RAS 3386

September 7, 2001

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

DOCKETED
USNRC

Before the Atomic Safety and Licensing Board

September 12, 2001 (1:42PM)

OFFICE OF SECRETARY
RULEMAKINGS AND
ADJUDICATIONS STAFF

In the Matter of	)	
	)	
PRIVATE FUEL STORAGE L.L.C.	)	Docket No. 72-22
	)	
(Private Fuel Storage Facility)	)	ASLBP No. 97-732-02-ISFSI
	)	

APPLICANT'S RESPONSE TO STATE OF UTAH'S SECOND REQUEST TO MODIFY THE BASES OF LATE-FILED CONTENTION UTAH QQ IN RESPONSE TO MORE REVISED CALCULATIONS FROM THE APPLICANT

I. INTRODUCTION

Applicant Private Fuel Storage L.L.C. ("Applicant" or "PFS") hereby responds to the State of Utah's ("State") "Second Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to More Revised Calculations from the Applicant," filed August 23, 2001 ("Second Request"). In this, its second attempt to widen the bases of its Proposed Contention QQ, the State asserts that revised calculations filed by Applicant on July 27 and August 7, 2001 in response to NRC Staff's requests for additional information require the modification of the bases of proposed Contention Utah QQ.[1] The modifications that the State seeks to make to Proposed Utah QQ, however, do not constitute admissible new contentions.

The "new" claims propounded by the State in its Second Request are in many cases not new, since they are restatements of allegations previously made in Proposed Utah QQ or in the State's First Request. Indeed, the State acknowledges in numerous places that various allegations it seeks to raise in its Second Request were previously set forth in Proposed Utah QQ and its First Request and that certain claims have been of "long-standing dispute between PFS and

---

1 See State of Utah's Request for Admission of Late-Filed contention Utah QQ (Seismic Stability), dated May 16, 2001 ("Proposed Utah QQ"); State of Utah's Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to Further Revised Calculations from the Applicant, dated June 19, 2001 ("First Request").

Template = SECY-041

SECY-02

the State."[2] While the State raises twelve issues in its Second Request, six of the issues have been raised before.[3] These repetitious claims are inadmissible both because they are duplicative and for the reasons stated in Applicant's oppositions to the admission of Proposed Utah QQ[4] and the State's First Request.[5]

Four of the six remaining claims, while not raised previously, are clearly untimely in that they challenge methodologies (e.g., the use of the Newmark method for estimating sliding displacement) that have been used in PFS's geotechnical calculations for a year or more prior to the filing of the State's Second Request. The remaining issues, while new, are either clearly erroneous or irrelevant. Thus, the six claims in the Second Request that are asserted for the first time in that document do not meet the requirements for late-filed contentions, or the admission of contentions, or both.

Accordingly, the State's Second Request should be denied.

II. BACKGROUND

On December 16, 1999, Applicant filed License Application ("LA") Amendment No. 8 ("LA 8"). This amendment incorporated soil cement into the design of the PFS facility, to be used beneath and around the spent fuel cask storage pads. The amendment to the Safety Analysis Report ("SAR") filed with LA 8 included specifications for the soil cement, calling for the use of American Concrete Institute ("ACI") standards to govern the placement and treatment of the soil cement.[6] The calculations for the sliding stability of the cask storage pads under seismic loads were also revised to incorporate the increased stability afforded by the use of soil cement.[7]

---

2 See, e.g., Second Request at 4.

3 See Exhibit 1 hereto, which is a matrix summarizing the prior history of the claims asserted by the State in its Second Request.

4 Applicant's Response to State of Utah's Request for Admission of Late-Filed Contention Utah QQ, dated May 30, 2001 ("PFS's Response to Proposed Utah QQ").

5 Applicant's Response to State of Utah's Request to Modify the Bases of Late-Filed Contention QQ in Response to Further Revised Calculations from the Applicant, dated July 3, 2001 ("PFS's Response to First Request").

6 SAR at 2.6-91 (Rev. 8). See Exhibit A to PFS's Response to Proposed Utah QQ, item 15 for further details.

7 License Amendment No. 9, submitted on February 2, 2000. See Exhibit A to PFS's Response to Proposed Utah QQ, item 1.

On June 23, 2000, Applicant submitted LA Amendment No. 13 ("LA 13"), which revised certain seismic design calculations to take into account the effects of soil cement in increasing the storage pad stability and the operation of the soil cement as a "dynamic buttress."[8] The revised calculations submitted with LA 13 continued to apply the same assumptions and methods as the previous versions of the calculations.[9]

On March 30, 2001, PFS filed LA Amendment 22 ("LA 22"), which provided revised design basis ground motions derived from the use of additional soils data.[10] On April 26, 2001, the Board issued an order setting May 16, 2001, as the due date for a State submission of a proposed contention regarding "(CTB) design, including use of soil cement, or revisions to storage pad analyses, soils analyses, soil-cement design calculations/analyses, and Holtec site-specific cask analyses."[11]

On May 16, 2001, the State filed its request to admit Proposed Utah QQ. On May 30, 2001, Applicant filed its Response to Proposed Utah QQ. PFS opposed the admission of Proposed Utah QQ, as generally did the NRC Staff.[12]

At approximately the same time, the NRC Staff advised PFS that the Staff could not determine a schedule for completion of its review of LA 22 until certain information was provided including, *inter alia*:

---

[8] PFS letter, Donnell to NRC, "Submittal of Commitment Resolution Letter No. 3 Information," dated June 19, 2000. Calculation No. 05996.02-G(B)-4, Rev. 6, which was part of the package, incorporated the "buttress" effect of soil cement. See Exhibit A to PFS's Response to Proposed Utah QQ, item 12.

[9] SWEC Calculation No. 05996.02-G(B)-4 examines the stability of the storage pads, whereas SWEC Calculation No. 059906.02-G(B)-13 examines the stability of the Canister Transfer Building "(CTB"). The two calculations employ the same methodology in the areas challenged by the State in Proposed Utah QQ. See Attachments 1 and 2 to Exhibit A to PFS's Response to Proposed Utah QQ.

[10] PFS letter, Parkyn to NRC dated March 30, 2001 and attachments thereto. At the time LA Amendment 22 was submitted, PFS issued revised versions of SWEC Calculation No. 05996.02-G(B)-4 (Revision 7) ("Cask Storage Pad Stability Calc. Rev. 7") and SWEC Calculation No. 059906.02-G(B)-13 (Revision 4) ("Canister Transfer Building Stability Calc. Rev. 4").

[11] Memorandum and Order (Schedule for Late-Filed Submissions Regarding License Application Amendment and Page Limit Extension) (April 26, 2001) at 2.

[12] NRC Staff's Response to "State of Utah's Request for Admission of Late-Filed Contention Utah QQ (Seismic Stability)," dated May 30, 2001.

