitions require quantitative interpretations. As discussed next, the DOE intends to use mean annual exceedance probabilities of 1.0E-03 and 1.0E-04, respectively, as reference values in determining the Frequency-Category-1 and -2 design basis vibratory ground motions. These reference values will be used in the disaggregation of probabilistic seismic hazard estimates to identify those earthquakes that control the seismic hazard at the reference probabilities. The identification of controlling earthquakes and the DOE determination of the design basis ground motions are planned to be detailed in the third seismic topical report.

3.1.1 Frequency-Category-1 Reference Probability

The DOE intends to use a reference mean annual probability of exceedance of 1.0E-03 in determining the Frequency-Category-1 design basis ground motion. The DOE considers that this probability, which corresponds to a 1,000-year return period, represents a conservative quantitative translation of the qualitative frequency description for Category-1 design basis events in the revised 10 CFR 60, i.e., "events that are reasonably likely to occur regularly, moderately frequently, or one or more times before permanent closure of the geologic repository operations area." Assuming a Poisson temporal occurrence model (see Section 3.3.2.2), events with a 1.0E-03/yr recurrence rate would have an 86 percent chance of not occurring, a 13 percent chance of occurring once, and a 1 percent chance of occurring twice in 150 years. For facilities with a 100-year design lifetime, events with this recurrence rate would have a 90 percent chance of not occurring, a 9 percent chance of occurring once, and a 0.4 chance of occurring twice.

An annual occurrence rate of 1.0E-03 for Frequency-Category-1 design basis ground motions are more conservative than what is required by model building codes for ordinary structures, in terms of the annual probability of occurrence of the design basis earthquake, and is comparably conservative in terms of the probability of occurrence during the facility lifetime. The Uniform Building Code (ICBO 1994) and the National Earthquake Hazards Reduction Program (BSSC 1995) both recommend using peak ground motion values that have a 90 percent chance of not being exceeded in 50 years for the life-safety seismic design of new buildings; this corresponds to a return period of about 500 years. DOE Standard 1020-94 (DOE 1994b) is not being applied to the mined geologic disposal system program, but it documents a general DOE policy that a 500-year return period is to be used in establishing design basis ground motions for general facilities. This return period corresponds to an annual exceedance probability of about 2.0E-03 and a 90 percent chance of not occurring during a typical 50-year facility lifetime.

3.1.2 Frequency-Category-2 Reference Probability

For Frequency-Category-2 design basis ground motion, the DOE intends to use a reference mean annual exceedance probability of 1.0E-04. The DOE considers that this mean value is appropriate and conservative based on the observations that (1) it is comparable to the mean exceedance probabilities of the seismic design bases of operating nuclear power reactors in the United States, (2) these accepted reactor design bases and their associated design-acceptance criteria have resulted in acceptably safe seismic designs, (3) design acceptance criteria will be used in repository design that are the same as or comparable to those used in reactor designs, and (4) an operating mined geologic disposal system is inherently less hazardous and less vulnerable to earthquake-initiated accidents than is an operating nuclear power reactor.

3.1.2.1 Comparison with Nuclear Power Reactor Seismic Design Bases

In Regulatory Guide 1.165 (NRC 1997) NRC staff states that a reference median annual exceedance probability of 1.0E-05 will be acceptable for use in determining the safe shutdown earthquake for new nuclear power reactors. The cited rationale for this reference probability is that it is the annual probability level such that 50 percent of a set of currently operating plants (selected by the NRC) has an annual median probability of exceeding the safe shutdown earthquake that is below this level. In other words, 1.0E-05 is the median of the distribution of median exceedance probabilities. The selected plants represent relatively recent designs that used design response spectra in accordance with Regulatory Guide 1.60, *Design Response Spectra for Seismic Design of Nuclear Power Plants* (AEC 1973), or similar spectra. All of the plants selected are located in the central or eastern United States (CEUS). Regulatory Guide 1.165 provides an option for the applicant to use a different reference probability, to be reviewed and accepted on a case-by-case basis, considering the slope of the site-specific hazard curve, the overall uncertainty in hazard estimates, including differences between mean and median hazard estimates, and knowledge of the seismic sources that contribute to the hazard.

In developing Regulatory Guide 1.165, NRC staff considered whether to define the reference probability as a mean or median value. The mean value has the advantage of better reflecting the uncertainty in the seismic hazard evaluation (i.e., it is sensitive to the range of interpretations of seismic source zone configurations, earthquake magnitude recurrence relationships, and ground motion attenuation relationships). However, precisely because the median is less sensitive to uncertainties, it provides a more stable regulatory benchmark than does the mean. Another consideration leading to the staff's preference for the median was the finding that, when median hazard curves were disaggregated, the magnitudes and distances of the controlling earthquakes tended to be more sharply defined and to agree better with the safe shutdown earthquakes of the selected plants than when mean hazard curves were disaggregated (Bernreuter et al. 1996).

For the reasons discussed next, the DOE plans to use mean, rather than median, target annual exceedance probabilities in establishing design basis vibratory ground motions.

To identify the earthquakes that control the Frequency-Category-2 design basis ground motion, the DOE plans to use a mean annual exceedance probability of 1.0E-04. NRC-sponsored research has shown that a mean value of 1.0E-04 corresponds to a median value of 1.0E-05 at sites in the CEUS (NRC 1994b). That is, while 1.0E-05 is the median of the distribution of median exceedance probabilities of the safe shutdown earthquakes of the more recently designed nuclear power reactors in the CEUS, 1.0E-04 is the median of the distribution of means. So, 50 percent of the nuclear power reactors in the selected set have an annual mean probability of exceeding the safe shutdown earthquake that is below this level. Thus, using a mean value of 1.0E-04 to determine the safe shutdown earthquake for a new nuclear power reactor in the CEUS would be risk-consistent with using a median value of 1.0E-05.

In contrast to sites in the CEUS, the equivalency of 1.0E-04 mean and 1.0E-05 median annual probabilities of exceedance does not generally hold in the western United States and is not expected to hold at Yucca Mountain. Because the distributions of probabilistic seismic hazard estimates typically are skewed about the median towards higher probability levels, mean exceedance probabilities usually are greater than median probabilities, and the greater the uncertainty (i.e., spread of the distribution of hazard curves), the greater the difference between the mean and median values. This fact, together with the fact that the uncertainty in seismic hazard evaluations is almost always greater at CEUS sites than at western sites, indicates that mean values normally are closer to median values at western sites than at CEUS sites. Thus, if one were siting a nuclear power reactor at a typical western U.S. site, choosing a mean annual exceedance probability of 1.0E-04 would be consistent with the mean hazard levels associated with the seismic design bases of more recently designed power reactors in the CEUS, but choosing a median annual probability of 1.0E-05 would not be.

As a further check on the reasonableness of using a mean annual exceedance probability of 1.0E-04 as the reference probability for determining the Frequency-Category-2 design basis ground motion, the DOE compiled published probabilistic seismic hazard estimates for the sites of nuclear power plants in the western United States. The objective of the compilation was to determine whether a mean exceedance probability of 1.0E-04/yr is representative of the accepted seismic design response spectra of these plants, as it is for the more recently designed power plants in the CEUS.

Because the shapes of design response spectra rarely match the shapes of uniform hazard spectra, the probabilities of exceeding design response spectra vary with frequency. Therefore, an averaging convention is required to associate a single probability of exceedance with each design response spectrum. To assure comparability of results, this study used the same convention that was used in the study of CEUS plants (NRC 1994b) and that is recommended in Regulatory Guide 1.165 (NRC 1997), i.e., the average of the exceedance probabilities at 5 Hz and 10 Hz[1].

Footnote [1] There is no tacit assumption here that the 5 to 10 Hz frequency range is representative of the natural frequencies of SSCs in a repository. Repository design response spectra will be developed that cover a broad frequency range from 0.33 Hz to more than 20 Hz.

The power plants for which information was compiled are the Diablo Canyon Power Plant (Units 1 and 2) in Port San Luis, California; Palo Verde Nuclear Generating Station (PVNGS) in Wintersburg, Arizona; San Onofre Nuclear Generating Station (Units 2 and 3) in Southern California; Washington Nuclear Plant 2 near Hanford, Washington; and Washington Nuclear Plant 3 at Satsop, Washington. All of these power reactors are currently operating, with the exception of Washington Nuclear Plant 3, which was only partially constructed and which has now been canceled. It is included in this analysis because its seismic design basis was completed and accepted provisionally by NRC staff (NRC 1991a).

Results of the compilation are presented in Appendix C. As shown there the estimated mean annual probability of exceeding the safe shutdown earthquake of each western plant is greater than 1.0E-04/yr, with the single exception of the PVNGS, which is located in a low-seismic-hazard region. The average mean annual probability of exceeding the safe shutdown earthquake of each plant is 2.0E-04, which is twice the value of the reference probability to be used in determining the Frequency-Category-2 design basis ground motion.

3.1.2.2 Conservatism of the Frequency-Category-2 Reference Probability

As noted earlier, the use of NRC-accepted seismic design bases for nuclear power reactors as a benchmark for Frequency-Category-2 design basis ground motion is based on the premise that reactor design bases correspond to acceptable seismic risk levels. The seismic design bases of all nuclear power reactors operating in the United States have been reviewed extensively by NRC staff, using standardized review criteria, and all have been found to satisfy applicable regulatory requirements by NRC licensing boards. In addition, a substantial body of recently developed information indicates that these plants have adequate margins of safety against potential accidents and that they have acceptably safe seismic designs. In June 1991 the NRC requested that its nuclear power reactor licensees perform a plant-specific Individual Plant Examination of External Events (IPEEE) to identify vulnerabilities, if any, to earthquakes, fires, winds, floods, and nearby transportation and other-facility accidents (NRC 1991b). The IPEEE program corroborated the adequacy of the seismic design bases of the Nation's operating nuclear power reactors. For example, specific IPEEE findings for operating reactors in the western United States were as follows:

- In the IPEEE study of the Diablo Canyon Power Plant, Pacific Gas and Electric Company found that the mean core damage frequency due to external events is about 6.7E-05/yr (PG&E 1994) . The component of this risk due to earthquake-initiated accident scenarios was estimated to be 4.0E-05/yr.

- The PVNGS is located in Wintersburg, Arizona, and is operated by the Arizona Public Service Company (APS). The PVNGS site is in a region of low seismic hazard relative to most other regions of the western United States; the PVNGS horizontal design basis response spectrum is anchored at 0.25 g peak ground acceleration (APS 1988). Given the relatively low seismic hazard, APS successfully persuaded NRC staff to have the PVNGS review-level earthquake reduced from 0.5 g (NRC 1991b) to 0.3g. APS elected to conduct a seismic margins analysis for the IPEEE program, rather than a seismic risk assessment. The margins analysis found that at least one safe-shutdown path exists for a peak horizontal ground acceleration in excess of 0.3 g (APS 1995).

- The IPEEE study conducted by Southern California Edison (SCE 1995) for the San Onofre Nuclear Generating Station found that the mean core damage frequency due to external-event initiators is approximately 3.3E-05/yr. The component of this risk due to earthquake-initiated accident scenarios was estimated to be about 1.7E-05/yr.

- In the IPEEE study of the Washington Nuclear Plant 2, the Washington Public Power Supply System (WPPSS 1995) estimated that the mean core damage frequency due to external-event initiators is 2.1E-05/yr and that this risk is dominated by the seismic contribution.

The conservatism of 1.0E-04/yr as a target exceedance probability for the Category-2 design basis ground motion also is based on an assumption that repository design acceptance criteria will reduce the probability of a severe seismically initiated accident below the probability of the design basis ground motions by a "risk-reduction" factor that is comparable to or greater than the factor that is provided by the design acceptance criteria for power reactors. This assumption itself has two bases. The first basis is that the DOE intends to use design acceptance criteria that are the same as or comparable to those used in reactor designs. The DOE has evaluated the NRC standard review plans for the seismic design of nuclear power reactors and has determined that many of the acceptance criteria are applicable to the design of repository surface facilities (see Section 3.2). These facilities are anticipated to include the majority of SSCs important to safety. Acceptance criteria for underground facilities are detailed in Sections 3.3 and 3.4 of this report. The second basis is that a repository is inherently less hazardous and less vulnerable to seismic shaking (or fault displacement) than is an operating nuclear power reactor. As noted by the NRC in the Section-by-Section Analysis of Section 60.136, *Preclosure Controlled Area*, in the Supplementary Information published with the final rule for 10 CFR 60 (61 FR 64257):

> ". . . in comparison with a nuclear power plant, an operating repository is a relatively simple facility in which the primary activities are in relation to waste receipt, handling, storage, and emplacement. A repository does not require the variety and complexity of systems necessary to support an operating nuclear power plant. Further, the conditions are not present at a repository to generate a radioactive source term of a magnitude that, however unlikely, is potentially capable at a nuclear power plant (e.g., from a postulated loss of coolant event). As such, the estimated consequences resulting from limited source term generation at a repository would be correspondingly limited."

In summary, use of a mean annual probability of exceedance of 1.0E-04 as a reference probability for the Frequency-Category-2 vibratory ground motion is quite conservative. This probability is comparable to the probabilities of exceeding the accepted seismic design bases of more recently designed operating nuclear power reactors in the CEUS. A compilation of the mean annual exceedance probabilities of the safe shutdown earthquakes of nuclear power reactors in the western United States indicates that the average mean exceedance probability for this set of reactors exceeds 1.0E-04 by about a factor of two. The DOE considers that use of this value for the preclosure seismic design of the geologic repository operations area is very conservative, given that a repository is inherently less hazardous and less vulnerable to seismic shaking than is an operating nuclear power reactor. The seismic safety of the operating power reactors and, by extension, the adequacy of their seismic design bases, has been confirmed by in-depth, site-specific analyses conducted under the IPEEE program.

3.1.3 Use of Reference Probabilities in Establishing Design Response Spectra

The DOE intends to establish design response spectra that correspond to the Frequency-Category-1 and -2 reference probabilities in a manner similar to that described in Regulatory Guide 1.165 (NRC 1997). This is done by first disaggregating the hazard results to identify the magnitudes and distances of earthquakes that control the hazard at frequencies of engineering interest. Controlling earthquakes will be identified for both of the reference mean annual exceedance probabilities, 1.0E-03 (Frequency Category 1) and 1.0E-04 (Frequency Category 2). Site-specific response spectra will be developed for these controlling earthquakes and will be scaled by the hazard at the reference probability level, at one or more specified frequencies. Finally, smooth design response spectra will be developed that envelope the controlling-earthquake response spectra and that provide sufficient energy over the frequency range of significance to repository SSCs. The details of this process will be developed as part of the development of the repository seismic design and will be fully described in the third seismic topical report.

3.1.4 Use of Reference Probabilities for Other Types of Events

The 10 CFR 60.2 defines Category 1 design basis events as "those natural and human-induced events that are reasonably likely to occur regularly, moderately frequently, or one or more times before permanent closure of the geologic repository operations area," and Category 2 design basis events as "other natural and man-induced events that are considered unlikely, but sufficiently credible to warrant consideration, taking into account the potential for significant radiological impacts on public health and safety." The DOE interprets the frequencies of Frequency Category 1 events (using the DOE's terminology) to be one every 100 years for infrastructure systems (ventilation, surface facilities, etc.) and one every 150 years for ground support systems; events with frequencies less than these values but greater than one every million years are interpreted to be Frequency Category 2 events. This interpretation is consistent with the NRC's statement (61 FR 64257) that the upper probability bound for Category 2 design basis events is roughly 1.0E-02 per year and the lower bound is on the order of 1.0E-06 per year. To ensure conservatism and consistency in the preclosure repository seismic design, the DOE has adopted lower probability levels for design basis seismic loads, as noted above (i.e., annual probabilities of 1.0E-03 and 1.0E-04 for Frequency-Category-1 and -2 vibratory ground motions, respectively, and 1.0E-04 and 1.0E-05 for Frequency-Category-1 and -2 fault displacements, respectively).

The reference probabilities proposed here for seismic loads are not intended to be applicable to other types of design basis external events such as severe winds, fires, or floods, or to design basis internal events. The probabilities for seismic loads are based on professional practice in seismic design, engineering judgment, and industry-wide experience in the licensing of nuclear power reactor seismic designs. Other criteria can be expected to apply to other types of design basis events, considering the degree of uncertainty in characterizing the frequency and severity of events; the potential consequences of exceeding design basis events; the incremental cost of increasing the basis for design; the methodology used to identify the design basis events; and established standards, codes, guidelines, and professional practices.

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ESSOW No. 05996.02-G010 (Rev. 0)
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January 31, 2001

ENGINEERING SERVICES SCOPE OF WORK

FOR

LABORATORY TESTING OF SOIL-CEMENT MIXES

Private Fuel Storage Facility
Skull Valley, Utah
Private Fuel Storage, LLC

Applied Geotechnical Engineering Consultants, Inc. (AGEC)

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QA CATEGORY I
IMPORTANT TO SAFETY

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1.0 SCOPE OF WORK – GENERAL

This Engineering Services Scope of Work (ESSOW) provides the technical and quality assurance requirements for laboratory testing of soils obtained from the Private Fuel Storage Facility (PFSF) site, which is located in Skull Valley, about 50 miles southwest of Salt Lake City, UT. Based on previous subsurface investigations performed at this site, the soils to be tested are expected to consist of eolian silt.

The purpose of these tests is to obtain information relevant to the appropriate soil-cement design for replacement of soils in the pad emplacement area and to provide information needed for construction of this facility. Samples will be obtained by others and transported to the laboratory for testing. Gradations will be performed on each sample obtained in order to determine the variability of grain size distribution over the site both horizontally and vertically. Atterberg limits shall be performed on samples exhibiting plasticity. A minimum of three chosen percentages of cement shall be incorporated into the samples and testing procedures for soil-cement durability will be performed. Moisture-density, freeze-thaw, wet-dry, compressive strength, tensile strength, and permeability tests will also be performed on selected samples. Procedures for performance of the required tests are referenced to ASTM standards and the Portland Cement Association, Soil-Cement Laboratory Handbook. The sequence of sample testing will be determined by the Engineers. The investigation will cover a large area and samples recovered from the area where construction will begin will be given priority. It is desired to perform the soil-cement durability tests on those samples exhibiting the largest component of fines and highest level of plasticity.

The Engineers will specify the testing process, including percentages of cement to be tested. The cement contents investigated will depend on the type of soil being tested. It is expected that the material to be tested will be eolian silt; however, some of these soils may be lacustrine clayey silt/silty clay. The expected cement contents to be used in the testing process are 6, 9, and 12%. Specimens shall be molded at each cement content for use in the wet-dry test and the freeze-thaw tests. Additional soil-cement specimens will be required for compressive strength, tensile strength, and permeability testing.

The laboratory facilities, equipment, testing, and calibration procedures shall comply with the requirements of NRC Regulatory Guide 1.138, as well as the applicable portions of the regulatory requirements, codes, and standards identified below.

The entire laboratory testing program will be conducted in full compliance with the Quality Assurance (QA) Category I requirements of this ESSOW.

2.0 APPLICABLE DOCUMENTS

All work shall be performed in accordance with the latest version of the following regulatory requirements, codes, and standards:

If there is, or seems to be, a conflict between this ESSOW and a referenced document, the matter shall be referred to the Engineers.

<u>US NRC</u>

10CFR21	Reporting of Defects and Noncompliance
10CFR50, App. B	Quality Assurance Criteria for Nuclear Power Plants and Fuel Reprocessing Plants
10CFR72	Licensing Requirements for the Independent Storage of Spent

		Nuclear Fuel and High-level Radioactive Waste
Regulatory Guide 1.138 For Comment, April 1978		Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants

American Society for Testing and Materials

ASTM C496	1996	Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens
ASTM D421	1985, R(1998)	Standard Practice for Dry Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants
ASTM D422	1963, R(1998)	Standard Test Method for Particle-Size Analysis of Soils
ASTM D558	1996	Standard Test Method for Moisture-Density Relations of Soil-Cement Mixtures
ASTM D559	1996	Standard Test Methods for Wetting and Drying Compacted Soil-Cement Mixtures
ASTM D560	1996	Standard Test Methods for Freezing and Thawing Compacted Soil-Cement Mixtures
ASTM D854	2000	Standard Test Methods for Specific Gravity of Soil Solids by Water Pycnometer
ASTM D1140	2000	Standard Test Method for Amount of Material in Soils Finer Than the No. 200 (75-Micrometer) Sieve
ASTM D1633	2000	Standard Test Methods for Compressive Strength of Molded Soil-Cement Cylinders
ASTM D2216	1998	Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
ASTM D2217	1985, R(1998)	Standard Practice for Wet Preparation of Soil Samples for Particle-Size Analysis and Determination of Soil Constants
ASTM D2487	2000	Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)
ASTM D2488	2000	Standard Practice for Description and Identification of Soils (Visual-Manual Procedure)
ASTM D3740	1999	Standard Practice for Minimum Requirements for Agencies Engaged in the Testing and/or Inspection of Soil and Rock as Used in Engineering Design and Construction
ASTM D4318	2000	Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
ASTM D5084	1990 R(1997)	Standard Test Method for Measurement of Hydraulic Conductivity of Saturated Porous Materials Using a Flexible Wall Permeameter

Soil-Cement Laboratory Handbook (1971), Portland Cement Association, Old Orchard Road, Skokie, Illinois 60076.