> Revised analyses of the stability of the storage pads to include a clear identification of the potential failure modes and failure surfaces and the material strengths required to satisfy the regulatory requirement, considering the critical failure modes and failure surfaces. [13]

On May 31, 2001, PFS submitted to the NRC Staff Revision 8 to Calculation No. 05996.02-G(B)-04 ("Cask Storage Pad Stability Calc. Rev. 8") and Revision 5 to Calculation No. 05996.02-G(B)-13 ("Canister Transfer Building Stability Calc. Rev. 5"). These revised calculations provided the additional description requested by the Staff, but did not change the underlying analyses and assumptions in the two calculations from previous revisions.[14]

On June 19, 2001, the State filed its First Request to modify the bases of Proposed Utah QQ based on the revisions PFS made to the two above calculations in response to the NRC Staff's request for information. On July 3, 2001, PFS filed its Response to the State's First Request, opposing the State's request to modify Proposed Utah QQ. The NRC Staff also generally opposed the First Request.[15]

On June 20, 2001, the NRC Staff had advised PFS that, although PFS had addressed each of the items identified in the NRC's May 7 letter, the NRC had determined that additional information was needed from PFS for the Staff to complete its review of LA 22.[16] PFS responded to the NRC's request for additional information by providing further revised calculations. On July 27, 2001, PFS submitted to the NRC Staff Revision 9 to Calculation No. 05996.62-G(B)-04 ("Cask Storage Pad Stability Calc. Rev. 9") and Revision 6 to Calculation No. 05996-02-G(B)-13 ("Canister Transfer Stability Calc. Rev. 6"). Cask Storage Pad Stability Calc. Rev. 9 added a hypothetical analysis of the potential sliding of the cask storage pad in a seismic event under

[13] Letter dated May 7, 2001 from E. William Brach (NRC) to John D. Parkyn (PFS), Attachment, "Data Needed for the Completion of the PFS LA Amendment," "Soil Engineering" Section, item 3, attached as Enclosure 2 to letter dated June 20, 2001 from Sherwin Turk (NRC Staff Counsel) to Licensing Board and Parties.

[14] Declaration of Paul J. Trudeau, dated July 3, 2001 ("Trudeau 1st Dec.") ¶ 6.

[15] NRC Staff's Response to "State of Utah's Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to Further Revised Calculations from the Applicant," dated July 3, 2001.

[16] Letter dated June 20, 2001 from Mark S. Delligatti (NRC) to John D. Parkyn (PFS), attached as Enclosure 4 to letter dated June 20, 2001 from Sherwin Turk (NRC Staff Counsel) to Licensing Board and Parties.

"obviously conservative" assumptions. Declaration of Paul J. Trudeau, dated September 6, 2001 ("Trudeau 2nd Dec.") ¶ 6. On August 7, Applicant filed an analysis performed by Holtec of the effect of hypothetical sliding of the pad on Holtec's cask stability analysis. Again, these additional analyses were not intended to, and did not, change the results of previous calculations, but merely responded to the NRC Staff's requests for further information. Id. ¶ 10.

On August 23, 2001, the State filed its Second Request, seeking once more to modify the bases of Proposed Utah QQ. On August 30, 2001, the Board issued an Order setting September 7, 2001 as the date for filing responsive pleadings to the State's Second Request.[17] The instant response is being filed pursuant to the Board's Order.

III. LEGAL STANDARDS

The State's Second Request is concededly late. Thus, in order for its request to amend Proposed Utah QQ to be granted, the State must satisfy the requirements for the admission of late-filed contentions. Pursuant to 10 C.F.R. § 2.714(b)(1), late-filed contentions are admissible only if a balancing of the five factors listed in 10 C.F.R. § 2.714(a)(1) supports admission of the contention. Those five factors are: (i) good cause, if any, for the failure to file on time, (ii) the availability of other means to protect the petitioner's interest, (iii) the extent to which petitioner will assist in the development of a sound record, (iv) the extent to which the petitioner's interest will be represented by other parties, and (v) the extent to which admitting the contention will broaden the issues or delay the proceeding. 10 C.F.R. § 2.714(a)(1). The proposed contentions must also meet the NRC's standards for the admission of contentions. 10 C.F.R. § 2.714(b)(2).

As a general matter, as discussed below, the State has failed to show good cause for its late filing. The Board has ruled in this proceeding that, where a petitioner fails to show good cause for its untimely submission of a contention, it must make a compelling showing on the other four criteria of 10 C.F.R. § 2.714(a).[18] In the present instance, as with Proposed Utah QQ

[17] Order (Schedule for Responsive Pleadings) (August 30, 2001)

[18] Private Fuel Storage, L.L.C., (Independent Spent Fuel Storage Installation), LBP-98-7, 47 NRC 142, 208 (1998).

and the First Request, the State has not made a compelling showing on the other four factors to compensate for its lateness.

In those instances in which the State can claim good cause, the State's Second Request does not meet the NRC's standards for the admission of contentions set forth in 10 C.F.R. § 2.714(b)(2). The State bases it Second Request in part on a PFS document that it included as Exhibit 3 to its Second Request. The Commission has held that in such circumstances "boards must do more than uncritically accept a party's mere assertion that a particular document supplies the basis for its contention, without even reviewing the document itself to determine if it in fact says what the party claims it says and if it appears to support a litigable contention."[19] Moreover, licensing boards are not limited to the document itself in determining if the assertion by the party is accurate.[20] For example, such a determination "may even include consideration of the fact that the underpinnings of the document on which a contention is based have been subsequently repudiated by the document's own source."[21] As discussed below, the cited document does not stand for what the State asserts and the State's interpretation is clearly erroneous, as shown by related documents submitted herewith to provide a correct interpretation of the document. As those documents show, the Request fails to raise an admissible contention.

IV. APPLICATION OF LEGAL STANDARDS TO CLAIMS RAISED IN THE STATE'S SECOND REQUEST TO MODIFY UTAH QQ SHOWS THAT THE MODIFICATIONS ARE NOT ADMISSIBLE

A. UNTIMELINESS, REPETITIVENESS, AND LACK OF BASES FOR THE PROPOSED MODIFICATIONS TO QQ

In its Second Request, the State challenges the stability calculation for the storage pads (Cask Storage Pad Stability Calc. Rev. 9), the stability calculation for the CTB (CTB Stability

---

19 Vermont Yankee Nuclear Power Corporation (Vermont Yankee Nuclear Power Station), ALAB-919, 30 NRC 29, 48 (1989); vacated in part on other grounds and remanded, CLI-90-4, 31 NRC 333 (1990).

20 Id.; see also Yankee Atomic Electric Company (Yankee Nuclear Power Station), LBP-96-2, 43 NRC 61, 90 (1996) ("A document put forth by an intervenor as the basis for a contention is subject to scrutiny both for what it does and does not show").

21 Vermont Yankee, ALAB-919, 30 NRC at 48-49 (citations omitted).

Calc. Rev. 6) and the Holtec analysis of cask stability under hypothetical sliding conditions. These challenges concern (1) the adequacy of the soil cement testing program as it relates to the stability calculations (Items 1-2 in Exhibit 1 hereto); (2) the validity of the inertial forces calculated in Cask Storage Pad Stability Calc. Rev. 9 (Item 3 in Exhibit 1); (3) the appropriateness of the hypothetical sliding analysis presented in Cask Storage Pad Stability Calc. Rev. 9 (Items 4-8 in Exhibit 1); (4) the alleged failure of Cask Storage Pad Stability Calc. Rev. 9 to address the potential for pad-to-pad interactions (Item 9 in Exhibit 1); (5) the alleged failure of CTB Stability Calc. Rev. 6 to consider actual behavior of soil cement under tensile stresses, separation caused by vibration and impact of settlement on the integrity of the soil cement around the CTB (Item 10 in Exhibit 1); and (6) alleged inadequacies in Holtec's evaluation of cask stability under postulated sliding of the pads (Items 11-12 in Exhibit 1). None set forth an admissible contention.

1. The State's Claims Regarding PFS's Soil-Cement Testing Program Are Untimely, Repetitive, Speculative, and Based on Misconstruing Applicable Quality Assurance Requirements and PFS Documents

The State's request to modify the bases of Proposed QQ to contest PFS's soil-cement testing program is repetitive, untimely, speculative and, to the extent new, is based on a misunderstanding of Quality Assurance ("QA") requirements and PFS project records. The State essentially raises two issues regarding the soil cement: (1) that PFS has not shown that the soil cement will have the characteristics specified in PFS's revised calculations (Item 1 in Exhibit 1), and (2) that the soil-cement testing program is not being conducted under a proper quality assurance program (Item 2 in Exhibit 1).

With regard to the first issue, the State asserts that "PFS's analyses . . . still rely upon assumed values . . ." and are "incomplete without the soil test data results incorporated into stability calculations." Second Request at 2-3. In the same vein, the State claims that absent cyclic triaxial or cyclic direct shear tests, it cannot be shown that soil cement will perform properly. Id. at 3-4. These allegations are both repetitious and untimely; they were raised in Proposed Contention Utah QQ[22] and are untimely for the same reasons as when they were first filed.[23] Soil cement has been included in the design of the PFSF since December 1999 and any concerns that the State had with respect to the intrinsic characteristics, properties, and capabilities of soil cement should have been raised at that time. Thus, the State's latest attempt to revise the bases of Proposed Utah QQ is clearly untimely.

Moreover, the State's claim that PFS has not shown that the soil cement will have the properties specified in the calculations is speculative. The State has not contested that the soil cement, if placed in accordance with the design specifications, will have the properties set forth in the seismic calculations. It merely claims that PFS has not demonstrated that the soil cement meets the design specifications. As set forth in PFS's Response to Proposed Utah QQ at 18-19, this sort of hypothetical allegation does not constitute a defined disagreement with PFS that would give rise to a litigable issue of fact. Further, the State continues to ignore that PFS has committed in the SAR (at 2.6-113 & 114) to develop a soil-cement mix having the specified strength and to confirm its properties through testing conducted in accordance with industry standards. Trudeau 2nd Dec. ¶ 8. In order to operate the facility, PFS will have to meet its commitments made in the SAR, including the performance requirements for the soil cement. For that purpose, PFS is undertaking a testing program. As set forth in PFS's Response to Proposed Utah QQ at 19, to argue that PFS will not abide by its licensing commitments is not admissible.

The second attack on PFS's soil-cement testing program is based on the State's assertion that "the contractor chosen by PFS to conduct the soil-cement testing program <u>does not appear</u> to be qualified to conduct the work within the scope set forth by PFS . . ." in that "PFS was unaware of any of the recommended bidders' qualifications" as it awarded the soils testing contract. Second Request at 3 (citing Exhibit 3 to the Second Request) (emphasis added). On the basis of its interpretation of that document, the State alleges that the PFS testing program is not

---

[22] See, e.g., Proposed Utah QQ at 11-14.

[23] See PFS Response to Proposed Utah QQ at 6-7, 12-13; id. Exh. A, Items 15-28.

being conducted in compliance with applicable QA requirements, as specified in the Engineering Services Scope of Work ("ESSOW") for the testing program, and argues that:

> any reliance by PFS on its soil testing program will be unsupportable unless and until PFS validates and verifies the quality assurance program under which the testing is being performed.

Id. (emphasis added).

As the case law discussed above makes clear, however, a document must be scrutinized both for what it does and does not establish. Exhibit 3 clearly does not establish that the soil testing program is not being conducted pursuant to an NRC - approved QA program as asserted by the State. Indeed, this is clearly an erroneous interpretation of the document and an irresponsible allegation, as shown by other project documents provided to the State simultaneously with Exhibit 3 as part of discovery. See Trudeau 2nd Dec ¶¶ 12-15 and Exh. 5 thereto.[24]

First, contractors and subcontractors are not required to have their own independent quality assurance programs, as suggested by the State, but may work under the approved quality assurance programs of their customers.[25] The ESSOW for the testing program, which the State itself references, specifically provides in this respect that the soil cement testing contractor shall either have in effect its own approved QA program or, alternatively, "shall conform to the Engi-

[24] State's Exhibit 3 and other additional discovery documents were made available to the State (and other parties) at PFS's document repository at Parson Behle's offices in Salt Lake City on July 6, 2001. Letter from P. Gaukler (Counsel for PFS) to D. Chancellor (Counsel for State), dated July 6, 2001. The State requested copies of these documents and they were copied and sent to the State on July 16, 2001. Letter from M. Dabel (Legal Assistant for PFS) to D. Chancellor (Counsel for State), dated July 19, 2001. Among the documents provided to the State at that time were the State's Exhibit 3 as well as the Engineering Services Scope of Work for the testing program (described in Trudeau's 2nd Dec. ¶¶ 12, 15) and the January 11, 2001 letter to the PFS Board Chairman announcing the selection of the contractor for the testing program (described in Trudeau's 2nd Dec. ¶ 13 and attached as Exh. 5 thereto), both discussed further in the text below. (The copy of the January 11, 2001 letter submitted as Exh. 5 is a redacted copy which excludes proprietary and confidential information not relevant to the matter at issue here.)

[25] See NUREG-1567, *Standard Review Plan for Spent Fuel Dry Storage Facilities* (Final Report) (March 2000), Section 12.4.4.2., titled "Contractor QA Programs" which provides:

> The applicant must have responsibilities assigned and, prior to implementation, instructions and procedures issued for requiring, to the extent necessary, that contractors or subcontractors adhere to a QA program consistent with the applicable provisions of 10 CFR 72, Part G (10 CFR 72.148; NQA-1/Part I/Sec II 4S-1 Par 2.3; NUREG-0800).

(Emphasis added.)

neers' Quality Assurance Program," i.e., Stone & Webster's QA program.[26] Further, in selecting the contractor to perform the laboratory testing work, Stone & Webster made the decision that the contractor would in fact perform its work under the Stone & Webster NRC-approved QA program. Trudeau 2nd Dec. ¶¶ 13-14. This fact is reflected in Exhibit 5 to Trudeau's 2nd Declaration, which again was provided to the State in discovery (note 24, supra). As the State is aware, Stone & Webster is well qualified to perform nuclear QA work,[27] and the State has asserted no basis here on which to challenge the adequacy of the Stone & Webster QA program.

Thus, contrary to the State's erroneous interpretation of its Exhibit 3, the soil-cement testing is being conducted under an NRC approved quality assurance program. The State's misreading of a single document does not provide a basis for an admissible contention and the proposed modification to Proposed Utah QQ must be rejected.

2. Inertial Forces Acting on Storage Cask Pads

The second area contested by the State is PFS's calculation of inertial forces of the storage pads and the underlying soil cement acting on the native soil. The State challenges PFS's assumption in Cask Storage Pad Stability Calc. Rev. 9 that the storage pads and the cement-treated soil will act as a rigid system. The State readily acknowledges that this is not a new allegation but a long-standing dispute; indeed, both PFS's assumption and the State's challenge to it substantially predate Proposed Utah QQ.[28] Therefore, the State's claims should be rejected as repetitive, as well as for the reasons set forth previously.[29]

---

[26] Trudeau 2nd Dec. ¶ 12. Section 4.0 of the ESSOW provides that:

> 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.

Id. (emphasis added).

[27] See, e.g., SAR at 11.1-4 ("The QA program of the A/E, Stone and Webster Engineering Corporation (SWEC), (Reference 2), has been approved by the NRC as meeting the requirements of 10 CFR 50 Appendix B.")

[28] Second Request at 4-5 and note 4; see also, PFS Response to Proposed Utah QQ, Exh. A, Items 3, 12, 18.

[29] PFS Response to Proposed Utah QQ, at 5, 8-9; id. at Exh. A, Items 3, 12, 18.