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2.1 Definitions

Terms used herein are defined as follows:

Approved	-	This word, when applied by the Engineers to the Contractor's drawings or documents, means that the drawings or documents are satisfactory in that the Engineers have not observed any statement or feature that appears to deviate from the requirements. The Contractor shall retain the entire responsibility for complete conformance with all of the requirements.
Approved as Revised	-	These words, when applied by the Engineers to the Contractor's documents, mean that the drawings or documents are approved as defined above except that the changes shown are necessary to be in conformance with the requirements. On the basis that the Contractor shall retain the entire responsibility for compliance with all of the requirements, the Contractor shall either: a. Incorporate the changes into its document and resubmit it to the Engineers, or b. Inform the Engineers that the changes cannot be made without prejudice to the Contractor's responsibility under warranty and resubmit with full explanation of the reasons therefor.
Contractor	-	The company accepting the overall responsibility for fulfilling requirements of this ESSOW.
Engineers	-	Stone & Webster, Inc. (S&W).
ESSOW	-	Engineering Services Scope of Work
Owner	-	Private Fuel Storage, LLC
Performance Audit	-	An activity to determine through investigation the adequacy of and adherence to established procedures, instructions, codes, and other applicable contractual and licensing requirements and the effectiveness of implementation.
Purchaser	-	Stone & Webster, Inc. (S&W).

3.0 REQUIREMENTS

3.1 Laboratory Testing Services - General

The samples to be tested will be delivered to the laboratory by others. All samples will have been marked in the field to indicate the project, test pit designations, depth of sampling, and date of retrieval. Tests requested by the Engineers shall be performed according to the procedures listed below and in compliance with the requirements of US NRC Regulatory Guide 1.138.

3.2 Test Procedures

3.2.1 General

All tests shall be performed according to the Contractor's written procedures, approved by the Engineers. Modifications of the approved procedures will be permitted as approved by the Engineers for special testing purposes and to secure the most satisfactory test results for each type of soil material. Visual-manual description of all soil samples tested shall be performed in accordance with ASTM D2488. For samples where the necessary test data are available, classifications shall be performed in accordance with ASTM 2487. All soil descriptions shall include the Unified Soil Classification System letter designations, in accordance with ASTM 2487.

3.2.2 Water Content

Water content shall be determined in accordance with ASTM D2216. Specimens shall be oven-dried for a period of at least 15 hours at a temperature of 110 degrees C (+/- 5 degrees), unless initial tests indicate that a shorter drying period will yield a constant dry weight.

3.2.3 Liquid Limit

Liquid limits shall be measured in accordance with ASTM D4318. The one-point liquid limit method shall be satisfactory for most classification purposes. The multipoint liquid limit method shall be used when directed by the Engineers.

3.2.4 Plastic Limit

The plastic limit shall be measured in accordance with ASTM D4318. The specimen used for the plastic limit determination shall be taken from the liquid limit specimen.

3.2.5 Sieve Analysis

The gradation of the soil sample shall be determined in accordance with ASTM D422, with the following clarifications:

a. If any particle sizes exceed 3 in., the maximum size particle of the sample shall be measured and reported.

b. The material passing the No. 10 sieve shall be washed over a No. 200 sieve in accordance with ASTM D1140 to determine the amount of material passing the No. 200 sieve. The material retained on the No. 200 sieve will be sieved through a nest of sieves containing as a minimum the following sizes: No. 40, No. 60, No. 100 and No. 200. This does not apply if a hydrometer test will be performed on the soil sample.

c. The report shall include a graph plotting the diameters of the particles in mm on a logarithmic scale as the abscissa and the percentages by weight of the total sample smaller than the corresponding diameters on an arithmetic scale as the ordinate.

d. The report shall include a description of the soil sample, prepared in accordance with ASTM D2488 and the Unified Soil Classification Symbol in accordance with ASTM D2487.

3.2.6 Hydrometer Analysis

Hydrometer analysis shall be performed in accordance with ASTM D422 to determine the distribution of soil particle sizes smaller than 75 microns. Calculations shall be made as described in ASTM D422 Sections 12 through 15.

The report shall include the specific gravity, any difficulty in dispersing the soil, the dispersion device used, and a graph of the test results. The graph shall be made by plotting the diameters of the particles on a logarithmic scale as the abscissa and the percentages by weight of the total sample smaller than the corresponding diameters to an arithmetic scale as the ordinate.

3.2.7 Moisture-Density Test

Moisture-density tests shall be performed in accordance with ASTM D 558 for each cement content selected for testing. Refer to the Portland Cement Association, Soil-Cement Laboratory Handbook for recommended procedures.

The moisture-density test for a soil-cement mixture determines the relationship between the moisture content of the soil-cement mixture and the resulting density when the mixture is compacted before cement hydration using a standard compactive force. This test is used to determine the optimum moisture content and maximum density for molding laboratory test specimens and it is used during construction to determine the quantity of water to be added and the density to which the mixture should be compacted in the field.

After determination of the optimum moisture content and maximum density of the soil-cement mixture at a specified cement content, specimens shall be molded at different cement contents for performance of wet-dry and freeze-thaw tests. These tests will determine the minimum amount of cement required to produce a durable soil cement. The test specimens shall be molded at the optimum moisture content determined from the moisture-density test using the same compaction equipment. Samples prepared at various moisture contents, either plus or minus the optimum moisture content, may also be required.

3.2.8 Wet-dry Test

Wet-dry tests shall be performed on specimens prepared at optimum moisture content in accordance with ASTM D559. Note, two specimens are required for each test – Specimen No. 1 is used to obtain data on moisture and volume changes during the test and Specimen No. 2 is used to obtain data on soil-cement losses during the test.

Briefly, the wet-dry tests are performed in the following manner. At the end of a 7-day specimen storage period in an atmosphere of high humidity, the specimens are submerged in tap water at room temperature for a period of 5 hours and then removed. The specimens are then placed in an oven at 71 deg. C. (160 deg. F.) for 42 hours and removed. Specimen No. 1 is weighed and measured to determine moisture content and volume changes during the test. Weight determinations of Specimen No. 2 shall be made as well, before and after brushing. Specimen No. 2 is given two firm strokes on all areas with a wire scratch brush to remove all material loosened during the wetting and drying cycles. These strokes are applied to the full height and width of the specimen with a firm stroke corresponding to approximately 3-lb force. Approximately 18 to 20 vertical brush strokes are required to cover the sides of the specimen twice and 4 strokes are required on each end. This procedure constitutes one cycle (48 hours) of wetting and drying. The specimens are then submerged in water again and the wetting-drying cycles are continued for 12 cycles. If it is not possible to run the cycles continuously, for example, because of Sundays or holidays, the specimens shall be held in the oven during the layover period. After 12 cycles of tests, the specimens are dried to constant weight at 110 deg. C (230 deg. F.) and weighed to determine their oven-dry weights. The soil-cement loss of the specimen shall then be calculated.

3.2.9 Freeze-thaw test

Freeze-thaw tests shall be performed on specimens prepared at optimum moisture content in accordance with ASTM D560. Note, two specimens are required for each test – Specimen No. 1 is used to obtain data on moisture and volume changes during the test and Specimen No. 2 is used to obtain data on soil-cement losses during the test.

Briefly, the freeze-thaw tests are performed in the following manner. At the end of the 7-day storage period in an atmosphere of high humidity, water-saturated felt pads about 0.25 to 0.50 inches thick, blotters, or similar absorptive material are placed between the specimens and the specimen carriers and the assembly is placed in a refrigerator with a constant temperature of not more than –23 deg. C (-10 deg. F.) for 24 hours and then removed. The assembly is then placed to thaw in the moist room or in suitable covered containers with a temperature of 21 deg. C. (70 deg. F.) and a relative humidity of 100 percent for 23 hours and then removed. Free water shall be made available to the absorbent pads to permit the specimens to absorb water by capillary action during the thawing period. Specimen No. 1 is weighed and measured to determine moisture content and volume changes during the test. Weight determinations of Specimen No. 2 shall be made as well, before and after brushing. Specimen No. 2 is then brushed in the same manner as described for the wet-dry test.

After being brushed at the end of each thawing period, the specimens are turned over, end for end, before they are replaced on the water-saturated pads. This procedure constitutes one cycle (48 hours) of freezing and thawing. The specimens are then replaced in the refrigerator and the freezing-thawing cycles are continued for 12 cycles. If it is not possible to run the cycles continuously, the specimens shall be held in the freezing cabinet during the layover period. After 12 cycles, the specimens shall be dried to constant weight at 110 deg. C. (230 deg. F.) and weighed to determine their oven-dry weights. The approximate soil-cement loss of freeze-thaw test specimens shall then be calculated.

3.2.10 Compressive-strength Test

Compressive-strength tests shall be performed on soil-cement samples in accordance with ASTM D1633. Four-inch diameter samples are required. Compressive-strength specimens shall be molded in accordance with Method ASTM D559 at cement contents specified by the Engineers and stored at room temperature in an atmosphere of approximately 100 percent humidity until testing. The Engineers shall specify the length of time required for moist-curing the compression test specimens, which normally will be 7 or 28 days. At the end of the moist-cure time, the samples shall be soaked in water for four hours and shall then be broken in compression at a constant rate of application of load within the range of 20 ± 10 psi per second.

3.2.11 Permeability Test

Permeability tests shall be performed on soil-cement specimens in accordance with ASTM D5084. The type and number of soil-cement samples to be tested, as well as the cement contents of the specimens, will be specified by the Engineers.

3.2.12 Splitting Tensile Strength Test

Splitting tensile strength tests shall be performed on soil-cement specimens generally in accordance with the requirements of ASTM C496. The type and number of soil-cement samples to be tested, as well as the cement contents of the specimens, will be specified by the Engineers.

3.3 Presentation of Results

The Contractor shall submit formal results of the testing program in hardcopy and an electronic format approved by the Engineers. All graphs shall be at scales (engineering) that clearly and neatly present the data. If possible, all pages, graphs, and figures should be not greater than 11 in. in height, with folded pages no greater than 8.5 in. in width. In addition to the items required by the standards listed in Section 2.0, the results shall contain the following basic data:

a. Purchaser: Stone & Webster, Inc.

b. Project number: J.O. 05996.02

c. Project name: Private Fuel Storage Facility – Skull Valley, UT

d. Owner: Private Fuel Storage, LLC

e. A description of the methods used for each type of test.

f. Test pit designation, sample designation, depth, soil description (in accordance with ASTM D2487 and D2488), and percent cement content of each sample tested.

g. Where applicable, a table showing the range of values and average value for each measured or calculated parameter and the number of samples upon which these values are based.

h. A table summarizing all of the test results.

i. The person performing the test, the person preparing the presentation of the results, and the person checking the results shall sign or initial each sheet.

j. The final report shall be independently reviewed. The independent reviewer shall be a technically qualified individual other than:

- The preparer
- The immediate supervisor of the preparer
- Any individual who specified inputs, selected the approach, or ruled out any design considerations.

3.4 Sample Disposition

For each sample received by the Contractor, the remaining material from tested specimens shall be labeled with the project number, test pit number, and depth range of the original sample. All of the material shall be stored for a period of up to 90 days following the completion of testing. The Engineers will instruct the Contractor concerning either shipping or disposal of the samples during that period. An inventory list identifying the contents of the delivery shall be provided to the Engineers at the time of delivery for any samples shipped at the instruction of the Engineers.

4.0 QUALITY ASSURANCE

4.1 Quality Assurance Program Requirements

The Contractor shall have in effect a quality assurance program for the laboratory to ensure that the laboratory meets the requirements of this scope of work and federal regulations 10CFR50, Appendix B and 10CFR72, or as an alternative, shall conform to the Engineers' Quality Assurance Program. As a minimum, the program shall include: recording of samples received, stored, and final disposition; a laboratory equipment calibration schedule and file of results for each piece of equipment used in this program; and a geotechnical engineer assigned to review and inspect the testing to assure conformance with the laboratory's written procedures. If the Contractor elects to use his Quality Assurance Program, the quality assurance program shall be discussed with and approved by the Engineers prior to the start of the work.

If the Contractor elects to execute the work in accordance with the Engineer's Quality Assurance Program, a brief (1 to 2 hr) QA indoctrination session will be held at the laboratory prior to the start of work. If lab personnel are changed during the course of this work, the Contractor shall perform the QA indoctrination of such persons before they start work on this testing program.

The Engineers will have in effect an inspection, testing, and documentation program to ensure that the laboratory testing and equipment meet the requirements of this Scope of Work and federal regulations 10CFR50, Appendix B, and 10CFR72. The Quality Assurance Program implemented by the Engineers does not relieve the Contractor of his obligations to ensure the quality of his work. The Engineers shall have access at any reasonable time to all records pertaining to this Scope of Work for the purpose of inspections and performance audits.

The Contractor shall specifically ensure that a copy of this Scope of Work, with all addenda or appropriate work instructions, are readily available where work covered by this Scope of Work is in progress.

4.2 Written Procedures

The Contractor shall have written procedures for the calibration of the laboratory equipment to be used in this testing program, and shall have written procedures for the following tests:

a. Logging and classification of bulk samples

b. Moisture content

c. Atterberg limits

d. Sample preparation and analysis for grain size distribution, including percent fines passing the No. 200 sieve and hydrometer

e. Moisture-density of soil-cement specimens

f. Wet-dry testing of soil-cement specimens

g. Freeze-thaw testing of soil-cement specimens

h. Compressive strength

i. Permeability

j. Splitting tensile strength

No test shall be performed until the Contractor's written procedure for that test has been approved in writing by the Engineers. The equipment used for this testing program shall have been calibrated within the period specified by the Contractor's calibration procedure, which shall be in accordance with the requirements of the US NRC Regulatory Guide 1.138.

4.3 Qualified Personnel

The Contractor shall assign qualified personnel to perform, check, and review the laboratory tests and shall furnish records of qualifications of responsible project personnel to the Engineers. The activities to which these individuals are assigned shall be noted.

All test results shall be reviewed by a qualified individual other than the person who performed or checked plots and calculations for the test.

4.4 Engineers' Liaison

Authorized representatives of the Engineers shall be allowed access to the Contractor's offices and laboratories at all reasonable times to inspect the Contractor's or subcontractor's work, material, equipment, or inspection procedures, as applicable to the work covered by this Scope of Work, and to observe testing procedures and raw data in order to familiarize themselves with the soil conditions and test results. The Contractor shall cooperate with these representatives to assure complete documentation. The Engineers shall discuss with the Contractor anything they notice that may lead to rejection of the work.

It is not intended that the presence or activity of the Engineers shall relieve the Contractor in any way of his obligations under this Scope of Work. Furthermore, the fact that the Engineers may inadvertently overlook a deviation from some requirements of this Scope of Work shall not constitute a waiver of that requirement, nor of the Contractor's obligation to correct the condition when it is discovered, nor of any other obligation under this Scope of Work.

4.5 Performance Audit

Authorized representatives of the Owner and the Engineers shall be allowed access to the testing laboratory of the Contractor and any subcontractors at reasonable times for the purpose of performing audits and inspections. At least two (2) working days notice will be given prior to an audit. Such audits will be based on the technical and quality assurance requirements of this Scope of Work and will include examination of documentary evidence of activities affecting quality. Audits will be carried out on a planned basis during the course of work to verify compliance with this Scope of Work.

4.6 Documentation by Contractor

The Contractor shall specifically ensure that a copy of this Scope of Work, with all addenda and appropriate work instructions, are readily available where work covered by this Scope of Work is in progress.

The basic documentation required of the Contractor includes:

a. Written procedures of sample preparation and laboratory soils tests or a list of referenced standards used by the laboratory

b. Written calibration procedures and calibration intervals used by the laboratory

c. A list of equipment used in this testing program identifying the manufacturer's model number and the laboratory's unique identifier (i.e., serial number)

d. Written qualification statements for testing and review personnel actually used in performing the work

e. A log of samples received by the laboratory and the final disposition of each sample

f. Information reports, supplied every week, including progress of testing program and draft laboratory test results

g. Laboratory test report (draft and final), which shall be identified as Report No. 05996.02-G(POxx)-1, Rev. 0. (Note, replace "xx" in this identifier with the correct number of the purchase order issued for this ESSOW.) This report identifier shall be included on every page of the report and all pages shall be numbered in a logical fashion.

h. A file of test data, calibration data, calculations, inspections, communications, and other data documenting the work but not included in the final test report.

Each document submitted by the Contractor shall be clearly identified by the Purchaser's name and the project number (J.O. 05996.02).

5.0 SUPPLEMENTAL PROVISIONS

5.1 Conditions

5.1.1 Deviations and Nonconformances

No deviation or nonconformance from this ESSOW or applicable federal, state, and local codes and standards invoked by this ESSOW shall be accepted until approved by the Engineers. Deviations are considered departures from any requirement of this ESSOW. Uncorrectable nonconformances are considered to be conditions that cannot be corrected within the ESSOW requirements.

The Contractor shall promptly document and notify the Engineers of all deviations and nonconformances from the ESSOW (such as deviations from applicable codes or drawings). Further testing after detection of any deviation or nonconformance prior to the Engineers' approval shall be at the Contractor's risk. No changes to this ESSOW shall be binding on any party until an addendum or revision to the ESSOW is issued.

5.1.2 Compliance with 10CFR21

The services provided under this ESSOW are a basic component of an NRC-licensed facility or activity. Accordingly, the Contractor is subject to the provisions of Part 21, Chapter 1 of Title 10 of the Code of Federal Regulations.

5.2 Subcontractors

All subcontractors to be used by the Contractor shall be subject to approval by the Engineers. To the extent that they apply, the Contractor shall impose on each of his subcontractors, the complete requirements of this ESSOW. He shall be directly responsible to see that the subcontractors are completely aware of all these requirements and that they abide thereby.

5.3 Furnished by the Engineers

The Engineers will furnish the soil samples to be tested, as well as instructions on tests required.

5.4 Furnished by the Contractor

The Contractor shall furnish soil testing facilities, equipment, and personnel experienced in all laboratory testing procedures required herein.

The Contractor shall submit with his proposal experience records of his key laboratory personnel and a listing of companies for which he has recently performed similar work.

5.5 Schedule

On the premise that notification to proceed will be received by the Contractor not later than February 1, 2000, the laboratory work shall be completed and the draft laboratory testing report shall be delivered on or before March 30, 2001.

The final laboratory testing report shall be delivered on or before two weeks after receipt of comments from the Engineers on the draft report.

5.6 Measurement for Payment

5.6.1 Laboratory Soil Testing

The unit of measurement for laboratory testing shall be the individual test. The quantity to be paid for shall be the number of each type of test satisfactorily performed in accordance with this Scope of Work. The unit price for each test shall be full reimbursement for all labor, equipment, and supplies required to handle, store, prepare, and test samples, calculate and report test results, as necessary, and dispose of samples when directed by the Engineers. Storage, handling, and disposal of all samples that are non-hazardous and free of contamination, including those not tested, shall be considered incidental to the cost of the required laboratory testing. Shipping of samples at the end of the storage period will be paid for separately, if directed by the Engineers.

It is understood that the samples will be non-hazardous and free of contamination that may require an extra measure of care in handling, testing, storing, or disposal. Any leachate-damaged permeability equipment will be paid for at cost+15%. Any costs associated with contaminated sample disposal will be paid for at cost+15%.

ACI 230.1R-90

(Reapproved 1997)

State-of-the-Art Report on Soil Cement

reported by ACI Committee 230

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Soil cement is a densely compacted mixture of portland cement, soil/aggregate, and water. Used primarily as a base material for pavements, soil cement is also being used for slope protection, low-permeability liners, foundation stabilization, and other applications. This report contains information on applications, material properties, mix proportioning, construction, and quality-control inspection and testing procedures for soil cement. This report 's intent is to provide basic information on soil-cement technology with emphasis on current practice regarding design, testing, and construction.