3. The State's Assertions Regarding PFS's Application of the Newmark Sliding Block Analysis Are Not Admissible Contentions

The State next attacks PFS's "hypothetical sliding case where resistance to sliding is based on frictional resistance along the base of the pads and the cement-treated soil," which it finds flawed because: (i) the calculation does not meet NRC guidance for safety factors against sliding, (ii) the calculation assumes that the pads will remain rigid, (iii) the analysis does not take into account the possibility of unsymmetrical sliding, (iv) the Newmark charts may not be appropriate to calculate the sliding that may occur due to a design basis ground motion at the site. Second Request at 6-7. These claims are immaterial, untimely and incorrect.

The State's claims are immaterial because the sliding analysis challenged by the State is not part of the PFS design basis. The Newmark sliding analysis was conducted in response to a request for information by the NRC Staff, that a purely hypothetical scenario be evaluated of the potential for sliding of the pads assuming, contrary to fact, that the resistance to sliding is comprised only of frictional resistance along the base of the pads and soil cement plus passive resistance acting on the end of the pad and soil-cement block, ignoring the cohesive portion of the strength of the clayey soils at this interface.[30] The hypothetical nature of this analysis is clearly stated on page 36 of Cask Storage Pad Stability Calc. Rev. 9.[31] As such, it was a simplified calculation not intended to be part of the design basis of the facility, but rather to demonstrate the conservatism of the design.

Thus, the State's claim that PFS has failed to meet the regulatory guidance provided by NUREG-75/087 by postulating a design that does not meet the suggested factor of safety against sliding is both incorrect and immaterial.[32] The facility design meets or exceeds the recom-

---

[30] Trudeau 2nd Dec. ¶ 7.

[31] The relevant portions of the calculation are attached as Exhibit 1 to the Trudeau 2nd Declaration.

[32] It should also be noted that the technical provisions provided by the NRC in NUREG-75/087 (now NUREG-0800), which is the Standard Review Plan for nuclear power plants, do not have binding regulatory force and are only intended as guidance. See, e.g., NUREG-0800 at 1 ("Standard review plans are not substitutes for regulatory guides or the NRC's regulations, and compliance with them is not required.")

mended factor of safety.[33] For the same reason, the State's related claims concerning the application of the Newmark methodology to the hypothetical sliding analysis are also immaterial.

Second, the State's claims regarding the correct application of the Newmark methodology to the PFSF site are untimely. The Newmark methodology has been part of the pad stability calculation since Revision 4 (pp. 14A to 14F), dated September 3, 1999. See Trudeau 2nd Dec. ¶ 9 and Exh. 3. All that has changed in the latest version of the calculation is the addition of a hypothetical case using different, obviously conservative values of the strength available to resist sliding along the base of the sliding block. Id. ¶ 7.

Third, Holtec prepared its own independent sliding displacement analysis of the storage pads in which it did not use the Newmark analysis, but used the actual time histories for the design earthquake for the PFSF site. Id. ¶ 10 and Exh. 4 thereto. The State has not challenged the Holtec calculation in these areas.

Thus, not only are the State's attacks on the hypothetical Newmark Block Sliding Analysis contained in Cask Storage Pad Stability Calc. Rev. 9 immaterial, but they are also untimely and ignore that PFS has provided a more rigorous, site-specific sliding analysis independent of the simplified analysis performed per the Newmark methodology.

4. The State's Claims Concerning Failure to Account for Pad-to-Pad Interactions Are Late and Unsupported

The State asserts that Cask Storage Pad Stability Calc. Rev. 9 is also incorrect because it fails to recognize the potential for pad-to-pad interaction. Second Request at 8. However, the State acknowledges that it raised this identical issue in Proposed Utah QQ with respect to an earlier version of the calculation. Id. at 10. Therefore, no modification of Proposed Contention QQ is warranted to incorporate this claim which should be rejected as repetitious, and for the reasons set forth in PFS's Response to Proposed Utah Contention QQ at 14-16.

[33] See page 23 of Cask Storage Pad Stability Calc. Rev. 9, where the minimum factor of safety is clearly stated in bolded font and underlined as "1.27 (=Min)." Trudeau 2nd Dec., Exh. 1. This is also emphasized in the first paragraph of the "Conclusions" Section on page 99 entitled "Sliding Stability of the Cask Storage Pads." Id.

5. The State's Claim Regarding the Seismic Stability Analysis of the CTB Is Unjustifiably Late and Repetitive

The State tries once again to claim that PFS cannot credit any beneficial effects from the soil cement used as part of the CTB design because PFS does not consider the behavior "of the soil cement under tensile stresses; separation caused by vibration of the building; and the impact of settlement." Second Request at 8. As the State admits, it has already raised this concern in Proposed Utah QQ. Id. The claim is therefore neither new nor timely. The State has known about the calculation methodology, which did not incorporate tensile stresses, settlement or vibration of the building, since at least November 2000 when State counsel raised these issues during the deposition of PFS witness Paul Trudeau.[34] This alleged basis is not only not new, it is even more inexcusably late than when the State tried to raise it in Proposed Contention QQ.

6. Holtec's Sliding Analysis

Lastly, the State challenges Holtec's evaluation of the impact of hypothetical sliding of the pad as part of its cask stability analysis assuming a loss of cohesion at the bottom of the pad. As with the Newmark sliding analysis discussed above, this hypothetical case is not part of the PFS design basis, but was done to determine whether the hypothetical sliding of the pad would have a detrimental or beneficial impact on the casks' response to a design basis earthquake. See Trudeau 2nd Dec. ¶ 10 and Exh. 4 thereto. Based on physical principles, Holtec concluded that sliding of the pad relative to the soil would serve to decrease the energy imparted to the casks and therefore decrease the motion of the casks relative to the pad. Id. Holtec then did a confirmatory analysis which confirmed its qualitative conclusion. Id. As with the Newmark sliding analysis, the State's challenge to Holtec's evaluation of this hypothetical, beyond-design-basis set of conditions does not give rise to an admissible contention because it is immaterial.

Further, the State admits that it is seeking to raise the same alleged deficiencies with Holtec's analysis that it has repeatedly sought to raise in the past. See Second Request at 9, ref-

[34] See State's First Request at 6; Bartlett Declaration (June 19, 2001) ¶ 10 and Exhibit A (referring to transcript of deposition of Paul Trudeau, November 15, 2000 at 147-48).

erencing Ostadan's May 16, 2001 Declaration ¶ 11(a)-(f). These assertions are repetitious, as well as untimely, and indeed were previously rejected by the Board three years ago when raised in connection with Utah EE. See PFS Response to Proposed Utah QQ at 12-13, 19-20. The Board should reject these claims once again.

The State also claims that Holtec inappropriately failed to consider the effect of soil cement around the pad in its analysis of the impact of hypothetical sliding of the pad on cask response. However, this new allegation, while taking issue with this particular aspect of the calculation, leaves unchallenged Holtec's principal conclusion that sliding of the pad will decrease the energy input to the cask and, therefore, decrease the cask's potentially adverse response. Thus, this aspect of the State's claim is immaterial for this reason as well.

B. THE STATE HAS MADE NO COMPELLING SHOWING ON THE OTHER LATE FILING FACTORS

The State argues that the requested modification is timely, "because it is being filed in twenty-four days from receipt of the two revised seismic stability analyses calculations for the pads and CTB and within about two week [sic] of receipt of the revised Holtec calculation" and "the revised calculations raise additional safety concerns that were not evident in the revisions upon which Utah QQ is based." This argument is invalid. With only two exceptions, the issues that the State seeks to raise were either raised in Proposed Utah QQ, or could have been raised then or much earlier. The two new issues raised by the State, as discussed above, are clearly erroneous or immaterial. Thus, the State lacks good cause for late-filing this request to modify the bases of Proposed Utah QQ.

Lacking good cause for a delay in filing a late-filed contention, the State "must make a compelling showing on the other four factors" in 10 C.F.R. § 2.714(a)(1). LBP-98-7, 47 NRC at 208. The State seeks to broaden both the scope of Proposed Utah QQ and the scope of the proceeding with these issues. Thus, factor five weighs against the admission of the contention, which raises a range of issues relating to soil cement, Applicant's quality assurance programs and other seismic design issues that are not currently being litigated in this proceeding.

The admission of the modification would do little to develop a sound record, the third factor. The modifications to Proposed Utah QQ are in most cases yet another rehash of the issues that the State is seeking to raise in Proposed Utah QQ. Thus, litigation of the modifications to Proposed Utah QQ would be repetitive and do nothing to develop a sound record on which a licensing decision can be made at this time. The two issues that are not a rehash do nothing to develop a sound record because they are clearly erroneous or immaterial.

With respect to factor two, a number of concerns raised by the State relate to potential issues that may (or may not) become problems after the licensing of PFS.[35] For such issues, there is an adequate means of protecting the State's interest via a 10 C.F.R. §2.206 petition for Staff action against PFS. Even if there are not other means to protect the State's interest on this issue, and even if the State's position is not represented by another party (factor four), these factors carry less weight than the others. Thus, a balancing of the four remaining factors also militates against the modification of Proposed Utah QQ. The State has clearly failed to make the compelling showing required to overcome its lack of good cause for its late filing.

V. CONCLUSION

For the foregoing reasons, PFS submits that the State's Second Request to modify Proposed Contention Utah QQ fails to raise a litigable contention and should be denied.

Respectfully submitted,

Paul Gaukler

Jay E. Silberg
Ernest L. Blake, Jr.
Paul A. Gaukler
Matias F. Travieso-Diaz
SHAW PITTMAN LLP
2300 N Street, N.W.
Washington, DC 20037
Counsel for Private Fuel Storage L.L.C.

Dated: September 7, 2001

[35] For example, the State contends that the soil cement *may* not have the characteristics assumed in the design calculations. Such a concern may not materialize, if at all, until the PFSF is licensed.

EXHIBIT 1

EXHIBIT 1

Prior History of Claims Raised in State's Second Request to Modify the Bases of Proposed Contention Utah QQ

Item	Issue	Description of State's Claim	Filings in Which Claim was Previously Raised by State	Relevant PFS Documents
	A. PFS's Soil Cement-Testing Program			
1.	A.1. PFS's Soil Cement-Testing Program, including cyclic triaxial or cyclic direct shear testing	Calculation G(B)-04, Rev. 9 relies on assumed values for the soil-cement, and PFS does not intend to perform confirmatory cyclic triaxial or cyclic direct shear testing.	Proposed Contention Utah QQ at 6-9, 11-14.	PFS Letter, Parkyn to NRC, LA Amendment 8, dated December 16, 1999, introduced the use of soil cement in the storage pad emplacement area to enhance pad stability under seismic conditions. SAR page 2.6-84 (Rev. 8) stated: "The required engineering characteristics of the soil cement can be easily engineered during detailed design to meet the necessary strength requirements." See also SAR page 2.6-26 (Rev. 8). SAR page 2.6-91 (Rev. 8) stated: "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."
2.	A. 2. Quality Assurance Program	PFS's soil-cement testing program is not being conducted under an NRC approved QA program.	Newly Raised	PFS's Engineering Services Scope of Work for Laboratory Testing of Soil-Cement Mixes (ESSOW No. 05996.02-G010, Rev. 0), dated January 31, 2001, provides QA requirements for laboratory testing vendor; January 11, 2001 Letter from J. Cooper (Stone & Webster) to J. Parkyn (PFS) states that the work by vendor is "QA Category I and the vendor will perform all work per the [Stone & Webster] QA program."
3.	***B. Inertial Forces and the Rigidity of the Cask-Pad System***	Calculation G(B)-04, Rev. 9 incorrectly assumes that the soil cement-pad system will act as a rigid body.	Proposed Contention Utah QQ at 6, 9-11. The State had earlier raised this issue in Proposed Contention Utah EE at 7-9.	The storage cask pad stability analysis utilizing soil cement was specifically referenced in LA Amendment 13, in June 2000. SAR Section 2.6.1.12.1, Rev. 13, which references Calculation No. 05996.02- G(B)-04, Rev. 6, in discussions of storage pad stability analyses. This calculation was submitted to the NRC by PFS letter from J. Donnell dated June 19, 2000, "Submittal of Commitment Resolution Letter #34 Information".

	C. Newmark Block Sliding Analysis			
4.	C.1. Hypothetical Case Uses Factor of Safety Less Than 1.1	Calculation G(B)-04, Rev. 9 evaluates and accepts the results of an analysis in which the factor of safety against sliding is less than 1.1, which does not comply with NUREG-75/087.	Newly Raised	Cask Storage Pad Stability Calculation No. 05996.02-G(B)-4, Rev. 4, dated September 3, 1999, and subsequent revisions to the calculation, evaluate a hypothetical case in which the safety factor against sliding is less than 1.
5.	C.2. Pad Rigidity	In Calculation G(B)-04, Rev. 9, the Newmark Block Sliding Analysis assumes that the pads will behave in a rigid manner and uses peak vertical ground acceleration in calculating the maximum resistance coefficient.	Proposed Contention Utah QQ at 6, 10. The State has raised this issue in Proposed Contention Utah EE at 8-9.	Cask Storage Pad Stability Calculation No. 05996.02-G(B)-4, Rev. 4, dated September 3, 1999 incorporates a Newmark Block Sliding Analysis that is methodologically identical to the one in the current revision (Rev. 9).
6.	C.3. Unsymmetrical Sliding	In Calculation G(B)-04, Rev. 9, the Newmark Block Sliding Analysis does not consider unsymmetrical sliding.	Newly Raised	Same as Item 5 above.
7.	C.4. Newmark Chart – Standardized to .5g Ground Motion	In Calculation G(B)-04, Rev. 9, the Newmark Block Sliding Analysis is not based on the design basis ground motion for the site.	Newly Raised	Same as Item 5 above. Also, letter from Holtec, dated August 6, 2001 (submitted by PFS to the NRC under cover letter dated August 7, 2001), uses PFS site-specific data in Holtec parallel sliding analysis.
8.	C.5. Newmark Chart – Based on Four Western Earthquakes	In Calculation G(B)-04, Rev. 9, the Newmark Block Sliding Analysis uses data from earthquakes that may not be similar to the design basis earthquake.	Newly Raised	Same as Item 5 above. Also, letter from Holtec, dated August 6, 2001 (submitted by PFS to the NRC under cover letter dated August 7, 2001), uses PFS site-specific data in Holtec parallel sliding analysis.
9.	***D. Other Incorrect Calculations in the Pad Stability Analyses (Pad-to-Pad Interactions)***	Calculation G(B)-04, Rev. 9 fails to recognize the potential for pad-to-pad interaction.	Proposed Utah Contention QQ at 10.	The storage pad stability analysis was specifically referenced in LA Amendment 13, in June 2000. SAR Section 2.6.1.12.1, Rev. 13, references Calculation No. 05996.02- G(B)-04, Rev. 6, in discussions of storage pad stability analyses. This calculation was submitted to the NRC by PFS letter from J. Donnell dated June 19, 2000, "Submittal of Commitment Resolution Letter #34 Information".
10.	***E. Canister Transfer Building Sliding Stability Analyses***	Calculation G(B)-13, Rev. 6, incorrectly assumes that the soil cement will provide additional resistance without taking into account factors that may cause the soil cement to not provide the assumed additional resistance.	Proposed Utah QQ at 8-9.	Stone & Webster Calculation No. 05996.02- G(B)-04, Rev. 6, Stability Analyses of Storage Pad, was referenced in SAR Section 2.6.1.12.1, Stability and Settlement Analyses-Cask Storage Pads, Revision 13 (Submitted to the NRC by PFS Letter, Parkyn to NRC, LA Amendment 13, dated June 23, 2000). This calculation was submitted to the NRC by PFS letter from J. Donnell dated June 19, 2000, "Submittal of