Keywords: aggregates; base courses; central mixing plant; compacting; construction; fine aggregates; foundations; linings; mixing; mix proportioning; moisture content; pavements; portland cements; properties; slope protection; soil cement; soils; soil stabilization; soil tests; stabilization; tests; vibration.

CONTENTS

1-INTRODUCTION

1.1-Scope

This state-of-the-art report contains information on applications, materials, properties, mix proportioning, design, construction, and quality-control inspection and

Exhibit 23

testing procedures for soil cement. The intent of this report is to provide basic information on soil-cement technology with emphasis on current practice regarding mix proportioning, properties, testing, and construction.

This report does not provide information on fluid or plastic soil cement, which has a mortarlike consistency at time of mixing and placing. Information on this type of material is provided by ACI Committee 229 on Controlled Low-Strength Material (CLSM). Roller-compacted concrete (RCC), which is a type of no-slump concrete compacted by vibratory roller, is not covered in this report. ACI Committee 207 on Mass Concrete has a report available on roller-compacted concrete.

1.2-Definitions

Soil cement-ACI 116R defines soil cement as "a mixture of soil and measured amounts of portland cement and water compacted to a high density." Soil cement can be further defined as a material produced by blending, compacting, and curing a mixture of soil/aggregate, portland cement, possibly admixtures including pozzolans, and water to form a hardened material with specific engineering properties. The soil/aggregate particles are bonded by cement paste, but unlike concrete, the individual particle is not completely coated with cement paste.

Cement content-Cement content is normally expressed in percentage on a weight or volume basis. The cement content by weight is based on the oven-dry weight of soil according to the formula

$$C_w = \frac{\text{weight of cement}}{\text{Oven-dry weight of soil}} \times 100$$

The required cement content by weight can be converted to the equivalent cement content by bulk volume, based on a 94-lb U.S. bag of cement, which has a loose volume of approximately 1 ft^3, using the following formula'

$$C_v = \frac{D - \left[\dfrac{D}{1 + C_w/100}\right]}{94} \times 100$$

where

C_v = cement content, percent by bulk volume of compacted soil cement
D = oven-dry density of soil-cement in lb/ft^3
C_w = cement content, percent by weight of oven-dry soil

The criteria used to determine adequate cement factors for soil-cement construction were developed as a percentage of cement by volume in terms of a 94-lb U.S. bag of cement. The cement content by volume in terms of other bag weights, such as an 80-lb Canadian bag, can be determined by substituting 80 for 94 in the denominator of the preceding formula.

2-APPLICATIONS

2.1 -General

The primary use of soil cement is as a base material underlaying bituminous and concrete pavements. Other uses include slope protection for dams and embankments; liners for channels, reservoirs, and lagoons; and mass soil-cement placements for dikes and foundation stabilization.

2.2-Pavements

Since 1915, when a street in Sarasota, Fla. was constructed using a mixture of shells, sand, and portland cement mixed with a plow and compacted, soil cement has become one of the most widely used forms of soil stabilization for highways. More than 100,000 miles of equivalent 24 ft wide pavement using soil cement have been constructed to date. Soil cement is used mainly as a base for road, street, and airport paving. When used with a flexible pavement, a hot-mix bituminous wearing surface is normally placed on the soil-cement base. Under concrete pavements, soil cement is used as a base to prevent pumping of fine-grained subgrade soils under wet conditions and heavy truck traffic. Furthermore, a soil-cement base provides a uniform, strong support for the pavement, which will not consolidate under traffic and will provide increased load transfer at pavement joints. It also serves as a firm, stable working platform for construction equipment during concrete placement.

Failed flexible pavements have been recycled with cement, resulting in a new soil-cement base (Fig. 2.1). Recycling increases the strength of the base without removing the old existing base and subbase materials and replacing them with large quantities of expensive new base materials. In addition, existing grade lines and drainage can be maintained. If an old bituminous surface can be readily pulverized, it can be considered satisfactory for inclusion in the soil-cement mixture. If, on the other hand, the bituminous surface retains most of its original flexibility, it is normally removed rather than incorporated into the mixture.

The thickness of a soil-cement base depends on various factors, including: (1) subgrade strength, (2) pavement design period, (3) traffic and loading conditions, including volume and distribution of axle weights, and (4) thicknesss of concrete or bituminous wearing surface. The Portland Cement Association (PCA),[2,3] the American Association of State Highway and Transportation Officials (AASHTO),[4] and the U.S. Army Corps of Engineers (USACE),[5,6] have established methods for determining design thickness for soil-cement bases. Most in-service soil-cement bases are 6 in. thick. This thickness has proved satisfactory for service conditions associated with secondary roads, residential streets, and light-traffic air fields. A few 4 and 5 in. thick bases have given good service under favorable conditions of light traffic and strong subgrade support. Many miles of 7 and 8 in. thick soil-cement bases are providing good performance in primary and high-traffic secondary pavements. Although soil-cement bases more than

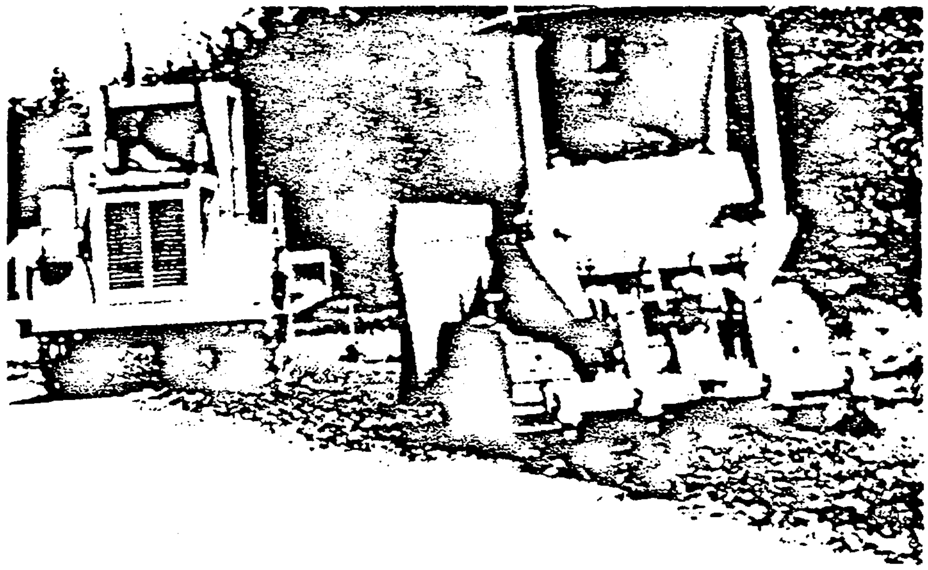

Fig.2.1-Old bituminous mat being scarified and pulverized for incorporation in soil-cement mix

9 in. thick are not common. a few airports and heavy industrial pavement project[3] have been built with multilayered thicknesses up to 32 in.

Since 1975, soil-cement base courses incorporating local soils with portland cement and fly ash have been constructed in 17 states.[7] Specification guidelines and a contractor's guide for constructing such base courses are available from the Electric Power Research Institute.[8]

2.3-Slope protection

Following World War II, there was a rapid expansion of water resource projects in the Great Plains and South Central regions of the U.S. Rock riprap of satisfactory quality for upstream slope protection was not locally available for many of these projects. High costs for transporting riprap from distant quarries to these sites threatened the economic feasibility of some projects. The U.S. Bureau of Reclamation (USBR) initiated a major research effort to study the suitability of soil cement as an alternative to conventional riprap. Based on laboratory studies that indicated soil cement made with sandy soils could produce a durable erosion-resistant facing, the USBR constructed a full-scale test section in 1951. A test-section location along the southeast shore of Bonny Reservoir in eastern Colorado was selected because of severe natural service conditions created by waves, ice, and more than 100 freeze-thaw cycles per year. After 10 years of observing the test section, the USBR was convinced of its suitability and specified soil cement in 1961 as an alternative to riprap for slope protection on Merritt Dam, Nebraska, and later at Cheney Dam, Kansas. Soil cement was bid at less than 50 percent of the cost of riprap and produced a total savings of more than $1 million for the two projects.

Performance of these early projects has been good. Although some repairs have been required for both Merritt and Cheney Dams, the cost of the repairs was far less than the cost savings realized by using soil cement over riprap. In addition, the repair costs may have been less than if riprap had been used.[9] The original test section at Bonny Reservoir has required very little maintenance and still exists today, almost 40 years later (Fig. 2.2).

Since 1961, more than 300 major soil-cement slope protection projects have been built in the U.S. and Canada. In addition to upstream facing of dams, soil cement has provided slope protection for channels, spillways, coastal shorelines, highway and railroad embankments, and embankments for inland reservoirs.

For slopes exposed to moderate to severe wave action (effective fetch greater than 1000 ft) or debris-carrying, rapid-flowing water, the soil cement is usually placed in successive horizontal layers 6 to 9 ft wide by 6 to 9 in. thick, adjacent to the slope. This is referred to as "stairstep slope protection" (Fig. 2.3). For less severe applications, like those associated with small reservoirs, ditches, and lagoons, the slope protection may consist of a 6 to 9 in. thick layer of soil cement placed parallel to the slope face. This method is often referred to as "plating" (Fig. 2.4).

The largest soil-cement project worldwide involved 1.2 million yd^3 of soil-cement slope protection for a

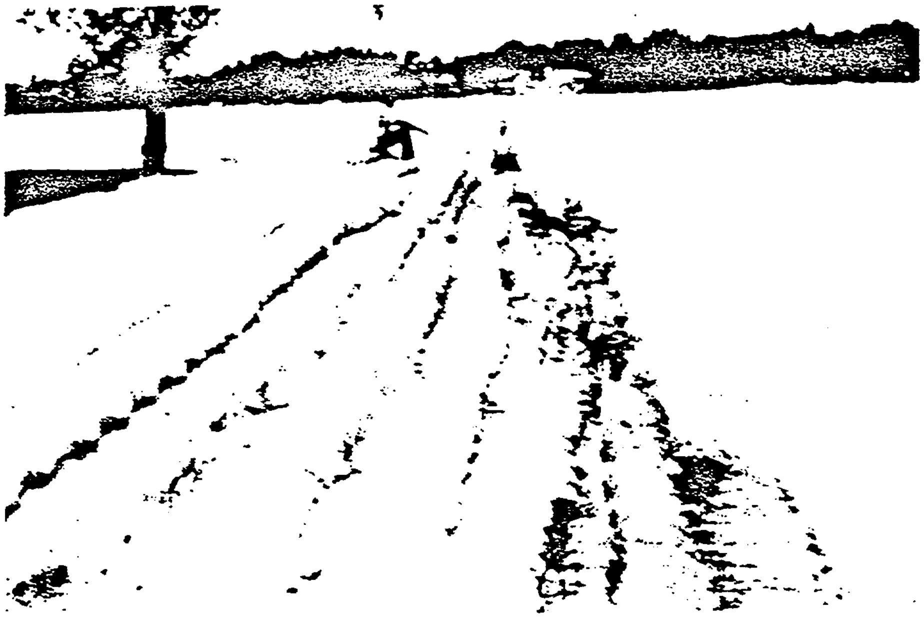

Fig. 2.2-Soil-cement test section at Bonny Reservoir, Colo., after 34 years

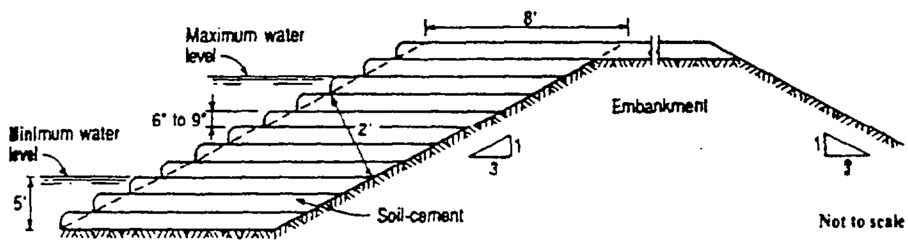

Fig. 2.3-Soil-cement slope protection showing layered design

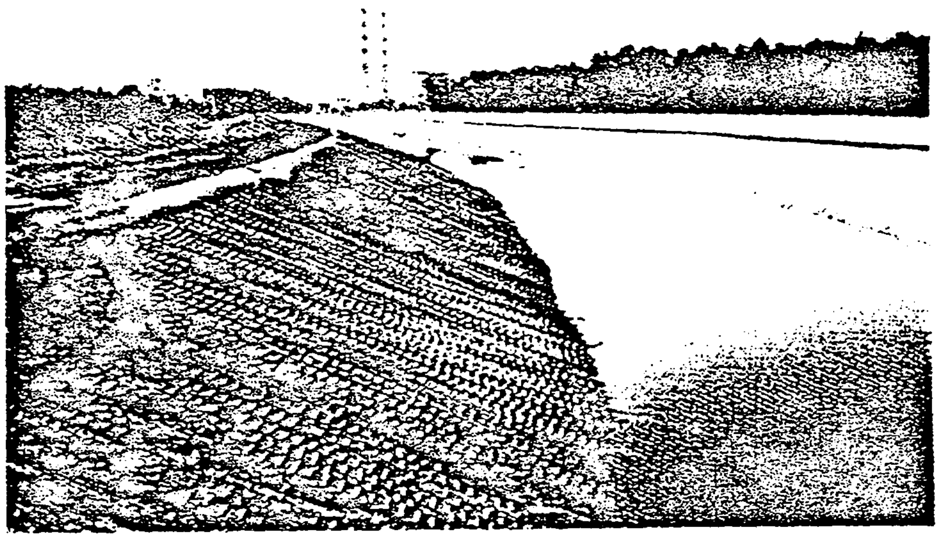

Fig. 2.4-Soil-cement slope plating for cooling water flume at Florida power plant

7000-acre cooling-water reservoir at the South Texas Nuclear Power Plant near Houston. Completed in 1979, the 39 to 52 ft high embankment was designed to contain a 15 ft high wave action that would be created by hurricane winds of up to 155 mph. In addition to the 13 miles of exterior embankment, nearly 7 miles of interior dikes, averaging 27 ft in height, guide the recirculating cooling water in the reservoir. To appreciate the size of this project, if each 6.75 ft wide by 9 in. thick lift were placed end-to-end rather than in stair-step fashion up the embankment, the total distance covered would be over 1200 miles.

Soil cement has been successfully used as slope protection for channels and streambanks exposed to lateral flows. In Tucson, Arizona, for example, occasional flooding can cause erosion along the normally dry river beds. From 1983 to 1988, over 50 soil-cement slope protection projects were constructed in this area. A typical section consists of 7 to 9 ft wide horizontal layers placed in stairstep fashion along 2:1 (horizontal to vertical) embankment slopes. To prevent scouring and subsequent undermining of the soil cement, the first layer or two is often placed up to 8 ft below the existing dry river bottom, and the ends extend approximately 50 ft into the embankment. The exposed slope facing is generally trimmed smooth during construction for appearance. To withstand the abrasive force of stormwater flows of 25,000 to 45,000 ft^3/sec at velocities up to 20 ft/sec, the soil cement is designed for a minimum 7-day compressive strength of 750 psi. In addition, the cement content is increased by two percentage points to allow for field variations.[10]

More detailed design information on soil-cement slope protection can be found in References 11 through 13.

2.4-Liners

Soil cement has served as a low-permeability lining material for over 30 years. During the mid-1950s, a number of 1 to 2 acre farm reservoirs in southern California were lined with 4 to 6 in. thick soil cement. One of the largest soil-cement-lined projects is Lake Cahuilla, a terminal-regulating reservoir for the Coachella Valley County Water District irrigation system in southern California. Completed in 1969, the 135 acre reservoir bottom has a 6 in. thick soil-cement lining, and the sand embankments forming the reservoir are faced with 2 ft of soil cement normal to the slope.

In addition to water-storage reservoirs, soil cement has been used to line wastewater-treatment lagoons, sludge-drying beds, ash-settling ponds, and solid waste landfills. The U.S. Environmental Protection Agency (EPA) sponsored laboratory tests to evaluate the compatibility of a number of lining materials exposed to various wastes.[14] The tests indicated that after 1 year of exposure to leachate from municipal solid wastes, the soil cement hardened considerably and cored like portland cement concrete. In addition, it became less permeable during the exposure period. The soil cement was also exposed to various hazardous wastes, including toxic pesticide formulations, oil refinery sludges, toxic pharmaceutical wastes, and rubber and plastic wastes. Results showed that for these hazardous wastes, no seepage had occurred through soil cement following 2½ years of exposure. After 625 days of exposure to these wastes, the compressive strength of the soil cement exceeded the compressive strength of similar soil cement that had not been exposed to the wastes. Soil cement was not exposed to acid wastes. It was rated "fair" in containing caustic petroleum sludges, indicating that the specific combination of soil cement and certain waste materials should be tested and evaluated for compatibility prior to final design decision.

Mix proportions for liner applications have been tested in which fly ash replaces soil in the soil-cement mixture. The fly ash-cement mixture contains 3 to 6 percent portland cement and 2 to 3 percent lime. Permeabilities significantly less than 1×10^{-7} cm/sec have been measured for such fly ash-lime-cement mixtures, along with unconfined compressive strengths before and after vacuum saturation, which indicate good freeze-thaw durability.[15] A similar evaluation has been made for liners incorporating fly ash, cement, and bentonite.[16]

For hazardous wastes and other impoundments where maximum seepage protection is required, a composite liner consisting of soil cement and a synthetic membrane can be used. To demonstrate the construction feasibility of the composite liner, a test section was built in 1983 near Apalachin, N.Y. (Fig. 2.5). The section consisted of a 30 and 40 mil high-density polyethylene (HDPE) membrane placed between two 6-in. layers of soil cement. After compacting the soil-cement cover layer, the membrane was inspected for signs of damage. The membrane proved to be puncture-resistant to the placement and compaction of soil cement even with ¾-in. aggregate scattered beneath the membrane.[17]

2.5-Foundation stabilization

Soil cement has been used as a massive fill to provide foundation strength and uniform support under large structures. In Koeberg, South Africa, for example, soil cement was used to replace an approximately 18 ft thick layer of medium-dense, liquifiable saturated sand under two 900-MW nuclear power plants. An extensive laboratory testing program was conducted to determine static and dynamic design characteristics, liquefaction potential, and durability of the soil cement. Results showed that with only 5 percent cement content by dry weight, cohesion increased significantly, and it was possible to obtain a material with enough strength to prevent liquefaction.[18]

Soil cement was used in lieu of a pile or caisson foundation for a 38-story office building completed in 1980 in Tampa, Fla. A soft limestone layer containing several cavities immediately below the building made the installation of piles or caissons difficult and costly. The alternative to driven foundation supports was to excavate the soil beneath the building to the top of

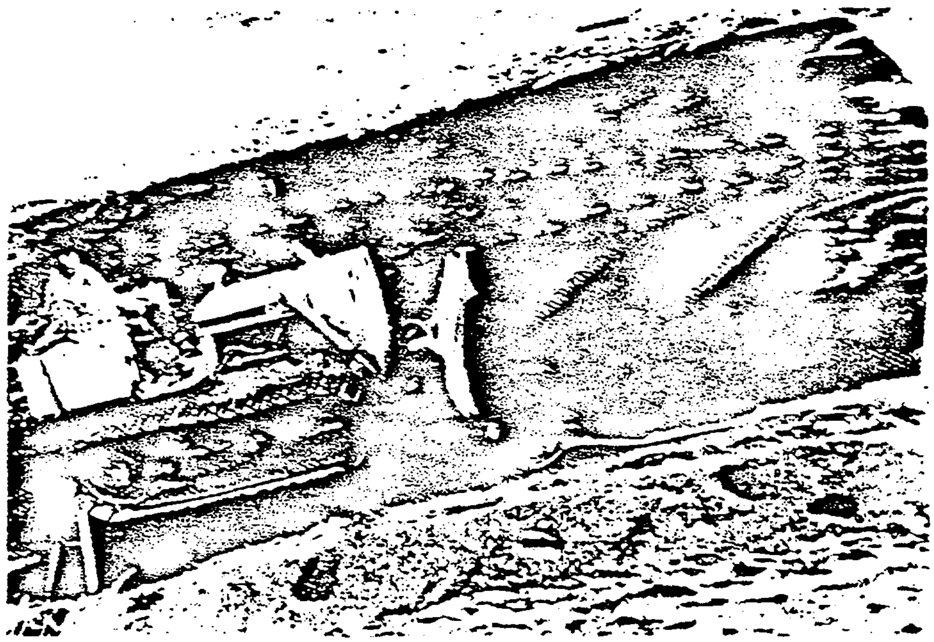

Fig. 2.5-Spreading soil cement on membrane at 3:1 slope, Apalachin, N.Y.

limestone. The cavities within the limestone were filled with lean concrete to provide a uniform surface prior to soil-cement placement. The excavated fine sand was then mixed with cement and returned to the excavation in compacted layers. The 12 ft thick soil cement mat saved $400,000 as compared to either a pile or caisson foundation. In addition to providing the necessary bearing support for the building, the soil cement doubled as a support for the sheeting required to stabilize the excavation's walls. The soil cement was ramped up against the sheeting and cut back vertically to act as formwork for the mat pour. As a result, just one brace was needed for sheeting rather than eight.[19]

At the Cochiti Dam site in north-central New Mexico, a 35 ft deep pocket of low-strength clayey shale under a portion of the outlet works conduit was replaced with 57,650 yd^3 of soil cement. The intent of the massive soil-cement placement was to provide a material with physical properties similar to the surrounding sandstone, thereby minimizing the danger of differential settlement along the length of the conduit. Unconfined 28-day compressive strengths for the soil cement were just over 1000 psi, closely approximating the average unconfined compressive strength of representative sandstone core samples.