				Commitment Resolution Letter #34 Information". CTB Stability Calculation No. 05996.02-G(B)-13, Rev. 4 was submitted to the NRC as part of LA Amendment dated March 30, 2001.
	F. Holtec Calculation			
11.	F.1. Simplified Assumptions	Holtec Report HI-2012653, Rev. 1 incorrectly models cask response.	Proposed Utah EE at 6-11. Proposed Utah Contention QQ at 8-11.	Holtec's cask stability analysis was included in original PFSF License Application submittal in June 1997. SAR Section 8.2.1.2, Rev. 0, referred to Holtec Report No. HI-971631, Multi-Cask Seismic Response at the PSF ISFSI, Rev. 0, dated May 19, 1997. The same methodology was used in subsequent cask stability analyses.
12.	F.2. Unsymmetrical Sliding	Holtec Report HI-2012653, Rev. 1 fails to model the unsymmetric loading that soil-cement will impart once sliding occurs.	Newly Raised.	Letter from Holtec, dated August 6, 2001 (Submitted by PFS to the NRC under cover letter dated August 7, 2001).

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
	)	
(Private Fuel Storage Facility)	)	ASLBP No. 97-732-02-ISFSI

CERTIFICATE OF SERVICE

I hereby certify that copies of the Applicant's Response to State of Utah's Second Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to More Revised Calculations From the Applicant, Exhibit 1 thereto, and the Declaration of Paul J. Trudeau were served on the persons listed below (unless otherwise noted) by e-mail with conforming copies by U.S. mail, first class, postage prepaid, this 7th day of September, 2001.

G. Paul Bollwerk III, Esq., Chairman Administrative Judge
Atomic Safety and Licensing Board Panel
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001
e-mail: GPB@nrc.gov

Dr. Jerry R. Kline
Administrative Judge
Atomic Safety and Licensing Board Panel
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001
e-mail: JRK2@nrc.gov; kjerry@erols.com

Dr. Peter S. Lam
Administrative Judge
Atomic Safety and Licensing Board Panel
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001
e-mail: PSL@nrc.gov

*Office of Commission Appellate Adjudication
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001

Office of the Secretary
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001
Attention: Rulemakings and Adjudications
Staff
e-mail: hearingdocket@nrc.gov
(Original and two copies)

* Adjudicatory File
Atomic Safety and Licensing Board Panel
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555-0001

Catherine L. Marco, Esq.
Sherwin E. Turk, Esq.
Office of the General Counsel
Mail Stop O-15 B18
U.S. Nuclear Regulatory Commission
Washington, D.C. 20555
e-mail: pfscase@nrc.gov

Denise Chancellor, Esq.
Assistant Attorney General
Utah Attorney General's Office
160 East 300 South, 5th Floor
P.O. Box 140873
Salt Lake City, Utah 84114-0873
e-mail: dchancel@state.UT.US

John Paul Kennedy, Sr., Esq.
David W. Tufts, Esq.
Confederated Tribes of the Goshute
Reservation and David Pete
Durham Jones & Pinegar
111 East Broadway, Suite 900
Salt Lake City, Utah 84105
e-mail: dtufts@djplaw.com

Joro Walker, Esq.
Land and Water Fund of the Rockies
1473 South 1100 East
Suite F
Salt Lake City, UT 84105
e-mail: lawfund@inconnect.com

Diane Curran, Esq.
Harmon, Curran, Spielberg &
Eisenberg, L.L.P.
1726 M Street, N.W., Suite 600
Washington, D.C. 20036
e-mail: dcurran@harmoncurran.com

Tim Vollmann, Esq.
Skull Valley Band of Goshute Indians
3301-R Coors Road, N.W.
Suite 302
Albuquerque, NM 87120
e-mail: tvollmann@hotmail.com

*Richard E. Condit, Esq.
Land and Water Fund of the Rockies
2260 Baseline Road, Suite 200
Boulder, CO 80302

Paul EchoHawk, Esq.
Larry EchoHawk, Esq.
Mark EchoHawk, Esq.
EchoHawk PLLC
P.O. Box 6119
Pocatello, ID 83205-6119
e-mail: pechohawk@hollandhart.com

* By U.S. mail only

Paul Gaukler

Paul A. Gaukler

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
	)	
(Private Fuel Storage Facility)	)	

DECLARATION OF PAUL J. TRUDEAU

Paul J. Trudeau states as follows under penalty of perjury:

1. I am a Senior Lead Geotechnical Engineer at Stone & Webster, Inc. ("S&W") in Stoughton, Massachusetts. I provide this declaration in support of Applicant's Response to "State of Utah's Second Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to More Revised Calculations From the Applicant" ("Second Request"). I have reviewed proposed Contention Utah QQ ("Proposed Utah QQ") as submitted by the State of Utah ("State") in this proceeding, the State's "Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to Further Revised Calculations from the Applicant" ("First Request"), and the Second Request, as well as the supporting Declarations of Dr. Steven F. Bartlett (dated August 23, 2001) ("Bartlett Declaration") and Dr. Farhang Ostadan (dated August 22, 2001 ("Ostadan Declaration"). I will address those documents in this Declaration.

2. My professional and educational experience is summarized in the curriculum vitae attached as Exhibit 1 to my July 3, 2001 Declaration in support of Applicant's response to the First Request. As indicated there, I have twenty-eight years of

experience in geotechnical engineering, including the performance of subsurface soil investigations and the analysis of foundations in support of the design of structures.

3. S&W is the Architect/Engineer for the Private Fuel Storage Facility ("PFSF") under contract with Private Fuel Storage, L.L.C. ("PFS" or "Applicant"). As such, it coordinates the facility design activities, including the studies needed to characterize the PFSF site and establish its suitability. My particular areas of concentration on the PFSF project are the analysis of soils – settlement, bearing capacity, and stability of foundations – as well as the conduct of soils investigations, laboratory testing of soils to measure static and dynamic properties, and the performance of computer-aided analyses of the behavior of soils and structures under static and dynamic loading conditions.

4. Part of my duties as lead geotechnical engineer is to perform, or direct the performance of, analyses of the response of the PFSF structures to the forces imparted by postulated seismic events. In particular, I was responsible for the preparation of Stone & Webster Calculation Nos. 05996.02-G(B)-04, Rev. 9, *Stability Analyses of Cask Storage Pads* (July 26, 2001) ("Cask Storage Pad Stability Calc. Rev. 9"), and 05996.02-G(B)-13, Rev. 6, *Stability Analyses of Canister Transfer Building* (July 26, 2001) ("Canister Transfer Building Stability Calc. Rev. 6"). Copies of relevant excerpts from these two calculations are included as Exhibits 1 and 2 hereto.

5. These calculations are updated versions of calculations of the same respective titles issued on May 31, 2001 ("Cask Storage Pad Stability Calc. Rev. 8" and "Canister Transfer Building Stability Calc. Rev. 5"), copies of which were included as exhibits

to my July 3, 2001 Declaration. I was responsible for the preparation of these earlier calculations and their predecessors.

6. Cask Storage Pad Stability Calc. Rev. 9 was prepared in July 2001 at the request of the Staff of the U.S. Nuclear Regulatory Commission ("Staff"). The primary reason for revising this calculation was to add a hypothetical case to the sliding stability analyses that uses an "obviously conservative value" (as requested by the NRC Staff and described at page 36 of the calculation) of the strength available to resist sliding of the pad and cement-treated soil on top of the underlying clayey soils. If the calculated factor of safety was less than 1.0, we would then, in accordance with commitments in the PFSF Safety Analysis Report ("SAR") at p. 2.6-45 and the project's Geotechnical Design Criteria (Calc. 05996.02-G(B)-5-1), estimate the amount of horizontal displacement that might occur due to the design earthquake. Thus, an analysis was added in Cask Storage Pad Stability Calc. Rev. 9 of a hypothetical set of conditions in which the resistance to sliding of the pad was assumed to be provided only by the frictional resistance along the base of the pads and underlying cement-treated soil and the passive resistance of the soil acting on the end of the sliding block, and all cohesion available along the base of the pad and underlying cement-treated soil was to be ignored. The hypothetical nature of this analysis is indicated on page 36 of Cask Storage Pad Stability Calc. Rev. 9.

7. As expected, this hypothetical analysis using "obviously conservative" shear resistance values yielded a factor of safety against sliding of less than 1.0. Therefore, a follow-up analysis was performed to estimate the maximum pad displacement that might occur for this case. This analysis showed that the resulting maximum horizontal displacements, in the range of 2 to 6 inches (page 46 of Cask Storage Pad

Stability Calc. Rev. 9), even if they were to occur, would have no adverse safety consequences.

8. Likewise, PFS's program to develop and test an appropriate soil-cement mix design for use at the PFSF site is continuing to be implemented in accordance with PFS commitments on pages 2.6-113-114 of the SAR. In this regard, the observation in Dr. Bartlett's Declaration at para. 5 that the calculations for the stability analysis of the cask storage pads and the Canister Transfer Building ("CTB") "are incomplete without [the soil cement] test results" is inaccurate. The calculations assume, as committed to in the SAR, that a soil-cement mixture having the specified strength will be developed and its properties confirmed through testing conducted in accordance with industry standards. Given these commitments, the calculations do not need to present the results of the test program, which is at this point ongoing.

9. The sliding displacement calculation uses a methodology known as the "Newmark sliding block analysis." This same methodology was utilized in the previous PFS stability analyses for the pads, dating all the way back to Revision 4 (issued September 3, 1999) of the cask storage pad stability calculation, to analyze a hypothetical situation that addresses the potential existence of cohesionless soils at some depth beneath the pads. To simplify this analysis, it was conservatively assumed that the pads were founded directly on cohesionless soil. In this case, the factor of safety against sliding was less than 1.0, meaning that the pads were subject to sliding. The analyses based on Newmark's method are presented on pages 14, 14A to 14F of Calculation 05996.02-G(B)-4, Rev. 4 (September 3, 1999) (see Exhibit 3 hereto). The Newmark analyses can also be found on pages 34 to 39 of Revision 8 and pages 46 to 51 of Revision 9 of Calculation 05996.02-G(B)-4. Although the

application of the methodology in these calculations used different values for the strength of the cohesionless soil assumed to exist at the base of the foundation, the remaining assumptions and modeling techniques used then are the same as are used now in the current version of the calculation that is challenged by the State.

10. In para. 10 of his declaration, Dr. Bartlett criticizes the use of the Newmark analysis, claiming that the charts presented by Newmark are based on earthquake records that were normalized to a ground acceleration of 0.5g instead of the peak ground acceleration of ~0.7g applicable for the design basis earthquake for the PFSF. Dr. Bartlett also notes that the charts in the Newmark paper are based on data from only four western earthquakes. I would note, first, that these features of the Newmark analysis have been present in all of the pad sliding stability calculations performed by PFS since September 3, 1999. Second, Holtec prepared its own independent sliding displacement analysis of the storage pads in which it used the actual time histories for the design earthquake for the PFSF site for this hypothetical case. See Exhibit 4 hereto, which is an August 6, 2001 letter from Holtec (submitted by PFS to the NRC under cover letter dated August 7, 2001) summarizing the results of the Holtec calculation. Thus, the criticisms raised by Dr. Bartlett are inapplicable to Holtec's calculation. Third, the Newmark sliding block analysis included in Cask Storage Pad Stability Calc. Rev. 9 yields approximately the same results – i.e., pad displacements on the order of a few inches – as does the more elaborate time-history analysis conducted by Holtec. Compare the table on page 4 of Exhibit 4 with the table on page 45 of Cask Storage Pad Stability Calc. Rev. 9.

11. The State, supported by Dr. Bartlett's Declaration at para. 6, disputes that the soil-cement laboratory testing program performed by PFS is being conducted in full

compliance with applicable Quality Assurance ("QA") Category I requirements. The State and Dr. Bartlett interpret certain statements in the recommended bidders list (Exhibit 3 to the Second Request) for the ESSOW for laboratory testing of soil-cement mixes as meaning that none of the bidders were qualified to perform the soil-cement testing in accordance with the QA Category I requirements applicable to the scope of work. However, the State and Dr. Bartlett misunderstand the QA requirements for contractors performing such work.

12. The laboratory testing program is being conducted in accordance with the requirements of the S&W Engineering Services Scope of Work (ESSOW No. 05996.02-G010), whose preparation I oversaw. Section 4.0 of the ESSOW addresses Quality Assurance requirements and states the following:

> *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.*

(In the ESSOW, S&W is identified as the "Engineers.") Thus, under the ESSOW, it is unnecessary for the contractor performing the laboratory testing work to demonstrate that it has an acceptable QA program, since it can perform its work in accordance with S&W's QA program.

13. In selecting the contractor to perform the laboratory testing work, S&W made the decision that the vendor would perform all work under S&W's QA program. See Exhibit 5, which is a letter dated January 11, 2001 from the PFS Assistant Project Manager to the Board Chairman announcing the selection of the laboratory testing contractor and stating that the laboratory testing work "is considered QA Category I and the vendor will perform all work per the S&W QA program." In fact, the contractor's testing work is being performed under S&W's QA program.

14. S&W has a Quality Assurance program in place that has been reviewed and approved by the NRC for QA Category I work. S&W also has in effect a training and indoctrination, inspection, testing, and documentation program to ensure that the laboratory testing and equipment meet the requirements of this ESSOW and federal regulations 10CFR50, Appendix B, and 10CFR72.

15. Section 3.1 of the ESSOW addresses Laboratory Testing Services and specifies the following performance requirements:

> *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.*

Reg. Guide 1.138, "Laboratory Investigations of Soils for Engineering Analysis and Design of Nuclear Power Plants" (April 1978), describes acceptable laboratory investigations and testing practices for determining soil and rock characteristics needed for engineering analysis and design for foundations for nuclear power plants. The ESSOW also requires that all testing be performed to industry (ASTM) standards or using written procedures approved by S&W.

16. Since S&W has an NRC-approved Quality Assurance program under which the testing program is being conducted, and all tests are being performed following industry standards and in accordance with the requirements of US NRC Regulatory Guide 1.138, the PFS soil-cement laboratory testing program is in full compliance with the

Quality Assurance Category I requirements of the ESSOW and the results of the program can appropriately be used for the design and construction of the PFSF.

17. Apart from the matters discussed in previous paragraphs, in my review of the Second Request and the supporting Declarations of Drs. Bartlett and Ostadan, I found no other technical claims that have not already been made in Proposed Utah QQ or the First Request, or which could not have been raised as part of that proposed contention or even earlier in this proceeding.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on September 6, 2001.

Paul J. Trudeau

Paul J. Trudeau

TRUDEAU SECOND DECLARATION

EXHIBIT 1

Calculation No. 05996.02-G(B)-04, Rev. 9

STONE & WEBSTER, INC.