In 1984, soil cement was used instead of mass concrete for a 1200 ft wide spillway foundation mat at Richland Creek Dam near Ft. Worth, Tex. About 10 ft of overburden above a solid rock strata was removed and replaced with 117,500 yd^3 of soil cement. To satisfy the 28-day 1000 psi compressive strength criteria, 10 percent cement content was used. The substitution of soil cement for mass concrete saved approximately $7.9 million.

2.6-Miscellaneous applications

Rammed earth is another name for soil cement used to construct wall systems for residential housing. Rammed-earth walls, which are generally 2 ft thick, are constructed by placing the damp soil cement into forms commonly made of plywood held together by a system of clamps and whalers. The soil cement is then compacted in 4 to 6 in. thick lifts with a pneumatic tamper. After the forms are removed, the wall can be stuccoed or painted to look like any other house. Rammed-earth homes provide excellent thermal mass insulation properties; however, the cost of this type of construction can be greater than comparably equipped frame houses. A typical rammed-earth soil mix consists of 70 percent sand and 30 percent noncohesive fine-grained soil. Cement contents vary from 4 to 15 percent by weight with the average around 7 percent.[20]

Soil cement has been used as stabilized backfill. At the Dallas Central Wastewater Treatment Plant, soil cement was used as economical backfill material to correct an operational problem for 12 large clarifiers. The clarifiers are square tanks but utilize circular sweeps. Sludge settles in the corners beyond the reach of the sweep, resulting in excessive downtime for maintenance. To operate more efficiently, sloped fillets of soil cement were constructed in horizontal layers to round out the four corners of each tank. A layer of shotcrete was placed over the soil-cement face to serve as a protective wearing surface.

Recently, the Texas State Department of Highways and Public Transportation has specified on several projects that the fill behind retained earth-wall systems be cement-stabilized sand. This was done primarily as a precautionary measure to prevent erosion from behind the wall and/or under the adjacent roadway.

At some locations, especially where clay is not available, embankments and dams have been constructed entirely of soil cement. A monolithic soil-cement embankment serves several purposes. It provides slope protection, acts as an impervious core, and can be built on relatively steep slopes due to its inherent shear strength properties. A monolithic soil-cement embankment was used to form the 1 100-acre cooling water reservoir for Barney M. Davis Power Plant near Corpus Christi, Tex. The reservoir consisted of 6.5 miles of circumferential embankment and 2.1 miles of interior baffle dikes. The only locally available material for construction was a uniformly graded beach sand. The monolithic soil-cement design provided both slope protection and served as the impervious core. By utilizing the increased shear strength properties of the compacted cement-stabilized beach sand, the 8 to 22 ft high embankment was constructed at a relatively steep slope of 1.5H:1V.

Coal-handling and storage facilities have used soil cement in a variety of applications. The Sarpy Creek coal mine, near Hardin, Mont., utilized soil cement in the construction of a coal storage slot. Slot storage basically consists of a long V-shaped trough with a reclaim conveyor at the bottom of the trough. The trough sidewalls must be at a steep and smooth enough slope to allow the stored coal to remain in a constant state of gravity flow. The Sarpy Creek storage trough is 750 ft long and 20 ft deep. The 15,500 yd^3 of soil cement were constructed in horizontal layers 22 ft wide at the bottom to 7 ft wide at the top. During construction, the outer soil-cement edges were trimmed to a finished side slope of 50 deg. A shotcrete liner was placed over the soil cement to provide a smooth, highly wear-resistant surface.

Monolithic soil cement and soil-cement-faced berms have been used to retain coal in stacker-reclaimer operations. The berm at the Council Bluffs Power Station in southwestern Iowa is 840 ft long by 36 ft high and has steep 55 deg side slopes. It was constructed entirely of soil cement with the interior zone of the berm containing 3 percent cement. To minimize erosion to the exposed soil cement, the 3.3 ft thick exterior zone was stabilized with 6 percent cement.

At the Louisa Power Plant near Muscatine, Iowa, only the exterior face of the coal-retaining berm was stabilized with soil cement. The 4 ft thick soil cement and interior uncemented sand fill were constructed together in 9 in. thick horizontal lifts. A modified asphalt paving machine was used to place the soil cement. A smooth exposed surface was obtained by trailing plates at a 55-deg angle against the edge during individual lift construction.

Several coal-pile storage yards have been constructed of soil cement. Ninety-five acres of coal storage yard were stabilized with 12 in. of soil cement at the Independence Steam Electric Station near Newark, Ark., in 1983. The soil consisted of a processed, crushed limestone aggregate. The 12 in. thick layer was placed in two 6 in. compacted lifts. By stabilizing the area with soil cement, the owner was able to eliminate the bedding layer of coal, resulting in an estimated savings of $3 million. Other advantages cited by the utility include almost 100 percent coal recovery, a defined perimeter for its coal pile, reduced fire hazard, and all-weather access to the area for service and operating equipment.

3-MATERIALS

3.1-Soil

Almost all types of soils can be used for soil cement. Some exceptions include organic soils, highly plastic clays, and poorly reacting sandy soils. Tests including ASTM D 4318 are available to identify these problem materials.[21,22] Section 5.3 of this report, which focuses on special design considerations, discusses the subject of poorly reacting sandy soils in more detail. Granular soils are preferred. They pulverize and mix more easily than fine-grained soils and result in more economical soil cement because they require the least amount of cement. Typically, soils containing between 5 and 35 percent fines passing a No. 200 sieve produce the most economical soil cement. However, some soils having higher fines content (material passing No. 200 sieve) and low-plasticity have been successfully and economically stabilized. Soils containing more than 2 percent organic material are usually considered unacceptable for stabilization. Types of soil typically used include silty sand, processed crushed or uncrushed sand and gravel, and crushed stone.

Aggregate gradation requirements are not as restrictive as conventional concrete. Normally the maximum nominal size aggregate is limited to 2 in. with at least 55 percent passing the No. 4 sieve. For unsurfaced soil cement exposed to moderate erosive forces, such as slope-protection applications, studies by Nussbaum[23] have shown improved performance where the soil contains at least 20 percent coarse aggregate (granular material retained on a No. 4 sieve).

Fine-grained soils generally require more cement for satisfactory hardening and, in the case of clays, are usually more difficult to pulverize for proper mixing. In addition, clay balls (nodules of clay and silt intermixed with granular soil) do not break down during normal mixing. Clay balls have a tendency to form when the plasticity index is greater than 8. For pavements and other applications not directly exposed to the environment, the presence of occasional clay balls may not be detrimental to performance. For slope protection or other applications where soil cement is exposed to weathering, the clay balls tend to wash out of the soil-cement structure, resulting in a "swiss cheese" appearance, which can weaken the soil-cement structure. The U.S. Bureau of Reclamation requires that clay balls greater than 1 in. be removed, and imposes a 10 percent limit on clay balls passing the 1-in. sieve.[11] The presence of fines is not always detrimental, however. Some nonplastic fines in the soil can be beneficial. In uniformly graded sands or gravels, nonplastic fines including fly ash, cement-kiln dust, and aggregate screenings serve to fill the voids in the soil structure and help reduce the cement content.

3.2-Cement

For most applications, Type I or Type II portland cement conforming to ASTM C 150 is normally used.

Table 3.1 — Typical cement requirements for various soil types[21]

AASHTO soil classification	ASTM soil classification	Typical range of cement requirement,* percent by weight	Typical cement content for moisture-density test (ASTM D 558), percent by weight	Typical cement contents for durability tests (ASTM D 559 and D 506), percent by weight
A-1-a	GW, GP, GM, SW, SP, SM	3-5	5	3-5-7
A-1-b	GM, GP, SM, SP	5-8	6	4-6-8
A-2	GM, GC, SM, SC	5-9	7	5-7-9
A-3	SP	7-11	9	7-9-11
A-4	CL, ML	7-12	10	8-10-12
A-5	ML, MH, CH	8-13	10	8-10-12
A-6	CL, CH	9-15	12	10-12-14
A-7	MH, CH	10-16	13	11-13-15

*Does not include organic or poorly reacting, soils. Also, additional cement may be required for severe exposure conditions such as slope-protect&.

Cement requirements vary depending on desired properties and type of soils. Cement contents may range from as low as 4 to a high of 16 percent by dry weight of soil. Generally, as the clayey portion of the soil increases, the quantity of cement required increases. The reader is cautioned that the cement ranges shown in Table 3.1 are not mix-design recommendations. The table provides initial estimates for the mix-proportioning procedures discussed in Chapter 5.

3.3-Admixtures

Pozzolans such as fly ash have been used where the advantages outweigh the disadvantages of storing and handling an extra material. Where pozzolans are used as a cementitious material, they should comply with ASTM C 618. The quantity of cement and pozzolan required should be determined through a laboratory testing program using the specific cement type, pozzolan, and soil to be used in the application.

For highly plastic clay soils, hydrated lime or quicklime may sometimes be used as a pretreatment to reduce plasticity and make the soil more friable and susceptible to pulverization prior to mixing with cement. Chemical admixtures are rarely used in soil cement. Although research has been conducted in this area, it has been primarily limited to laboratory studies with few field investigation.[24-29]

3.4-Water

Water is necessary in soil cement to help obtain maximum compaction and for hydration of the portland cement. Moisture contents of soil cement are usually in the range of 10 to 13 percent by weight of oven-dry soil cement.

Potable water or other relatively clean water, free from harmful amounts of alkalies, acids, or organic matter, may be used. Seawater has been used satisfactorily. The presence of chlorides in seawater may increase early strengths.

4-PROPERTIES

4.1-General

The properties of soil cement are influenced by several factors, including (a) type and proportion of soil, cementitious materials, and water content, (b) compaction, (c) uniformity of mixing, (d) curing conditions, and (e) age of the compacted mixture. Because of these factors, a wide range of values for specific properties may exist. This chapter provides information on several properties and how these and other factors affect various properties.

4.2-Density

Density of soil cement is usually measured in terms of dry density, although moist density may be used for field density control. The moisture-density test (ASTM D 558) is used to determine proper moisture content and density (referred to as optimum moisture content and maximum dry density) to which the soil-cement mixture is compacted. A typical moisture-density curve is shown in Fig. 4.1. Adding cement to a soil generally causes some change in both the optimum moisture content and maximum dry density for a given compactive effort. However, the direction of this change is not usually predictable. The flocculating action of the cement tends to produce an increase in optimum moisture content and a decrease in maximum density, while the high specific gravity of the cement relative to the soil tends to produce a higher density. In general, Shen[30] showed that for a given cement content, the higher the density, the higher the compressive strength of cohesionless soil-cement mixtures.

Prolonged delays between the mixing of soil cement and compaction have an influence on both density and strength. Studies by West[31] showed that a delay of more than 2 hr between mixing and compaction results in a significant decrease in both density and compressive strength. Felt[32] had similar findings but also showed that the effect of time delay was minimized, provided the mixture was intermittently mixed several times an hour, and the moisture content at the time 'of compaction was at or slightly above optimum.

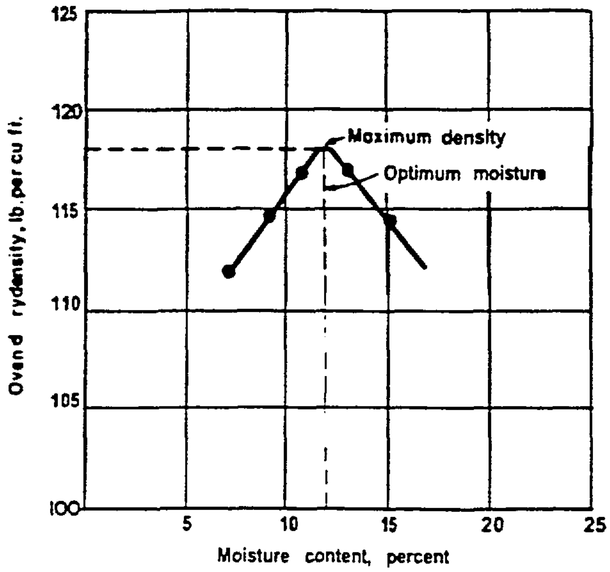

Fig. 4.1- Typical moisture-density curve

Table 4.1 — Ranges of unconfined compressive strengths of soil-cement[33]

Soil type	Soaked compressive strength,* (psi) 7-day	28-day
Sandy and gravelly soils: AASHTO groups A-1, A-2, A-3 Unified groups GW, GC, GP, GM, SW, SC, SP, SM	300-600	400-1000
Silty soils: AASHTO groups A-4 and A-5 Unified groups ML and CL	250-500	300-900
Clayey soils: AASHTO groups A-6 and A-7 Unified groups MH and CH	200-400	250-600

*Specimens moist-cured 7 or 28 days, then soaked in water prior to strength testing.

4.3-Compressive strength

Unconfined compressive strength f_c' is the most widely referenced property of soil cement and is usually measured according to ASTM D 1633. It indicates the degree of reaction of the soil-cement-water mixture and the rate of hardening. Compressive strength serves as a criterion for determining minimum cement requirements for proportioning soil cement. Because strength is directly related to density, this property is affected in the same manner as density by degree of compaction and water content.

Typical ranges of 7- and 28-day unconfined compressive strengths for soaked, soil-cement specimens are given in Table 4.1. Soaking specimens prior to testing is recommended since most soil-cement structures may become permanently or intermittently saturated during their service life and exhibit lower strength under saturated conditions. These data are grouped under broad textural soil groups and include the range of soil types normally used in soil-cement construction. The range of values given are representative for a majority of soils

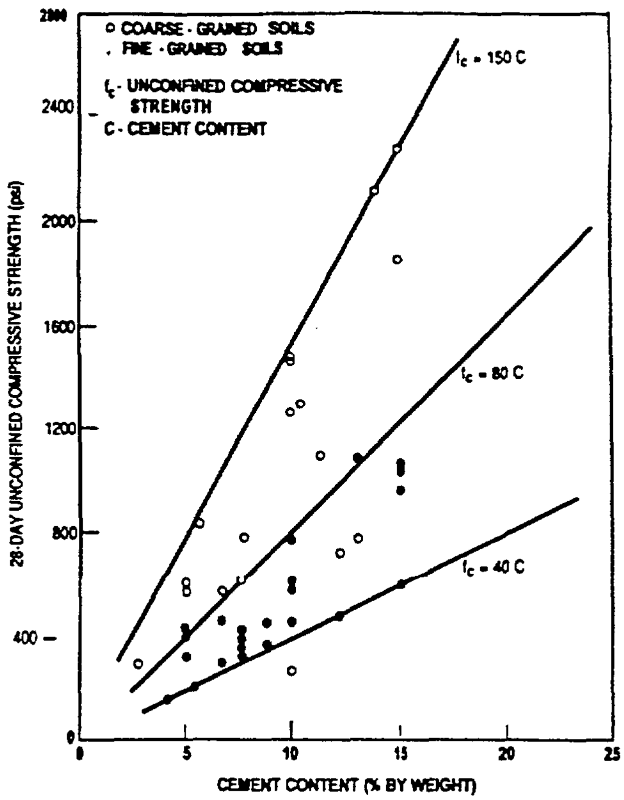

Fig. 4.2-Relationship between cement content and unconfined compressive strength for soil-cement mixtures

normally used in the United States in soil-cement construction. Fig. 4.2 shows that a linear relationship can be used to approximate the relationship between compressive strength and cement content, for cement contents up to 15 percent and a curing period of 28 days.

Curing time influences strength gain differently depending on the type of soil. As shown in Fig. 4.3, the strength increase is greater for granular soil cement than for fine-grained soil cement.

4.4-Flexural (tensile) strength (modulus of rupture)

Flexural-beam tests (ASTM D 1635), direct-tension tests, and split-tension tests have all been used to evaluate flexural strength. Flexural strength is about one-fifth to one-third of the unconfined compressive strength. Data for some soils are shown in Fig. 4.4. The ratio of flexural to compressive strength is higher in low-strength mixtures (up to 1/3 f_c') than in high-strength mixtures (down to less than 1/5 f_c'). A good approximation for the flexural strength R is[34]

$$R = 0.51\,(f_c')^{0.88}$$

where

R = flexural strength, psi

f_c' = unconfined compressive strength, psi

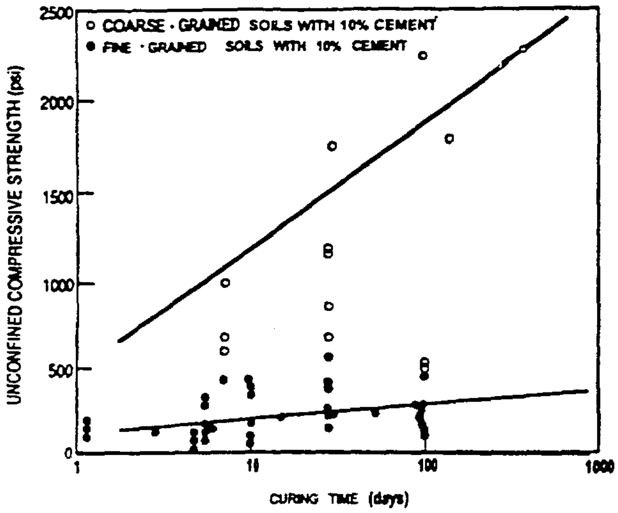

Fig. 4.3-Effect of curing time on unconfined concrete compressive strength of some soil-cement mixture34

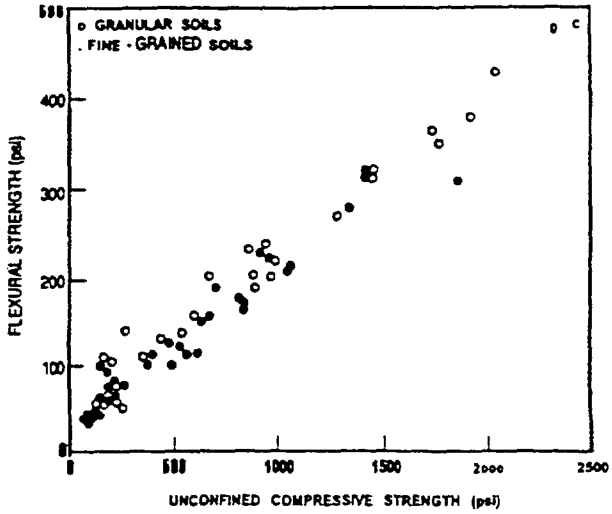

Fig. 4.4-Relationship between unconfined compressive strength and flexural strength of soil-cement mixtures34

Table 4.2 — Permeability of cement-treated soils[17]

ASTM soil classification	Dry density, lb/ft³	Moisture content, percent	Cement content percent by weight	K coefficient of permeability ft per yr, 10^{-6} cm/sec	Gradation analysis, percent passing						
					#4 (4.75 mm)	#10 (2.0 mm)	#40 (425 μm)	#200 (75 μm)	005 mm	0005 mm	Cement* required, by weight
Standard Ottawa sand	108.2 112.8 117.6	10.8 9.4 9.7	0 5.3 10.5	48,800 6900 76	(100 percent passing #20 (850 μm): 0 percent passing #30 (600 μm)						—
Graded Ottawa sand	103.2 104.7 107.4	13.7 13.6 12.3	0 5.4 10.5	16,300 470 21	100	100	28	2	—	—	—
Fine sand (SP)	101.0 100.9 103.6 105.3	12.2 13.2 12.3 12.0	0 3.2 6.5 9.5	750 560 190 21	100	100	91	7	1	—	11.5
Silty sand (SM)	100.8 99.9 104.0	14.9 14.7 15.1	0 3.2 6.4	5000 1400 60	100	100	96	13	12	2	8.0
Fine sand (SP)	100.1 105.8 109.3	16.0 14.8 13.5	0 6 12.2	360 20 1	99	99	96	6		61	—
Fine sand (SP)	101.0 106.7 108.2 108.8	13.8 13.3 13.4 13.4	3.1 6.3 ... 9.6	140 33 0.3 0.02	100	100	94	2		-	11.0
Fine sand (SP)	112.5 115.8	11.0 10.4	0 5.5	36 5	—	97	—	—	11	4	—
Fine sand (SP)	111.7 115.2	12.0 11.7	0 5.5	23 8	100	99	—	—	9	3	—
Silty sand (SM)	121.9 125.5	9.6 8.0	0 8.6	16 0.1	98	94	66	20	18	5	—
Silty sand (SM)	117.9 123.0	10.8 8.1	0 8.9	10 2 5	99	97	69	16	12	4	—
Silty sand (SM)	112.5 115.0	11.5 12.3	0 5.5 0	3 5	—	98	—	—	12	5	—
Silty sand (SM)	118.7 119.2	11.0 10.5	9.1	0.1	100	99	88	36	25	7	—
Silty sand (SM)	125.0	— 10.1	0 3.3 7.3	16 0.4 0.07	100	75	41	13	12	5	5.0

*Cement requirement based on ASTM Standard Freeze-Thaw and Wet-Dry Tests for soil-cement mixtures and PCA paving criteria.

Values of tensile strength deduced from the results of flexure, direct-tension, and split-tension tests may differ, due to the effects of stress concentrations and differences between moduli in tension and compression. Research by Radd[35] has shown that the split-tension test yields values that do not deviate by more than 13 percent from the direct tensile strength.

4.5-Permeability

Permeability of most soils is reduced by the addition of cement. Table 4.2 summarizes results from laboratory permeability tests conducted on a variety of soil types. A large-scale seepage test was performed by the U.S. Bureau of Reclamation on a section of layered stairstep soil cement facing at Lubbock Regulating Reservoir in Texas.[36] Results indicated a decrease in permeability with time, possibly due to shrinkage cracks in the soil-cement filling with sediment and the tendency for the cracks to self-heal. Seepage was as much as 10 times greater in the cold winter months than the hot summer months. The reduced summer seepage was probably caused by thermal expansion which narrowed the crack widths and by the presence of algae growth in the cracks.

In multiple-lift construction, higher permeability can generally be expected along the horizontal surfaces of the lifts than perpendicluar to the lifts. Research by Nussbaum[23] has shown permeabilities for flow parallel to the compaction plane were 2 to 20 times larger than values for flow normal to the compaction plane.

4.6-Shrinkage

Cement-treated soils undergo shrinkage during drying. The shrinkage and subsequent cracking depend on cement content, soil type, water content, degree of compaction, and curing conditions. Fig. 4.5 shows the results of field data on shrinkage cracking from five test locations in Australia.[37] Soil cement made from each soil type produces a different crack pattern. Soil cement made with clays develops higher total shrinkage, but crack widths are smaller and individual cracks more closely spaced (e.g., hairline cracks, spaced 2 to 10 ft apart). Soil cement made with granular soils produces less shrinkage, but larger cracks spaced at greater intervals (usually 10 to 20 ft or more apart).[33] Methods suggested for reducing or minimizing shrinkage cracks include keeping the soil-cement surface moist beyond the normal curing periods and placing the soil cement at slightly below optimum moisture content.

4.7-Layer coefficients and structural numbers

Several different methods are currently being used for pavement design. In the AASHTO method for flexible pavement design, layer coefficient a, values are assigned to each layer of material in the pavement structure to convert actual layer thicknesses into a structural number SN. This layer coefficient expresses the empirical relationship between SN and thickness D, and is a measure of the relative ability of the material to function as a structural component of the pavement.

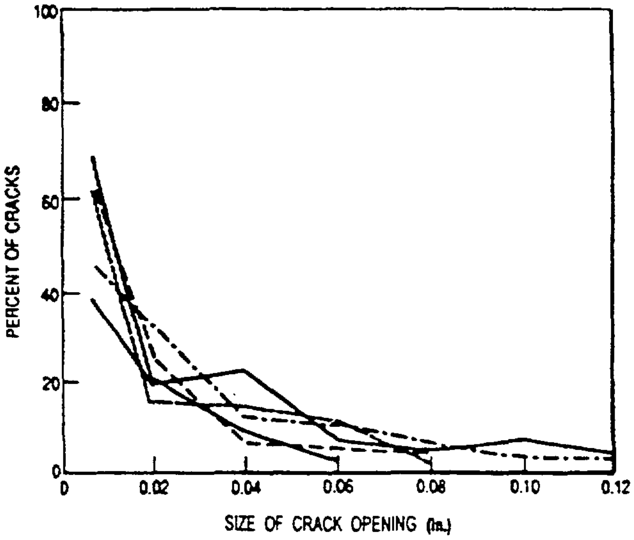

Fig. 4.5-Frequency distribution of various sizes of shrinkage cracks in soil cement[37]

Table 4.3 — Examples of AASHTO layer coefficients for soil cement used by various state DOTs

State	Layer coefficient a	Compressive strength requirement
Alabama	0.23 0.20 0.15	650 psi min 400-650 psi Less than 400 psi
Arizona	0.28 0.23	For cement-treated base with minimum 800 psi (plant mixed) For cement-treated subgrade with 800 psi min (mixed-in-place)
Delaware	0.20	
Florida	0.15 0.20	300 psi (mixed-in-place) 500 psi (plant mixed)
Georgia	0.20	350 psi
Louisiana	0.15 0.18 0.23	200 psi min 400 psi min Shell and sand with 650 psi min
Montana	0.20	400 psi min
New Mexico	0.23 0.17 0.12	650 psi min 400-650 psi Less than 400 psi
Pennsylvania	0.20 0.30	650 psi min (mixed-in-place) 650 psi min (plant mixed)
Wisconsin	0.23 0.20 0.15	650 psi min 400-650 psi Less than 400 psi

The following general equation for structural number reflects the relative impact of the layer coefficient and thickness[4]

$$SN = a_1 D_1 + a_2 D_2 + a_3 D_3$$

where a_1, a_2, and a_3 = layer coefficients of surface, base, and subbase, respectively; and D_1, D_2, and D_3 = corresponding layer thicknesses.

Table 5.1 — PCA criteria for soil-cement as indicated by wet-dry and freeze-thaw durability tests[1]

AASHTO soil group	Unified soil group	Maximum allowable weight loss, percent
A-1-a	GW, GP, GM, SW, SP, SM	14
A-1-b	GM, GP, SM, SP	14
A-2	GM, GC, SM, SC	14*
A-3	SP	14
A-4	CL, ML	10
A-5	ML, MH, CH	10
A-6	CL, CH	7
A-7	OH, MH, CH	7

*10 percent is maximum allowable weight loss for A-2-6 and A-2-7 soils.

Additional criteria

1. Maximum volume changes during durability test should be less than 2 percent of the initial volume.
2. Maximum water content during the test should be less than the quantity required to saturate the sample at the time of molding.
3. Compressive strength should increase with age of specimen.
4. The cement content determined as adequate for pavement, using the PCA criteria above, will be adequate for soil-cement slope protection that is 5 ft or more below the minimum water elevation. For soil cement that is higher than that elevation, the cement content should be increased two percentage points.

The layer coefficients are actually the average of a set of multiple regression coefficients, which indicate the effect of the wearing course, the base course, and the subbase on the pavement's performance. Typical soil-cement layer coefficient a, values used by state departments of transportation are given in Table 4.3.

5-MIX PROPORTIONING

5.1-General

The principal structural requirements of a hardened soil-cement mixture are based on adequate strength and durability. For water resource applications such as liners, permeability may be the principal requirement. Table 3.1 indicates typical cement contents for pavement applications. Detailed test procedures for evaluating mix proportions are given in the Portland Cement Association Soil-Cement Laboratory Handbook[1] and by the following ASTM test standards:

ASTM D 558	Test for Moisture-Density Relations of Soil-Cement Mixtures
ASTM D 559	Wetting-and-Drying Tests of Compacted Soil-Cement Mixtures
ASTM D 560	Freezing-and-Thawing Tests of Compacted Soil-Cement Mixtures
ASTM D 1557	Moisture-Density Relations of Soils and Soil Aggregate Mixtures Using 10-lb Rammer and 18-in. Drop
ASTM D 1632	Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory
ASTM D 1633	Test for Compression Strength of Molded Soil-Cement Cylinders
ASTM D 2901	Test for Cement Content of Freshly Mixed Soil-Cement

5.2-Proportioning

Various criteria are used by different organizations to determine acceptable mix proportions. The Portland Cement Association (PCA) criteria are summarized in Table 5.1. Cement contents sufficient to prevent weight losses greater than the values indicated after 12 cycles of wetting-drying-brushing or freezing-thawing-brushing are considered adequate to produce a durable soil cement.

Table 5.2 — USACE durability requirement[38]

Type of soil stabilized*	Maximum allowable weight loss after 12 wet-dry or freeze-thaw cycles, percent of initial specimen weight
Granular, *PI< 10*	11
Granular, *PI> 10*	
Silt	:
Clays	6

*Refer to MIL-STD-619B and MIL-STD-621A, U.S. Army corps of Engineers.

Table 5.3 — USACE minimum unconfined compressive strength criteria[38]

Stabilized soil layer	Minimum unconfined compressive strength at 7 days, psi: Flexible pavement	Rigid pavement
Base course	750	500
Subbase course, select material or subgrade	250	200

The U.S. Army Corps of Engineers (USACE) follows its technical manual, "Soil Stabilization for Pavements," TM 5-822-4.[38] The durability and strength requirements for portland cement stabilization are given in Tables 5.2 and 5.3, respectively. USACE requires that both criteria be met before a stabilized layer can be used to reduce the required surface thickness in the design of a pavement system. USACE frequently increases the cement content by 1 or 2 percent to account for field variations. For bank protection, USACE has an unnumbered draft Engineer Technical letter for interim guidance.[39]

The U.S. Bureau of Reclamation (USBR) design criteria for soil-cement slope protection on dams allow maximum losses during freeze-thaw and wet-dry durability tests of 8 and 6 percent, respectively. These criteria were developed specifically for soil cement slope protection using primarily silty sands. In addition, USBR requires a minimum compressive strength of 600 psi at 7 days and 875 psi at 28 days. To allow for variations in the field, it is USBR's practice to add two percentage points to the minimum cement content that meets all of the preceding design criteria.[11]

Pima County, Ariz., uses a considerable amount of soil cement for streambank slope protection. The county requires the soil cement to have a minimum 7-day compressive strength of 750 psi. The cement content is increased two percentage points for additional erosion resistance and to compensate for field variation. This results in a 7-day compressive strength of about 1000 psi. To facilitate quality-control testing during construction, the county has established an acceptance criterion based on a 1-day compressive strength test. For the local soils typically used, the 1-day strength is generally between 50 to 60 percent of the 7-day value.

The PCA Soil-Cement Laboratory Handbook[1] describes a shortcut test procedure that can be used to determine the cement content for sandy soils. The procedure uses charts developed from previous tests on similar soils. The only tests required are a sieve analysis, a moisture-density test, and a compressive strength test. Relatively small samples are needed. All tests can be completed in 1 day, except the 7-day compressive strength test.

5.3-Special considerations

5.3.1 *Strength versus durability*-In many soil-cement applications, both strength and durability requirements must be met to achieve satisfactory service life. ASTM D 559 and D 560 are standard test methods that are conducted to determine, for a particular soil, the amount of cement needed to hold the mass together permanently and to maintain stability under the shrinkage and expansive forces that occur in the field. It is common practice, however, to use compressive strength to determine the minimum cement content. Fig. 5.1 illustrates the general relationship between compressive strength and durability for soil cement. It is apparent from these curves that a compressive strength of 800 psi would be adequate for all soils, but this strength would be higher than needed for most soils and would result in a conservative and more costly design. The determination of a suitable design compressive strength is simplified when materials within a narrow range of gradations and/or soil types are used. As a result, some agencies have determined and used successfully, for a particular type of material, a compressive strength requirement generally based on results of the wet-dry and freeze-thaw tests.

5.3.2 *Compressive strength specimen size*-Compressive strength tests are frequently conducted on test specimens obtained from molds commonly available in soil laboratories and used for other soil-cement tests. These test specimens are 4.0 in. in diameter and 4.584

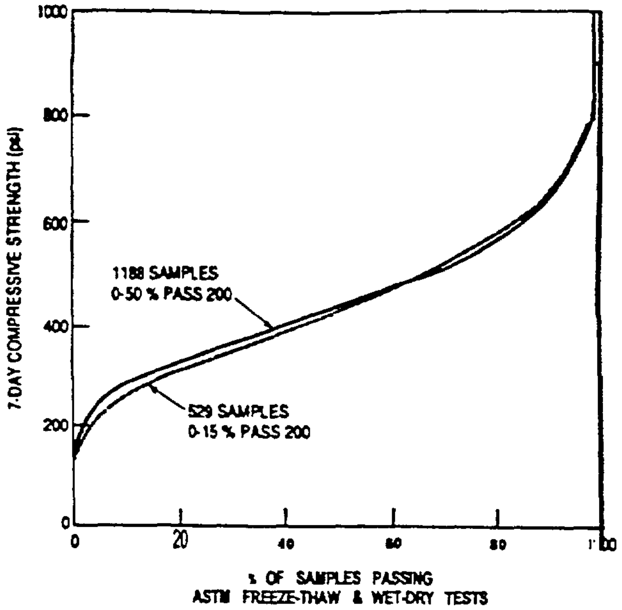

Fig. 5.1-Relationship between compressive strength and durability of soil cement based on Portland Cement Association durability criteria[1]

in. in height with a height-to-diameter *(h/d)* ratio of 1.15. This differs from conventional concrete molds, which use *h/d* of 2.00. The *h/d* of 2.00 provides a more accurate measure of compressive strength from a technical viewpoint, since it reduces complex stress conditions that may occur during crushing of lower *h/d* specimens. In soil-cement testing, however, the lower *h/d* (1.15) specimens are frequently used. Most of the compressive strength values given in this report are based on *h/d* = 1.15. Using the correction factor for concrete given in ASTM C 42, an approximate correction can be made for specimens with *h/d* of 2.00 by multiplying the compressive strength value by a factor of 1.10.

5.3.3 *Poorly reacting sandy soils*-Occasionally, certain types of sandy soils are encountered that cannot be treated successfully with normal amounts of portland cement. Early research[21] showed that organic material of an acidic nature usually had an adverse effect on soil cement. The study showed that organic content and pH do not in themselves constitute an indication of a poorly reacting sand. However, a sandy soil with an organic content greater than 2 percent or having a pH lower than 5.3, in all probability, will not react normally with cement. These soils require special studies prior to use in soil cement.

5.3.4 *Sulfate resistance*-As with conventional concrete, sulfates will generally attack soil cement. Studies by Cordon and Sherwood[40,41] have indicated that the resistance to sulfate attack differs for cement-treated coarse-grained and fine-grained soils and is a function of the clay and sulfate concentrations. The studies showed that sulfate-clay reactions are more detrimental than sulfate-cement reactions, resulting in deterioration of fine-grained soil cement more rapidly than coarse-grained soil cement. Also, increasing the cement content of soil-cement mixtures may be more beneficial than changing to a sulfate-resistant type of cement.

6-CONSTRUCTION

6.1-General

In the construction of soil cement, the objective is to obtain a thoroughly mixed, adequately compacted, and cured material. Several references are available[8,13,42-44] that discuss soil-cement construction methods for various applications. Specifications on soil-cement construction are also readily available.[45-47]

Soil cement should not be mixed or placed when the soil or subgrade is frozen or when the air temperature is below 45 F. However, a common practice is to proceed with construction when the air temperature is at least 40 F and rising. When the air temperature is expected to reach the freezing point, the soil cement should be protected from freezing for at least 7 days. Soil-cement construction typically requires the addition of water equivalent to 1 to 1 ½ in. of rain; therefore, a light rainfall should not delay construction. However, a heavy rainfall that occurs after most of the water has been added can be detrimental. If rain falls during cement-spreading operations, spreading should be stopped and the cement already spread should be quickly mixed into the soil mass. Compaction should begin immediately and continue until the soil cement is completely compacted. After the mixture has been compacted, rain usually will not harm it.

Fig. 6.1 Transverse single-shaft mixer processing soil cement in place; multiple passes are required

6.2-Materials handling and mixing

Soil cement is either mixed in place or mixed in a central mixing plant. The typical types of mixing equipment are:

1. In-place traveling mixers
   a. Transverse single-shaft mixer
   b. Windrow-type pugmill
2. Central mixing plant
   a. Continuous-flow-type pugmill
   b. Batch-type pugmill
   c. Rotary-drum mixer

6.2.1 *Mixed in place*- Mixing operations with subgrade materials are performed with transverse single-shaft-type mixers (Fig. 6.1 and 6.2). Mixing with borrow materials may be performed with single-shaft or windrow-type pugmill mixers (Fig. 6.3). Almost all types of soil, from granular to fine-grained, can be adequately pulverized and mixed with transverse single-shaft mixers. Windrow-type pugmills are generally limited to nonplastic to slightly plastic granular soils.

6.2.1.1 *Soil preparation*-During grading operations, all soft or wet subgrade areas are located and corrected. All deleterious material such as stumps, roots, organic soils, and aggregates larger than 3 in. should be removed. For single-shaft mixers, the soil is shaped to the approximate final lines and grades prior to mixing. Proper moisture content aids in pulverization. For granular soils, mixing at less than optimum moisture content minimizes the chances for cement balls to form. For fine-grained soils, moisture content near optimum may be necessary for effective pulverization.

Fig. 6.2-Mixing chamber of a transverse single-shaft mixer

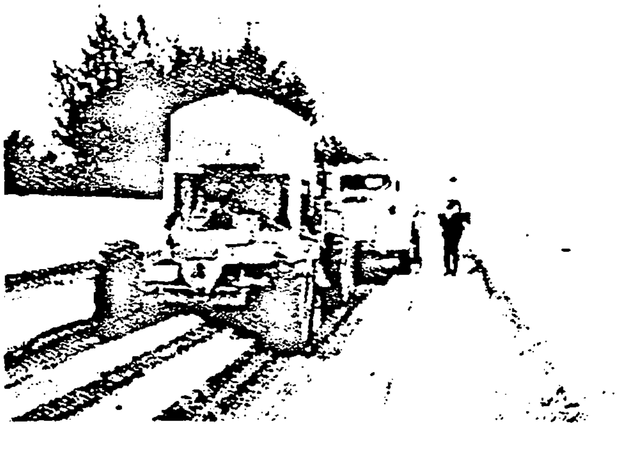

Fig. 6.3- Windrow-type traveling pugmill mixing soil cement from windrows of soil material

6.2.1.2 *Cement application*-Cement is generally distributed in bulk using a mechanical spreader (two examples of which are shown in Fig. 6.4 and 6.5) or, for small projects, by hand-placing individual cement bags. The primary objective of the cement-spreading operation is to achieve uniform distribution of the cement in the proper proportions.

To obtain a uniform cement spread, the mechanical spreader must be operated at uniform speed with a relatively constant level of cement in the hopper. The spreader must have adequate traction to produce a uniform cement spread. Traction can be aided by wetting and rolling the soil before spreading the cement. When operating in loose sands or gravel, slippage can be overcome by using cleats on the spreader wheels. The mechanical cement-spreader can also be attached directly behind a bulk-cement truck. Cement is moved pneumatically from the truck through an air-separator cyclone that dissipates the air pressure; it then falls into the hopper of the spreader. Forward speed must be slow and even. Sometimes a motor grader or loader pulls the truck to maintain this slow, even, forward speed. Although pipe cement-spreaders attached to cement-transport trucks have been used in some areas with mixed results, mechanical spreaders are generally preferred. The amount of cement required is specified as a percentage by weight of oven-dry soil, or in lb of cement per ft^3 of compacted soil cement. Table 6.1 can be used to determine quantities of cement per yd^2 of soil-cement placement.

6.2.1.3 *Pulverization and* mixing-Single-shaft mixers are typically utilized to pulverize and mix cement with subgrade soils. Agricultural-type equipment is not recommended due to relatively poor mixing uniformity. Pulverization and mixing difficulties increase with higher fines content and plasticity of the soils being treated. In-place mixing efficiency, as measured by the strength of the treated soil, may be less than that obtained in the laboratory. This reduced efficiency is sometimes compensated for by increasing the cement content by 1 or 2 percentage points from that determined in the laboratory testing program.

Windrow-type traveling mixing machines will pulverize friable soils. Nonfriable soils, however, may need preliminary pulverizing for proper mixing. This is usually done before the soil is placed in windrows for processing. The prepared soil is bladed into windrows and a "proportioning" device is pulled along to provide a uniform cross section. When borrow materials are used, a windrow spreader can be used to proportion the material. Nonuniform windrows cause variations in cement content, moisture content, and thickness. The number and size of windrows needed depend on the width and depth of treatment and on the capacity of the mixing machine.

Cement is spread on top of a partially flattened or slightly trenched prepared windrow. A mixing machine then picks up the soil and cement and dry-mixes them with the first few paddles in the mixing drum. At that point, water is added through spray nozzles and the re-

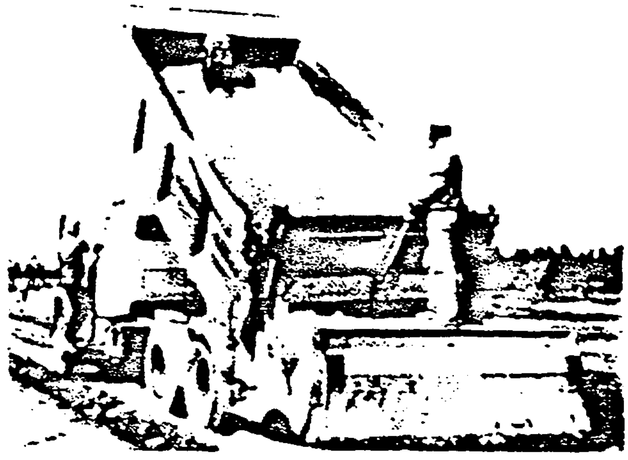

Fig. 6.4-Mechanical cement spreader attached to dump truck

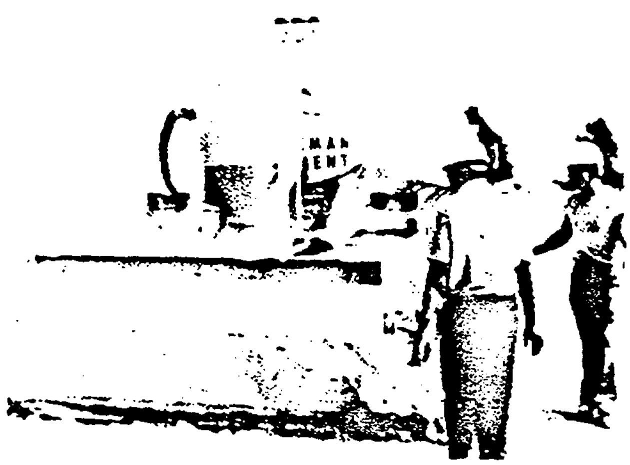

Fig. 6.5-Mechanical cement spreader attached to bulk cement transport truck

Table 6.1 — Cement spread requirement[51]

Cement content, lb/ft^3 of compacted soil cement	Cement spread, lb/yd^2/in. of thickness of compacted soil cement
4.5	3.38
5.0	3.75
5.5	4.13
6.0	4.50
6.5	4.88
7.0	5.25
7.5	5.63
8.0	6.0
8.5	6.38
9.0	6.75
9.5	7.13
10.0	7.50
10.5	7.88
11.0	8.25
11.5	8.63
12.0	9.0
12.5	9.38
13.0	9.75
13.5	10.13
14.0	10.50
14.5	10.88
15.0	11.25
15.5	11.63
16.0	12.0

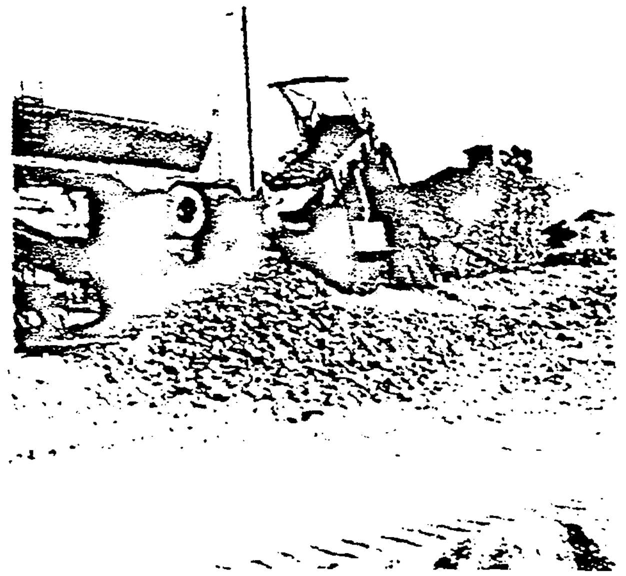

Fig. 6.6- Vibrating screen removing oversized material from soil portion of mixture

maining paddles complete the mixing. A strikeoff attached to the mixing machine spreads the mixed soil cement.

6.2.2 *Central plant mixing*-Central mixing plants are normally used for projects involving borrow materials. Granular borrow materials are generally used because of their low cement requirements and ease in handling and mixing. Clayey soils or materials containing clay lenses should be avoided because they are difficul to pulverize. There are two basic types of central plant mixers-pugmill mixers, either continuous-flow or batch, and rotary-drum mixers. Although batch pugmills and rotary-drum mixers have been used satisfactorily, the most common central plant mixing method is the continuous-flow pugmill mixer. Production rates with this type of mixer vary between 200 and 800 t/hr.

6.2.2.1 *Borrow material*-Soil borrow sources are usually located near the construction site. Natural deposits are generally variable to an extent and do not contain consistent, uniform materials throughout.

The U.S. Bureau of Reclamation recommends the following procedure for handling borrow material.[48] If the material in the borrow area varies with depth, full-face cuts should be made with the excavation machinery. This selective excavation insures that some material from each layer is obtained in each cut. If the material varies laterally across the borrow area, or differs from one spot to another, loads from different locations in the borrow area should be mixed. After the material has been excavated, soil can be further blended at the stockpile. Alternating the loads from different parts of the borrow area helps to blend soil gradations in the stockpile. Mixing for uniformity of gradation and moisture can also be done as the material is pushed into the stockpile. For example, if excavated material is dumped at the base of the stockpile, it can be pushed up the stockpile with a bulldozer. A front-end loader can then be used to load the soil feed. This tends to mix a vertical cut of the stockpile, which causes further mixing of any layers that might exist in the pile.

As the borrow material is excavated it should be checked for unsuitable material such as clay lenses, cobbles, or cemented conglomerates. Such materials do not adequately break down in a pugmill mixer. Removal of some oversize clay balls and other large particles can be done by screening through 1 to 1 ½-in. mesh (Fig. 6.6). In some cases, selective excavation may be necessary to avoid excessive clay lenses.

6.2.2.2 *Mixing*-The objective is to produce a thorough and intimate mixture of the soil, cement, and water in the correct proportions. A diagram of a continuous-flow pugmill plant is shown in Fig. 6.7. A typical plant consists of a soil bin or stockpile, a cement silo with surge hopper, a conveyor belt to deliver the soil and cement to the mixing chambers, a mixing chamber, a water-storage tank for adding water during mixing, and a holding or gob hopper to temporarily store the mixed soil cement prior to loading (Fig. 6.8).

A pugmill mixing chamber consists of two parallel shafts equipped with paddles along each shaft (Fig.

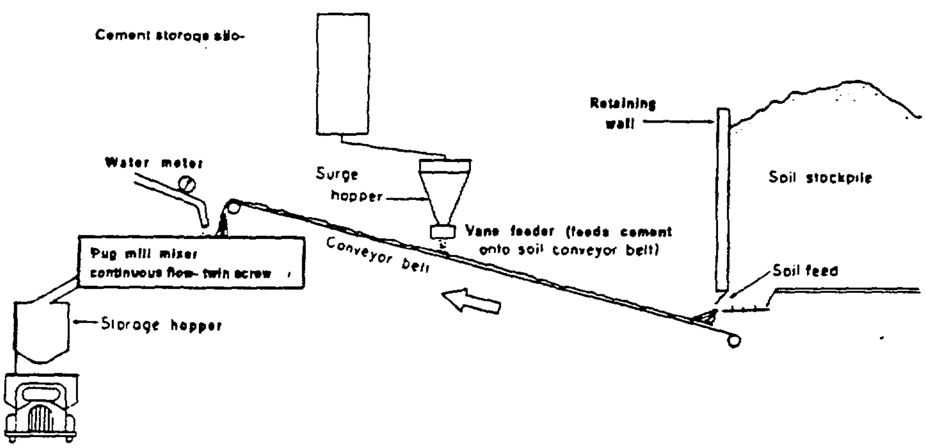

Fig. 6.7-Diagram of continuous-flow central plant for mixing soil cement

6.9). The twin-shafts rotate in opposite directions, and the soil cement is moved through the mixer by the pitch of the paddles.

Material feed, belt speed, pugmill tilt, and paddle pitch are adjusted to optimize the amount of mixing in the pugmill. Thorough blending in the mixer is very important, and the length of mixing time is used to control this factor. Some specifications dictate the minimum blending time. Usually 30 sec is specified, although satisfactory blending has been achieved in shorter periods, depending on the efficiency of the mixer.

6.2.2.3 *Transporting*-To reduce evaporation losses during hot, windy conditions and to protect against sudden showers, rear and bottom dump trucks are often equipped with protective covers. No more than 60 min should elapse between the start of moist-mixing and the start of compaction. Haul time is usually limited to 30 min.

For multiple-layer stairstep construction, as used for slope protection, earthen ramps are constructed at intervals along the slope to enable trucks to reach the layer to be placed. These are constructed at a 45 deg horizontal angle to the slope, normally 2 ft thick and spaced about 300 to 400 ft apart.

At large-volume projects, such as the South Texas Nuclear Power Plant, a conveyor system can be used to deliver the soil cement to the spreader. This removes the necessity for ramp construction and truck maneuvering, and provides a cleaner end-product. Narrower layers can also be placed using the conveyor system, since the width needed to facilitate the haul trucks is eliminated. The soil cement can be delivered either from above or below directly to a spreader box.

6.2.2.4 *Placing and spreading*-The mixed soil cement should be placed on a firm subgrade, without segregation, and in a quantity that will produce a compacted layer of uniform thickness and density conforming to the design grade and cross section. The subgrade and all adjacent surfaces should be moistened prior to placing soil cement.

There is a wide variety of spreading devices and methods. Using a motor grader or spreader box attached to a dozer are the most commonly used means. Spreading may also be done with asphalt-type pavers. Some pavers are equipped with one or more tamping bars, which provide initial compaction. Soil cement is usually placed in a layer 25 to 50 percent thicker than the final compacted thickness. For example, a 8 to 9 in. loosely placed layer will produce a compacted thickness of about 6 in. This relationship varies slightly with the type of soil, method of placement and degree of compaction. The actual thickness of the loosely spread layer is determined from contractor experience or trial-and-error methods. Compacting, finishing, and curing follow the same procedures as for mixed-in-place construction.

6.2.2.5 *Bonding successive layers*--Bonding successive layers of soil cement is an important requirement for applications such as slope protection. It is essential that each completed surface remain clean and moist, but not wet, until it is covered with the next layer. Mud and debris tracked onto a surface will significantly reduce bonding. Other methods which have been effective in improving bond between layers include the following:[49,50]

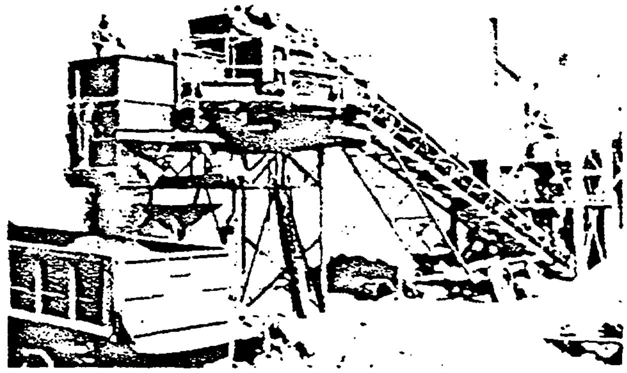

Fig. 6.8- Typical continuous-flow central mixing plant

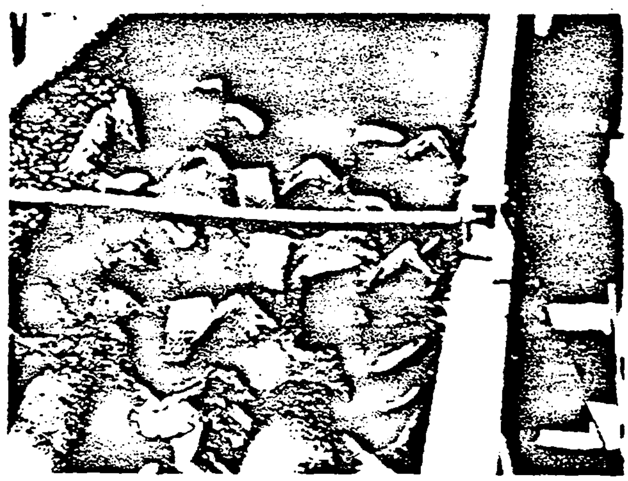

Fig. 6.9-Mixing paddles of a twin-shaft, continuous flow central mixing plant

1. Minimizing time between placement of successive layers.
2. Use of either dry cement or cement slurry. The dry cement should be applied at about 1 lb/yd^2 to a moistened surface immediately prior to placement. The cement slurry mix should have a water-cement ratio of about 0.70 to 0.80.
3. After the soil cement has set, brushing the surface with a power broom to provide a roughened surface texture.
4. Use of chemical retarding agents.

6.3-Compaction

Compaction begins as soon as possible and is generally completed within 2 hr of initial mixing. The detrimental effects of delayed compaction on density and strength have already been described in Section 4.2. No section is left unworked for longer than 30 min during

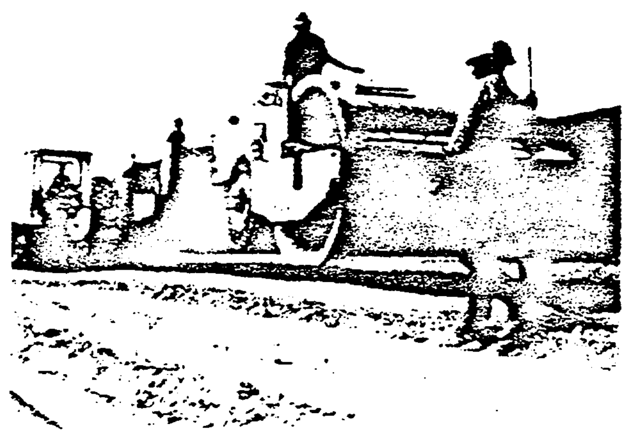

Fig. 6.10-Compacting outer edge with rounded steel flange welded to steel-wheel roller

compaction operations. The principles governing compaction of soil cement are the same as those for compacting the same soil without cement treatment. For maximum density, the soil-cement mixture should be compacted at or near optimum moisture content as determined by ASTM D 558 or D-1557. Most specifications require soil cement to be uniformly compacted to a minimum of between 95 and 98 percent of maximum density. Moisture loss by evaporation during compaction, indicated by a graying of the surface, should be replaced with light applications of water.

Various types of rollers have been used for soil cement. Tamping or sheepsfoot rollers are used for initial compaction of fine-grained soils. The sheepsfoot roller is often followed by a multiple-wheel, rubber-tired roller for finishing. For granular soils, vibratory steel-wheeled or heavy rubber-tired rollers are generally used. To obtain adequate compaction, it is sometimes necessary to operate the rollers with ballast to produce greater contact pressure. The general rule is to use the greatest contact pressure that will not exceed the bearing capacity of the soil-cement mixture. Compacted layer thicknesses generally range from 6 to 9 in. Greater thicknesses, particularly for granular soils, can be compacted with heavy equipment designed for thicker lifts. Regardless of the lift thickness and compaction equipment used, the fundamental requirement is that the compacted layer achieve the specified minimum density throughout the lift.

6.4-Finishing

As compaction nears completion, the surface of the soil cement is shaped to the design line, grade, and cross section. Frequently, the surface may require lift scarification to remove imprints left by equipment or potential surface compaction planes.* The scarification can be done with a weeder, nail drag, spring tooth, or spiketooth harrow. For soils containing an appreciable quantity of gravel, scarification may not be necessary. Following scarification, final surface compaction is performed using a nonvibrating steel-wheeled roller or a rubber-tired roller. Electronic, automatic fine graders may be used on soil-cement bases for pavements when very tight tolerances are required. For stairstepped embankment applications, several methods have been used to finish the exposed edges of each lift, including cutting back the uncompacted edges and using special attachments on compaction equipment (Fig. 6.10).

*Surface compaction planes are smooth areas left near the surface by the wheels of equipment or by motor grader blades. A thin surface layer of compacted soil cement may not adhere properly to these areas and in time may fracture, loosen, and spall. For good bond the base layer should be rough and damp.

6.5-Joints

When work stoppages occur for intervals longer than the specified time limits for fresh soil cement, transverse joints are trimmed to form straight vertical joints. This is normally done using the toe of a motor grader or dozer. Joints made in this way will be strong and will be easy to clean before resuming placement. When the freshly mixed soil cement is ready for placement against the construction joint, a check is made to assure that no dry or unmixed material is present on the joint edge. Retrimming and brooming may be necessary. Freshly mixed soil cement is then compacted against the construction joint. The fresh soil cement is left slightly high until final rolling, when it is trimmed to grade with the motor grader and rerolled. Joint construction requires special attention to insure that joints are vertical and that material in the joint area is adequately mixed and thoroughly compacted. For such multiple-layer constructions as stairstepped embankments, joints are usually staggered to prevent long continuous joints through the structure.

6.6-Curing and protection

Proper curing of soil cement is important because strength gain is dependent upon time, temperature, and the presence of water. Generally, a 3 to 7 day curing period is required, during which time equipment heavier than rubber-tired rollers is prohibited. Light local traffic, however, is often allowed on the completed soil cement immediately after construction, provided the curing coat is not damaged.

Water-sprinkling and bituminous coating are two popular methods of curing. Sprinkling the surface with water, together with light rolling to seal the surface, has proven successful. In bituminous curing, the soil cement is commonly sealed with an emulsified asphalt. The rate of application is dependent on the particular emulsion, but typically varies from 0.15 to 0.30 gal/yd^2. Before the bituminous material is applied, the surface of the soil cement should be moist and free of dry, loose material. In most cases, a light application of water precedes the bituminous coating. If traffic is allowed on the soil cement during the curing period, it is desirable to apply sand over the bituminous coating to minimize tracking of the bituminous material. Bituminous material should not be applied to any surfaces where bonding of subsequent soil-cement layers is required. Additionally, bituminous curing should not be applied on soil-cement linings for ponds or reservoirs which will be used to hold aquatic life.

Curing can also be accomplished by covering the compacted soil cement with wet burlap, plastic tarps, or moist earth.

Soil cement must be protected from freezing during the curing period. Insulation blankets, straw, or soil cover are commonly used.

7-QUALITY CONTROL TESTING AND INSPECTION

7.1-General

Quality control is essential to assure that the final product will be adequate for its intended use. Additionally, it must assure that the contractor has performed work in accordance with the plans and specifications. Field inspection of soil-cement construction involves controlling the following factors:

1. Pulverization/gradation
2. Cement content
3. Moisture content
4. Mixing uniformity
5. Compaction
6. Lift thickness and surface tolerance
7. Curing

References 48 and 51 provide excellent information on quality-control inspection and testing of soil cement during construction.

7.2-Pulverization (mixed in place)

Most soils require minimum pulverization before processing starts. However, the heavier clay soils require a considerable amount of preliminary work. The keys to pulverization of clayey soils are proper moisture control and proper equipment. Since clayey soils cannot be adequately pulverized in a central plant, their use is restricted to mixed-in-place construction.

PCA specifications[45,46] require that, at the completion of moist mixing, 80 percent of the soil-cement mixture pass the No. 4 sieve and 100 percent pass the 1-in. sieve, exclusive of gravel or stone retained on these sieves. This is checked by doing a pulverization test, which consists of screening a representative sample of soil cement through a No. 4 sieve. Any gravel or stone retained on the sieve is picked out and discarded. The clay lumps retained and the pulverized soil passing the No. 4 sieve are weighed separately and their dry weights determined. The degree of pulverization is calculated as follow[51]

$$\text{Percent pulverization} = \frac{\text{Dry weight of soil-cement mixture passing No. 4 sieve}}{\text{Dry weight of total sample exclusive of gravel retained on No. 4 sieve}} \times 100$$

Note that for practical purposes, wet weights of materials are often used instead of the corrected dry weights. The wet-weight measurements are reasonably accurate and permit immediate adjustments in pulverization and mixing procedures if necessary.

Fig. 7.1-Weighing cement collected on 1 yd² of canvas to check on quantity of cement spread

Pulverization can be improved by:

1. Slower forward speed of the mixing machine
2. Additional passes of the mixing machine
3. Replacing worn mixer teeth
4. Prewetting and premixing the soil before processing begins
5. Adding lime to highly plastic soils to reduce plasticity and improve workability.

Soil that contains excessive moisture will not mix readily with cement. The percentage of moisture in the soil at the time of cement application should be at or near optimum moisture content. Excess moisture may be reduced by additional pulverization and air drying, or in extreme cases by the addition of lime.

7.3-Cement-content control

7.3.1 *Mixed in place*-Cement is normally placed using bulk cement spreaders. A check on the accuracy of the cement spread is necessary to insure that the proper quantity is actually being applied. When bulk cement is being used, the check is made in two ways:

1. *Spot check*-A sheet of canvas, usually 1 yd² in area, is placed ahead of the cement spreader. After the spreader has passed, the canvas with cement is carefully picked up and weighed (Fig. 7.1). The spreader is then adjusted if necessary and the procedure repeated until the correct spread per yd² is obtained.

2. *Overall check*-The distance or area is measured over which a truckload of cement of known weight is spread. This actual area is then compared with the theoretical area, which the known quantity of cement should have covered.

Generally, the spreader is first adjusted at the start of construction after checking the cement spread per yd^2 with the canvas. Then slight adjustments are made after checking the distance over which each truckload is spread. It is important to keep a continuous check on cement-spreading operations.

On small jobs, bagged cement is sometimes used. The bags should be spaced at approximately equal transverse and longitudinal intervals that will insure the proper percentage of cement. Positions can be spotted by flags or markers fastened to ropes at proper intervals to mark the transverse and longitudinal rows.

7.3.2 *Central mixing plant*-In a central mixing-plant operation, it is necessary to proportion the cement and soil before they enter the mixing chamber. When soil cement is mixed in a batch-type pugmill or rotary-drum mixing plant, the proper quantities of soil, cement, and water for each batch are weighed before being transferred to the mixer. These types of plants are calibrated simply by checking the accuracy of the weight scales.

For a continuous-flow mixing plant, two methods of plant calibration may be used.

1. With the plant operating, soil is run through the plant for a given period of time and collected in a truck. During this same period, cement is diverted directly from the cement feeder into a truck or suitable container. Both the soil and cement are weighed and the cement feeder is adjusted until the correct amount of cement is discharged.

2. The plant is operated with only soil feeding onto the main conveyor belt. The soil on a selected length of conveyor belt is collected and its dry weight is determined. The plant is then operated with only cement feeding onto the main conveyor belt. The cement feeder is adjusted until the correct amount of cement is being discharged.

It may be necessary to calibrate the mixing plant at various operating speeds. Typically, plants are calibrated daily at the beginning of a project, and periodically thereafter, to assure that no change has occurred in the operation.

7.4-Moisture content

Proper moisture content is necessary for adequate compaction and for hydration of the cement. The proper moisture content of the cement-treated soil is determined by the moisture-density test (ASTM D 558 or D 1557). This moisture content, known as optimum moisture, is used as a guide for field control during construction. The approximate percentage of water added to the soil is equal to the difference between the optimum moisture content and the moisture content of the soil. About 2 percent additional moisture may be added to account for hydration of the dry cement and for evaporation that normally occurs during processing.

An estimate of the moisture content of a soil-cement mixture can be made by observation and feel. A mixture near or at optimum moisture content is just moist enough to dampen the hands when it is squeezed in a tight cast. Mixtures above optimum will leave excess water on the hands while mixtures below optimum will tend to crumble easily. If the mixture is near optimum moisture content, the cast can be broken into two pieces with little or no crumbling (Fig. 7.2). Checks of actual moisture content can be made daily, using conventional or microwave-oven drying.

During compaction and finishing, the surface of the soil-cement mixture may become dry, as evidenced by graying of the surface. When this occurs, very light fog-spray applications of water are made to bring the mois-

Fig. 7.2-Soil cement at optimum moisture casts readily when squeezed in the hand and can be broken into two pieces without crumbling

ture content back to optimum. Proper moisture content of the compacted soil cement is evidenced by a smooth, moist, tightly knit, compacted surface free of cracks and surface dusting.

7.5-Mixing uniformity

7.5.1 *Mixed in place*-A thorough mixture of pulverized soil, cement, and water is necessary to make high-quality soil cement. Where heavy clay soils are being treated, pulverization tests should be conducted prior to compaction as described in Section 7.2. The uniformity of all soil-cement mixtures is checked by digging trenches or a series of holes at regular intervals for the full depth of treatment and then inspecting the color of the exposed material. When the mixture is of uniform color and texture from top to bottom, the mixture is satisfactory. A mixture that has a streaked appearance has not been mixed sufficiently. Depth of mixing is usually checked at the same time as uniformity. Routine depth checks are made during mixing operations and following compaction to assure that the specified thickness is attained. Following compaction, a final check on mixing uniformity and depth can be made using a 2 percent solution of phenolphthalein. The phenolphthalein solution can be squirted down the side of a freshly cut face of newly compacted soil cement. The soil cement will turn pinkish-red while the untreated soil and subgrade material (unless it is calcium-rich soil) will retain its natural color.

7.5.2 *Central mix plant*- For central-plant-mixed soil cement, the uniformity is usually checked visually at the mixing plant. It can also be checked at the placement area in a manner similar to the method used for mixed-in-place construction. The mixing time necessary to achieve an intimate uniform mixture will depend on the soil gradation and mixing plant used. Usually 20 to 30 sec of mixing are required.

7.6-Compaction

The soil-cement mixture is compacted at or near optimum moisture content to some specified minimum percent of maximum density. Generally, the density requirements range from 95 to 100 percent of the maximum density of the cement-treated soil as determined by the moisture-density test (ASTM D 558 or D 1557). The most common methods for determining in-place density are:

1. Nuclear method (ASTM D 2922 and D 3017)
2. Sand-cone method (ASTM D 1556)
3. Balloon method (ASTM D 2167)

In-place densities are determined daily at frequencies that vary widely, depending on the application. The tests are made immediately after rolling. Comparing in-place densities with the results of maximum density results from the field moisture-density test indicates any adjustments in compaction procedures that may be required to insure compliance with job specifications.

7.7-Lift thickness and surface tolerance

7.7.1 *Lift thickness*- Compacted lift thickness is usually checked when performing field-density checks with the sand cone or the balloon method, or by digging small holes in the fresh soil cement to determine the bottom of treatment. Thickness can also be checked by coring the hardened soil cement. This provides a small diameter core for measuring thickness and for strength testing if required. Lift thickness is usually more critical for pavements than for embankment applications. For pavements, the U.S. Army Corps of Engineers typically tests thickness with a 3 in. diameter core for every 500 yd^2 of soil cement. Other agencies, such as Caltrans, require that thickness measurements be taken at intervals not to exceed 1000 linear ft.

7.7.2 *Surface tolerance*-Surface tolerances are usually not specified for soil-cement embankment applications, although lift elevation may be monitored with survey techniques. The U.S. Bureau of Reclamation controls only the soil-cement embankment crest road elevation to within 0.01 ft of design grade. To provide a reasonably smooth surface for pavement sections, smoothness is usually measured with a 10-ft or 12-ft straightedge, or with surveying equipment. The U.S. Army Corps of Engineers typically requires that deviations from the plane of a soil-cement base course not exceed 3/8 in. in 12 ft using a straightedge placed perpendicular to the centerline at about 50-ft intervals. Most state transportation departments limit the maximum departure from a 12-ft or 10-ft straightedge to about 3/8 in. In addition, a departure from design grade of up to 5/8 in. is usually allowed.

CONVERSION FACTORS

1 ft=0.305 m
1 in.=25.4 mm
1 lb=1.454 kg
1 mile=1.61 km
1 psi=6.895 kPa
1 lb/ft^3 = 16.02 kg/m^3
1 lb/yd^3=0.5933 kg/m^3
1 ft/sec =30.5 cm/sec
1 acre=0.4047 ha

8-REFERENCES

8.1-Specified references

The standards referred to in this document are listed below with their serial designation. The standards listed were the latest effort at the time this document was prepared. Since some of these standards are revised frequently, generally in minor detail only, the user of this document should check directly with the sponsoring group if it is desired to refer to the latest edition.

American Concrete Institute
207.5R-89 Roller Compacted Mass Concrete

ASTM	
C 42-87	Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete
C 150-89	Standard Specification for Portland Cement
C 595-86	Standard Specification for Blended Hydraulic Cements
C 618-89	Standard Specification for Fly Ash and Raw or Calcined Natural Pozzolan for Use as a Mineral Admixture in Portland Cement Concrete
D 558-82	Standard Test Method for Moisture-Density Relations of Soil-Cement Mixtures
D 559-82	Standard Methods for Wetting-and-Drying Tests of Compacted Soil-Cement Mixtures
D 560-82	Standard Methods for Freezing-and-Thawing Tests of Compacted Soil-Cement Mixtures
D 1556-82	Standard Test Method for Density of Soil in Place by the Sand-Cone Method
D 1557-78	Standard Test Methods for Moisture-Density Relations of Soils and Soil-Aggregate Mixtures Using 1 O-lb (4.54-kg) Rammer and 18-in. (457-mm) Drop
D 1632-87	Standard Methods of Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory
D 1633-84	Standard Test Method for Compressive Strength of Molded Soil-Cement Cylinders
D 1635-87	Standard Test Method for Flexural Strength of Soil-Cement Cylinders
D 2167-84	Standard Test Method for Density and Unit Weight of Soil in-Place by the Rubber Balloon Method
D 2901-82	Standard Test Method for Cement Content of Freshly Mixed Soil-Cement
D 2922-81	Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)
D3017-78	Standard Test Method for Moisture Content of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth)
D 4318-84	Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

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This report was submitted to letter ballot of the committee and approved in accordance with ACI balloting procedures.

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION
BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of	
PRIVATE FUEL STORAGE L.L.C.	Docket No. 72-22 ASLPB No. 97-732-02-ISFSI

Washington, D.C.

Friday, March 15, 2002

Deposition of

JAMES K. MITCHELL

a witness, called for examination by counsel for Private Fuel Storage, pursuant to notice and agreement of counsel, beginning at approximately 8:30 a.m., at the law offices of Shaw Pittman, 2300 N Street, NW., Washington, D.C., before Barbara A. Huber of Beta Reporting & Videography Services, notary public in and for the District of Columbia, when were present on behalf of the respective parties:

characteristic? Is that what you're saying?

A That's I think a reasonable way to put it. It's certainly possible to obtain a strength of 250 PSI. But to date, I have seen the results of -- I have not seen the results of any tests that show me that for this soil.

Q Again, I'm jumping way ahead, but we my go back to this. I provided to the state at their request earlier this week some preliminary test results of the program that PFS is conducting.

Have you seen those?

A Yes.

Q So you have seen those test results?

A I have.

Q Going back --

A That is, if it's the same set of results -- you know, there may -- I don't know how many sets of results there are, but I have seen one set of results.

report on soil cement in developing the site specific procedures for mixed portion and testing, construction and quality control?

A I'd have to go back and look at the guidelines in some detail to be sure whether you would follow them exactly in all respects. But I think that there's good guidance there, yes. The same kind of guidance are available through the Portland Cement Association publications and elsewhere.

Q Is it, in fact, your understanding that the state-of-the-art report on soil cement references all the publications, such as the Portland cement standard that you talked about?

A I think it does. References the AFTM standards that are often used.

Q Those would be the standards that you would expect somebody designing or constructing soil cement probably would be follow; is that correct?

A Yes, I believe so. Yes, I would.

Q As long as we are on that page, 26117, would you take a look at the discussion we just began looking? Take a second to read that, or more than a second. I would like you to take a look at that paragraph on page 26117, and then three paragraphs with bullets that go on page 26118 and 119. Take a second to review those. Let me know when you're finished.

A Okay.

Q Before we go into the specifics of this bullets, maybe it would be good for the record if we talked about your understanding of what PFS intends to do with soil cement.

Are you sufficiently familiar with what you understand to be their intents, so you can describe it for us?

A Do you want me to describe it?

Q Yes. If you could describe your understanding of what they're trying to do.

A They're using it in two ways, my

strengths of mixes, if you will, of soil cement underneath the pad, as opposed to elsewhere; is that correct?

A Yes.

Q So what is your understanding of the compressive strength, if you will, of soil cement that they want to use underneath the pads, as opposed to in other areas?

A My understanding if -- is that the strength of the treated soil beneath the pads is low. Is 40 PSI right, the right number? That the soil cement surrounding the building is stronger, 250 PSI.

Q The soil cement around the pads is also stronger?

A I don't remember on that but I -- I don't remember whether it's still the 40 or whether it's 250.

Q With that background, let's turn to the first bullet on page 26118.

In that paragraph, with the first bullet that is entitled, soil/cement mix and procedure development, the first paragraph of that entire section says, The sliding forces due to design bases ground motion will be resisted by bond between the base and sides of the foundation and the soil cement, and by passive resistance of the soil cement acting against the vertical side of the foundation. The soil cement mix will be designed and constructed to exceed the minimum shear resistance requirements.

Do you have any reason to believe that this approach as a technical proposition will not be successful if done properly?

A I don't have any reason to believe that it wouldn't be successful, no.

Q It goes on to say that there be direct shear testing conducted to replicate the soil conditions and to confirm the adequate shear resisting and other strength requirements will be provided by the final soil cement mix.

A My response to that is that's important, yes.

Q Isn't it true that if PFS performs durability tests as specified in Exhibit 14 that demonstrate that the mix that they propose to use passes or survives these durability tests, that that mixture would be qualified, in your opinion, as true soil cement?

A Yes.

Q If it doesn't, therefore it doesn't qualify as such?

A It would not.

Q But that's independent of whether the mixture that they intend to use achieves the strength that is specified?

A Yes.

Q You testified earlier that you see no problem with the ability to get the 250 PSI mix as such?

A My opinion is that it should be possible, but I would like to see it demonstrated.

Q Also you would like to see demonstrated that in addition to having 250 PSI, it meets the durability test?

A That's correct.

Q Let's move to paragraph 13 in your declaration.

It starts with, It is not surprising that no site specific testing has been done to date to obtain the strength and durability properties of the cement-treated soil.

Do you see that?

A I see that. But what I heard I don't believe is what I said.

Q Did I misread it?

A I believe you said it is not surprising. It's an important distinction. Because I said it is surprising.

Q If I did that, it was a Freudian slip, as they call it.

What I'm asking you, actually,

some thermal studies that would tell us.

Q If, in fact, there was some heat that was being moved downwards by the mechanism that we just described, then would that heat tend to move the moisture away the top layer or towards the top layer?

A I would expect it to move it away. I'd be very interested in seeing the thermal results of this. It's an interesting issue.

Q Of course, this is not something that you have analyzed to date?

A I have not analyzed. But I have, in the past, done both experimental and theoretical research on the heat flow around buried things.

Q This mechanism that I described to you is one that you have reason to believe its possible, or at least it's --

A Well, the heat transfer and the temperature. Oh, yes.

Q Now, let's go back to paragraph 14. Because I think in addition

Q What will your comments be on that particular issue?

A Well, we have no data to demonstrate what the modulus is at this point. If the material is a soil cement, I would be seriously concerned about whether the modulus could ever be that low. That's a very low value for soil cement.

But, also, as I think I understand it now, the rules of the game have changed a little bit since I first did this. The material beneath the pads will not necessarily be a soil cement. It will be a cement-treated soil.

I think at this point it's a question of: All right. For the cement treatment that you're now going to use or it's being proposed for use, will the modulus be within that design limit? To that question, I have no answer. Because I don't see any data.

Q Let's talk about that question.

First, as a technical engineering matter, is it within what is achievable, given the state-of-the-art, to build a cement-treated soil moisture will that have a Youngs modulus of 75,000 PSI or less?

A I can only say it potentially is. But it's going to be an issue of how much cement for this soil and what placement condition. Because the placement condition can be tremendously important in determining the strength and stiffness, as well as the cement content. It's at the low end of modulus values for this kind of a material, where we just don't have much data.

I was looking at information on this, and trying to see do we have good data points down in that modulus range. That's about where you go off the chart.

Q Now, assuming that, in fact, the design intent is carried out to have cement-treated soil with a strength of 40 PSI, do you believe that that's in the range of values that, subject to proven by testing, could yield a modulus of 70,000 PSI or less?

A I think it is potentially possible. I'm trying to remember a number. I think it might be in that ACI report, about modulus value is a function of cement content for fine grain soils. It's way down in the lower left corner.

Q How would you go, first of all, about testing the soil, the cement-treated soil mixture that you intend to use, to determine whether it meets the upper bound limitations of the Youngs modulus? What kind of test would you expected that would be performed?

A I think the -- I would test soil from the site over a range of proposed cement and water contents. I would have specimens -- cured specimens, for which I could determine both the strength and the modulus.

There are different ways that you can get the modulus: From strength test, from some dynamic tests that are possible. Then you simply -- you have to find a condition that will give you this strength, which is 40 PSI compressor strength; and for those materials, what range of conditions will give you a modulus that is less than 75,000 PSI.

Q That would you determine through a testing program under the lines that you talked about?

A Testing program, yes.

Q Now, as to the second part -- which I thought that you mentioned as being pretty important -- what do you mean by placement conditions?

What is it that you would like to see in order to assure yourself that even if you have been able to through testing to determine that you have a cement-treated soil mix that emits a 75,000 PSI limit, what surrounding field, the strain will be considerably less. So a comprehensive response of all that would have to take all that into account.

Q Fair enough. Let me ask you a more general question. We have been talking about the various issues that you have identified in your declaration, and in subsections C and D of Contention QQ, subsection C and D of part C of Contention QQ.

Would it be fair to characterize your responses as indicating that many of these issues are in the nature of things that you would like to see proved through testing, as opposed to being unachievable technically?

A Yes.

MR. TRAVIESO-DIAZ: I have nothing else.

MR. TURK: I may have none, or very, very little.

2.6.4.11 Techniques to Improve Subsurface Conditions

Soil Cement

Discussions presented in Section 2.6.1.12, above, indicate that the soils underlying the eolian silt layer at the surface of the PFSF site are suitable for support of the proposed structures; therefore, no special construction techniques are required for improving the subsurface conditions below the eolian silt. The eolian silt, in its *in situ* loose state, is not suitable for founding the structures at the site. The basemat of the Canister Transfer Building will be founded on the silty clay/clayey silt layer beneath the eolian silt. It was originally intended that the cask storage pads also would be founded on the silty clay/clayey silt layer. However, instead of excavating the eolian silt from the pad emplacement area and replacing it with suitable structural fill, it will be mixed with sufficient portland cement and water and compacted to form a strong soil-cement subgrade to support the cask storage pads. Soil cement will also be utilized around the Canister Transfer Building. The required characteristics of the soil cement will be engineered during detailed design and constructed to meet the necessary strength requirements.

During construction of the storage pads, all of the eolian silt in the quadrant under construction will be excavated. The eolian silt will be mixed with sufficient cement and water and compacted to produce soil cement across the pad area, up to the design elevations of the bottoms of the storage pads. The layer of soil cement beneath the storage pads will have a minimum thickness of 12 inches and a maximum thickness of 24 inches. In the event that the eolian silt layer extends to a depth greater than 2 ft below the elevations of the bottoms of the storage pads, compacted clayey soils will be used to raise the elevation of the subgrade that will support the soil cement layer to an elevation of 2 ft or less below the design elevations of the bottoms of the pads. This will ensure that the layer of soil cement does not exceed a thickness of 2 ft. This is the maximum permissible thickness of the soil cement layer, since the storage cask hypothetical tipover and drop analyses were performed assuming a 2.0-ft thick layer of soil cement underlying the storage pads.

Strength of Soil Cement and Minimum/Maximum Thickness Requirements

The soil cement underlying the pads shall have a minimum unconfined compressive strength of 40 psi to ensure that there is an adequate factor of safety against sliding of an entire column of pads (S&W Calculation 05996.02-G(B)-4, SWEC, 2001b). This layer of soil cement is required to be no greater than 2-ft thick and have a static modulus of elasticity less than or equal to 75,000 psi to ensure that the decelerations from a hypothetical storage cask tipover event or vertical end drop accident do not exceed HI-STORM design criteria (Section 3.2.11.3).

Following construction of the storage pads on top of this layer of soil cement, additional soil cement will be placed around and between the cask storage pads, extending from the bottoms of the pads to a level that is 28 inches above the bottoms of the storage pads. The remaining 8 inches, from the top of the soil cement up to grade, will be filled with coarse aggregate, placed and compacted to be flush with the tops of the pads to permit easy access by the cask transporter. The soil cement placed around the sides of the storage pads is expected to have a minimum unconfined compressive strength of at least 250 psi to satisfy durability requirements within the depth of frost penetration (based on S&W Calculation 05996.02-G(B)-4 (SWEC, 2001b), as discussed in Section 2.6.1.12.1).

The Canister Transfer Building basemat will be founded on the silty clay/clayey silt layer that is below the eolian silt. The design calls for soil cement to be placed around the Canister Transfer Building base mat to make the free-field soil profile for the building consistent with that for the storage pad emplacement area and to help resist sliding forces due to the higher design basis ground motions. Soil cement will surround the foundation mat and will extend outward from the mat to a distance equal to the associated mat dimension; i.e., approximately 240 ft out from the mat in the east and west directions and approximately 280 ft out in the north and south directions. Existing soils (eolian silt and silty clay/clayey silt) will be excavated to a depth of approximately 5 ft 8 inches below grade, mixed with cement, and placed and compacted around the foundation mat.

The soil cement placed around the Canister Transfer Building foundation mat will be 5 ft thick and have a minimum unconfined compressive strength of 250 psi to ensure that there is an adequate factor of safety against sliding of the Canister Transfer Building (based on Calculation 05996.02-G(B)-13 (SWEC, 2001c), as discussed in Section 2.6.1.12.2). The top 8 inches will be filled with compacted coarse aggregate, similar to that used in the pad emplacement area.

PFS is developing the soil-cement mix design using standard industry practice. This effort includes performing laboratory testing of soils obtained from the site. This on-going laboratory testing is being performed in accordance with the requirements of Engineering Services Scope of Work (ESSOW) for Laboratory Testing of Soil-Cement Mixes, ESSOW 05996.02-G010 (SWEC, 2001e). This program includes measuring gradations and Atterberg limits of samples of the near-surface soils obtained from the site. It includes testing of mixtures of these soils with varying amounts of cement and the testing of compacted specimens of soil-cement to determine moisture-density relationships, freeze/thaw and wet/dry characteristics, compressive and tensile strengths, and permeability of compacted soil-cement specimens. The entire laboratory testing program is being conducted in full compliance with the Quality Assurance (QA) Category I requirements of the ESSOW.

As part of this effort, PFS is performing so-called durability testing. These tests are performed in accordance with ASTM D559 and D560 to measure the durability of soil cement specimens exposed to 12 cycles of wet/dry and freeze/thaw conditions. As indicated on p. 16 of PFS Calculation 05996.02-G(B)-4 (SWEC, 2001b):

> *"The unconfined compressive strength of the soil cement adjacent to the pads needs to be at least 50 psi to provide an adequate subbase for support of the cask transporter, in lieu of placing and compacting structural fill, but it likely will be at least 250 psi to satisfy the durability requirements associated with environmental considerations (i.e., freeze/thaw and wet/dry cycles) within the frost zone (30 in. from the ground surface)."*

PFS is performing these tests to determine the amounts of cement and water that must be added to the site soils and to determine the compaction requirements to ensure that the soil cement will be durable and will withstand exposure to the elements. As indicated on p. 8 of Portland Cement Association (1971):

"The freeze-thaw and wet-dry tests were designed to determine whether the soil-cement would stay hard or whether expansion and contraction on alternate freezing-and-thawing and moisture changes would cause the soil-cement to soften."

And on p. 32:

"The principle requirement of a hardened soil-cement mixture is that it withstand exposure to the elements. Thus the primary basis of comparison of soil-cement mixtures is the cement content required to produce a mixture that will withstand the stresses induced by the wet-dry and freeze-thaw tests. The service record of projects in use proves the reliability both of the results based on these tests and of the criteria given below.

The following criteria are based on considerable laboratory test data, on the performance of many projects in service, and on information obtained from the outdoor exposure of several thousand specimens. The use of these criteria will provide the minimum cement content required to produce hard, durable soil-cement, suitable for base-course construction of the highest quality.

1. *Soil-cement losses during 12 cycles of either the wet-dry test or freeze-thaw test shall conform to the following limits:*

   Soil Groups A-1, A-2-4, A-2-5, and A-3, not over 14 percent;
   Soil Groups A-2-6, A-2-7, A-4, and A-5, not over 10 percent;
   Soil Groups A-6 and A-7, not over 7 percent.

2. *Compressive strengths should increase both with age and with increases in cement content in the ranges of cement content producing results that meet requirement 1."*

The on-going laboratory testing program will also include additional tests to confirm that the bond at the interfaces between concrete and soil-cement, soil-cement and soil-cement, and soil-cement and the site soils will exceed the strength of the in situ clayey soils. These tests will include direct shear tests, performed on specimens prepared from the site soils at various cement and moisture contents, in a manner similar to that used by DeGroot in his testing of bond along soil-cement interfaces.

Based on the above, PFS has adequately defined the measures that will be followed in the design and construction of the soil cement to assure that the assumed bonds can be sustained through the period of interest. PFS has committed to performing site-specific testing to confirm that the required interface strengths are available to resist sliding forces due to an earthquake. As indicated above, this testing will include direct shear tests to be performed in the laboratory in the near-term (pre-construction) during the soil-cement mix development to demonstrate that the required interface strengths can be achieved and during construction to demonstrate that the required interface strengths are achieved. In addition, PFS has committed to augmenting this field testing program by performing additional site-specific testing of the strengths achieved at the interface between the bottom of the soil cement and the underlying soils.

The most recent analyses of the PFSF design basis ground motions assumed the incorporation of a 5 ft thick soil cement layer over the entire pad emplacement area and also surrounding the Canister Transfer Building. The 5 ft soil cement layer around the Canister Transfer Building extends to the free field boundary from the edge of the building basemat. This soil cement layer is assumed to have a minimum shear wave velocity greater than 1,500 fps (Geomatrix 2001a and 2001b). As indicated in Section 2.6.1.2.2, soil cement around the Canister Transfer Building should have a minimum unconfined compressive strength of 250 psi to ensure a factor of safety greater than 1.1 for seismic sliding stability. The design requirements for the 5 ft thick soil cement layer around the Canister Transfer Building will be based on the results of laboratory and field testing to be conducted during the final design stage.

The surficial layer of eolian silt, existing across the entire site as shown in the pad emplacement area foundation profiles (Figure 2.6-5, Sheets 1 through 14), is a major factor in the earthwork required for construction of the facility. This layer consists of a nonplastic to slightly plastic silt, and it has an average thickness of approximately 2 feet across the pad emplacement area. This layer was expected to be removed prior to construction of the storage pads. However, based on evaluation of the earthwork associated with site grading requirements for flood protection and the environmental impacts of truck trips required to import fill to replace this material, PFS will stabilize this soil with cement and use it as base material beneath the storage pads and adjacent driveways.

Section 2.6.1.12 indicates that there is ample margin in the factor of safety against a bearing capacity failure of the silty clay/clayey silt underlying the site and that the settlements are acceptable for these structures. They indicate that the critical design factor with respect to stability of these structures is the resistance to sliding due to loadings from the design basis ground motion. As discussed in that section, the silty clay/clayey silt layer has sufficient strength to resist these dynamic loadings; therefore, adequate sliding resistance can be provided by constructing the structures directly on the silty clay/clayey silt layer. The soil cement around the storage pads and Canister Transfer Building will be designed and constructed to have a minimum unconfined compressive strength of 250 psi and quality assurance testing will be performed during construction to demonstrate that this minimum strength is achieved. The soil cement directly beneath the storage pads will be designed and constructed to have an unconfined compressive strength of at least 40 psi with static elastic modulus of less than ~75,000 psi. Therefore, the resistance to sliding due to loadings from the design basis ground motion will be enhanced by constructing the cask storage pads on a properly designed and constructed soil-cement subgrade. See the section titled "Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement" in 2.6.1.12.1 for additional details.

Using soil cement to stabilize the eolian silt will reduce the amount of spoil materials generated, create a stable and level base for pad construction, and substantially improve the sliding resistance of the storage pads. The soil cement will be placed above the *in situ* silty clay/clayey silt layer and will be designed to improve the strength of the eolian silt so that it will be stronger than the clayey soils that were originally intended for use as the founding medium for the pads. The soil cement will also be used to replace the compacted structural fill that the original plan included between the rows of pads. This continuous layer of soil cement, existing under and between the pads, will spread the loads from the pads beyond the footprint of the pads, resulting in decreased total and differential settlements of the pads. The layer of soil cement above the base of the pads and the bond and friction of the pad foundation with the underlying soil-cement layer will greatly increase the sliding resistance of the pad.

Soil cement has been used extensively in the United States and around the world since the 1940's. It was first used in the United States in 1915 for constructing roads. It also has been used at nuclear power plants in the United States and in South Africa. The largest soil-cement project worldwide involved construction of soil-cement slope protection for a 7,000-acre cooling-water reservoir at the South Texas Nuclear Power Plant near Houston, TX. Soil cement also was used to replace an ~18-ft thick layer of potentially liquefiable sandy soils under the foundations of two 900-MW nuclear power plants in Koeberg, South Africa (Dupas and Pecker, 1979). The strength of soils can be improved markedly by the addition of cement. The eolian silt at the site is similar to the soils identified as Soil A-4 in Nussbaum and Colley (1971), Soils 7 and 8 in Balmer

(1958), and Soil 4 in Felt and Abrams (1957). As indicated for Soil A-4 in Table 5 of Nussbaum and Colley (1971), the addition of just 2.5% cement by weight to the silt increased the cohesion from 5 psi (720 psf) to 30 psi (4,320 psf). The cohesion for Soils 7 and 8 also were increased significantly by the addition of low percentages of cement, as shown on Tables VI and VII of Balmer (1958). Figure 10 in Felt and Abrams (1957) illustrates the continued strength increase over time for these soil-cement mixtures. Other examples of soil-cement strength increases over time are presented in Figure 4.3 of ACI (1998), Table 6 of Nussbaum and Colley (1971), and Figures 6 and 7 of Dupas and Pecker (1979). Therefore, the soil cement will be much stronger than the underlying silty clay/clayey silt and the strength will increase with time, providing an improved foundation material. This will provide additional margin against sliding compared to the original plan to construct the pads directly on the silty clay/clayey silt layer.

As shown in the section titled "Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement" in Section 2.6.1.12.1 above, the shear resistance required at the base of the pads can be provided easily by the passive resistance of the soil cement acting against the vertical side of the foundation and by bond between the pad foundation and soil-cement contact and the cohesive strength of the soil cement. Shear resistance will be transferred through the approximately 2-ft thick soil-cement layer and into the underlying silty clay/clayey silt subgrade. Additional resistance will be provided by the continuous layer of soil cement under and between the pads; therefore, shear resistance requirements within the silty clay/clayey silt layer will be less with the soil-cement layer compared to the original plan to construct the pads directly on the silty clay/clayey silt without the proposed soil-cement layer.

DeGroot (1976) indicates that this bond strength can be easily obtained between layers of soil cement. He performed nearly 300 laboratory direct shear tests to determine the effect of numerous variables on the bond between layers of soil cement. These variables included the length of time between placement of successive layers of soil cement, the frequency of watering while curing soil cement, the surface moisture condition prior to construction of the next lift, the surface texture prior to construction of the next lift, and various surface treatments and additives.

His results demonstrated that, with the exception of treating the surface of the lifts with asphalt emulsion, asphalt cutback, and chlorinated rubber compounds, the bond strength always exceeded 6.6 psi, the minimum required value of cohesion if the passive resistance acting on the sides of the pads is ignored. The minimum bond strength he reports, other than for the asphalt and chlorinated rubber surface treatments identified above, is 8.7 psi. This value applied for two tests that were performed on samples that had time delays of 24 hours and did not have a cement surface treatment along the lift line. He reports that nearly all of the specimens that used a cement surface treatment broke along planes other than along the lift lines, indicating that the bond between the layers of soil cement was stronger than the remainder of the specimens. Excluding the specimens that had 24-hr delays between lift placements and which did not use the cement surface treatment, the minimum bond strength was 10.7 psi and there were only two others that had bond strengths that were less than 20 psi. Even these minimum values for the group of specimens that did not use a cement surface treatment exceeded the cohesive strength (6.6 psi) required to obtain an adequate factor of safety against sliding without including the passive resistance acting on the sides of the pads, and all of the rest were much greater, generally more than an order of magnitude greater.

DeGroot reached the following conclusions:

1. Increasing the time delay between lifts decreases bond.
2. High frequency of watering the lift line decreases the bond.

3. Moist curing conditions between lift placements increases the bond.
4. Removing the smooth compaction plane increases the bond.
5. Set retardants decreased the bond at 4-hr time delay.
6. Asphalt and chlorinated rubber curing compounds decreased the bond.
7. Small amounts of cement placed on the lift line bonded the layers together, such that failure occurred along planes other than the lift line, indicating that the bond exceeded the shear strength of the soil cement.

DeGroot (1976) noted that increasing the time delay between placement of subsequent lifts decreases the bond strength. The nature of construction of soil cement is such that there will be occasions when the time delay will be greater than the time required for the soil cement to set. This will clearly be the case for construction of the concrete storage pads on top of the soil-cement surface, because it will take some period of time to form the pad, build the steel reinforcement, and pour the concrete. He noted that several techniques can be used to enhance the bond between these lifts to overcome this decrease in bond due to time delay. In these cases, more than sufficient bond can be obtained between layers of soil cement and between the set soil-cement surface and the underside of the cask storage pads by simply using a cement surface treatment.

DeGroot's direct shear test results demonstrate that the specimens having a cement surface treatment all had bond strengths that ranged from 47.7 psi to 198.5 psi, with the average bond strength of 132.5 psi. Even the minimum value of this range is nearly an order of magnitude greater than the cohesion (6.6 psi) required to obtain a factor of safety against sliding of 1.1, conservatively ignoring the passive resistance available on the sides of the pads. Therefore, when required due to unavoidable time delays, the techniques DeGroot describes for enhancing bond strength will be used between the top of the soil cement and succeeding lifts or the concrete cask storage pads, to assure that the bond at the interfaces are greater than the minimum required value. These techniques will include roughening and cleaning the surface of the underlying soil cement, proper moisture conditioning, and using a cement surface treatment.

A fundamental assumption in the PFS approach is that sufficient bonding and shear transfer between clay and soil cement interfaces can be achieved using various construction techniques. As indicated above, DeGroot has demonstrated that techniques are available that will enhance the bond between lifts of soil cement. These techniques should be equally effective when applied to the soils at the PFSF site. PFS has committed to perform direct shear tests of the interface strengths during the design phase of the soil cement to demonstrate that the required interface strength can be achieved, as well as during construction, to demonstrate that they are achieved.

PFS has discussed the change to use soil cement beneath the storage pads with the project consultants who have analyses in-place that are based on the storage pads resting on the silty clay/clayey silt. The consultants contacted were Geomatrix (development of seismic criteria and soil dynamic properties), Holtec International (cask stability analysis), and International Civil engineering Consultants (pad design). Each has indicated their analyses would not be adversely affected by this proposed change.

The design, placement, testing, and performance of soil cement is a well-established technology. The "State-of-the-Art Report on Soil Cement" (ACI, 1998) provides information about soil cement, including applications, materials, properties, mix proportioning, design, construction, and quality-control inspection and testing techniques. PFS will develop site-specific procedures to implement the recommendations presented in ACI (1998) regarding mix proportioning, testing, construction, and quality control. The following describes the processes that will be used to develop a proper soil-cement mix design and establish adequate sliding resistance at each material interface in the storage pad and soil system:

- Soil-Cement Mix and Procedure Development – The sliding forces due to the design basis ground motion will be resisted by bond between the base and sides of the foundation and the soil cement and by passive resistance of the soil cement acting against the vertical side of the foundation. The soil-cement mix will be designed and constructed to exceed the minimum shear resistance requirements. During the soil-cement design phase, direct shear testing will be conducted along manufactured soil-cement lift contacts and concrete contacts that represent anticipated field conditions. The direct shear testing, along with other standard soil-cement testing, will be used to confirm that adequate shear resistance and other strength requirements will be provided by the final soil-cement mix design. Procedures required for placement and treatment of the soil cement, lift surfaces, and foundation contact will be established in accordance with the recommendations of ACI (1998) during the mix design and testing process. Specific construction techniques and field quality control requirements will be identified in the construction specifications developed by PFS during this detailed design phase of the project.

- Soil-Cement Lift and Concrete Interface – The soil cement will be constructed in lifts approximately 6-in. thick (compacted thickness) as described in ACI (1998). Construction techniques will be used to ensure that the interface between the soil-cement layers will be adequately bonded to transmit shear stresses. As described in Section 6.2.2.5 of ACI (1998), these techniques will include, but will not be limited to: minimizing the time between placement of successive layers of soil cement, moisture conditioning required for proper curing of the soil cement, producing a roughened surface on the soil cement prior to placement of additional lifts or concrete foundations, and using a dry cement or cement slurry to enhance the bonding of concrete or new soil cement layers to underlying layers that have already set. In addition to conventional quality control testing performed for soil-cement projects, direct shear testing will be performed on representative samples obtained from placed lift contacts to confirm design requirements are obtained. Sacrificial soil-cement lifts may be used to protect the soil-cement subgrade in the pad foundation areas.

- Soil Cement and *In Situ* Clay Interface – The soil cement and *in situ* clay interface will be constructed such that a good bond will be established between the two materials. Construction techniques will be utilized that will ensure that the integrity of the upper surface of the clay is maintained and that a good interface bond between the two materials is obtained. Specific construction techniques and field quality control requirements will be identified in the construction specifications developed by PFS during the detailed design phase of the project.

An additional benefit of incorporating the soil cement into the design is that it will minimize the environmental impacts of constructing the facility. Using on-site materials to construct the soil cement, rather than excavating and spoiling those materials, will reduce environmental impacts of the project. In addition, replacement of some of the structural fill layer between the rows of pads with soil cement, as shown in Figure 4.2-7, will result in reduced trucking requirements associated with transporting those materials to the site.

Adequacy of the Soil Cement Design

The adequacy of the design of the soil cement surrounding and underlying the pads to ensure the sliding stability of the pads under seismic conditions is demonstrated by S&W Calculation 05996.02-G(B)-04 (SWEC, 2001b). This calculation determined that there is sufficient shear strength at the interfaces between the concrete pad and the underlying soil cement and between that soil cement layer and the underlying clayey soils that the factor of safety against sliding exceeds the minimum required value, with no credit for the soil cement placed between storage pads above the bottom of the pads. The underlying layer of soil cement is also required to have a static modulus of elasticity less than or equal to 75,000 psi to ensure that decelerations of a cask resulting from a hypothetical storage cask tipover event or vertical end drop accident do not exceed design criteria (Sections 4.2.1.5.1.E and 8.2.6).

The large extent of soil cement in the storage pad emplacement area allows the soil cement layer to be considered as part of the free field soil profile for the site response analyses. The properties of the soil cement, higher shear wave velocity and higher density than the existing soils in the area, help to minimize the response at the surface of the site caused by the design basis ground motions. Soil cement was added around the Canister Transfer Building foundation mat to make the free field soil profile for the building consistent with that for the storage pad emplacement area (as discussed in Section 2.6.4.11), and to help resist sliding forces, in conjunction with the building's perimeter key, due to the revised design basis ground motions. The adequacy of this design feature is demonstrated in Calculation No. 05996.02–G(B)-13 (SWEC, 2001c), which determined that the design of the soil cement surrounding the Canister Transfer Building (in conjunction with the building's perimeter key) is adequate to ensure the stability of the Canister Transfer Building under seismic conditions.

2.6.4.12 Criteria and Design Methods

The allowable bearing capacity of footings is limited by shear failure of the underlying soil and by footing settlement. The minimum factor of safety against a bearing capacity failure from static loads (dead load plus maximum live loads) is 3.0 and from static loads plus loads due to extreme environmental conditions, such as design basis ground motion, is 1.1. Allowable settlements are determined based on Table 14.1, "Allowable Settlement," of Lambe & Whitman (1969) and assume that the differential settlement will be 3/4 of the maximum settlement. Section 2.6.1.12 provides more details.