5010.64

CALCULATION SHEET

CLIENT & PROJECT	PAGE 1 OF 115
PRIVATE FUEL STORAGE, LLC – PFSF	+ 22 pp of ATTACHMENTS

CALCULATION TITLE	QA CATEGORY (✓)
STABILITY ANALYSES OF CASK STORAGE PADS	☑ I NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ (other)

CALCULATION IDENTIFICATION NUMBER				
JOB ORDER NO.	DISCIPLINE	CURRENT CALC NO	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.02	**G(B)**	**04**		

PREPARER(S)/DATE(S)	REVIEWER(S)/DATES(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES CALC NO. OR REV NO.	CONFIRMATION REQUIRED ☑ YES	NO
Original Signed By: TESponseller / 2-18-97 PJTrudeau / 2-24-97	Original Signed By: PJTrudeau / 2-24-97 TESponseller / 2-24-97	Original Signed By: NTGeorges / 2-27-97	0		✓	
Original Signed By: TESponseller / 4-30-97 PJTrudeau / 4-30-97	Original Signed By: PJTrudeau / 4-30-97 TESponseller / 4-30-97	Original Signed By: AFBrown / 5-8-97	1	0		✓
Original Signed By: PJTrudeau / 6-20-97	Original Signed By: NTGeorges / 6-20-97	Original Signed By: AFBrown / 6-20-97	2	1		✓
Original Signed By: PJTrudeau / 6-27-97	Original Signed By: LPSingh / 7-1-97	Original Signed By: LPSingh / 7-1-97	3	2		✓
Original Signed By: DLAloysius / 9-3-99 SYBoakye / 9-3-99	Original Signed By: SYBoakye / 9-3-99 DLAloysius / 9-3-99	Original Signed By: TYChang / 9-3-99	4	3	✓	
Original Signed By: PJTrudeau / 1-26-00	Original Signed By: TYC for SYBoakye 1-26-00 Lliu / 1-26-00	Original Signed By: TYChang / 1-26-00	5	4		✓
Original Signed By: PJTrudeau / 6-16-00	Original Signed By: TYChang / 6-16-00	Original Signed By: TYChang / 6-16-00	6	5		✓
Original Signed By: SYBoakye / 3-30-01	Original Signed By: TYChang / 3-30-01	Original Signed By: TYChang / 3-30-01	7	6	✓	

DISTRIBUTION

GROUP	NAME & LOCATION	COPY SENT (✓)	GROUP	NAME & LOCATION	COPY SENT (✓)
RECORDS MGT. FILES (OR FIRE FILE IF NONE)	JOB BOOK R4.2G	ORIG			
	FIRE FILE - Denver	☑			
Geotechnical	PJTrudeau – Stoughton/3	☑			

STONE & WEBSTER, INC.

5010.64

CALCULATION SHEET

CLIENT & PROJECT					PAGE 2
PRIVATE FUEL STORAGE, LLC – PFSF					
CALCULATION TITLE					QA CATEGORY (✓)
STABILITY ANALYSES OF CASK STORAGE PADS					☑ I NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ (other)
CALCULATION IDENTIFICATION NUMBER					
JOB ORDER NO.	DISCIPLINE	CURRENT CALC NO	OPTIONAL TASK CODE		OPTIONAL WORK PACKAGE NO.
05996.02	**G(B)**	**04**			

APPROVALS - SIGNATURE & DATE			REV. NO. OR NEW CALC NO.	SUPERSEDES CALC NO. OR REV NO.	CONFIRMATION REQUIRED ☑	
PREPARER(S)/DATE(S)	REVIEWER(S)/DATES(S)	INDEPENDENT REVIEWER(S)/DATE(S)			YES	NO
Original Signed By: PJTrudeau / 5-31-01	Original Signed By: TYChang / 5-31-01	Original Signed By: TYChang / 5-31-01	8	7		✓
PJTrudeau / 7-26-01 Paul J. Trudeau	TYChang / 7-26-01 Thomas Y. Chang	TYChang / 7-26-01 Thomas Y. Chang	9	8		✓

DISTRIBUTION

GROUP	NAME & LOCATION	COPY SENT (✓)	GROUP	NAME & LOCATION	COPY SENT (✓)
RECORDS MGT. FILES (OR FIRE FILE IF NONE)	JOB BOOK R4.2G	ORIG			
	FIRE FILE - Denver	☑			
Geotechnical	PJTrudeau – Stoughton/3	☑			

TABLE OF CONTENTS

TABLE OF CONTENTS 3
RECORD OF REVISIONS 5
OBJECTIVE OF CALCULATION 9
ASSUMPTIONS/DATA 9
GEOTECHNICAL PROPERTIES 10
METHOD OF ANALYSIS 12
DESCRIPTION OF LOAD CASES 12
OVERTURNING STABILITY OF THE CASK STORAGE PADS 13
SLIDING STABILITY OF THE CASK STORAGE PADS 15
Design Issues Related to Sliding Stability of the Cask Storage Pads 15
Sliding Stability at Interface Between In Situ Clayey Soils and Bottom of Soil Cement Beneath the Pads 18
Active Earth Pressure 19
Dynamic Earth Pressure 19
Weights 21
Earthquake Accelerations – PSHA 2,000-Yr Return Period 21
Cask Earthquake Loadings 21
Foundation Pad Earthquake Loadings 21
Soil Cement Beneath Pad Earthquake Loadings 21
Case III: 0% N-S, -100% Vertical, 100% E-W (Earthquake Forces Act Upward) 22
Case IV: 0% N-S, 100% Vertical, 100% E-W (Earthquake Forces Act Downward) 22
Minimum Shear Strength Required at the Base of the Pads to Provide a Factor of Safety of 1.1 23
Adhesion between the Base of Pad and Underlying Clayey Soils 24
Limitation of Strength of Soil Cement Beneath the Pads 27
Sliding Along Contact Between the Concrete Pad and the Underlying Soil Cement 28
Soil Cement Above the Base of the Pads 29
Sliding Resistance of Entire N-S Column of Pads 31
Determine Residual Strength Required Along Base of Entire Column of Pads in N-S Direction, Assuming Full Passive Resistance is Provided by 250 psi Soil Cement Adjacent to Last Pad in Column 33
Sliding Resistance of Last Pad in Column of Pads ("Edge Effects") 35
Width of Soil Cement Adjacent to Last Pad to Provide Full Passive Resistance 35
Sliding Stability of the Pads Assuming Resistance Is Based on Only Frictional Resistance Along Base Plus Passive Resistance 36
Summary of Horizontal Displacements Calculated Based on Newmark's Method for assumption that Cask Storage Pads Are Founded Directly on Cohesionless Soils with ϕ = 17° and Passive Pressure Due to Site Soils Acts on 5-ft Thick Layer of Soil Cement at End of Row of 20 Pads 45
Evaluation of Sliding on Deep Slip Surface Beneath Pads 46
Estimation of Horizontal Displacement using Newmark's Method 46
Maximum Ground Motions 47
Load Cases 48

Ground Motions for Analysis 48
Summary of Horizontal Displacements Calculated Based on Newmark's Method for assumption that Cask Storage Pads Are Founded Directly on Cohesionless Soils with $\phi = 30°$ and No Soil Cement 50
ALLOWABLE BEARING CAPACITY OF THE CASK STORAGE PADS 52
Bearing Capacity Factors 52
Shape Factors (for *L>B*) 53
Depth Factors (for $\frac{D_f}{B} \leq 1$) 53
Inclination Factors 53
Static Bearing Capacity of the Cask Storage Pads 53
Dynamic Bearing Capacity of the Cask Storage Pads 57
Based on Inertial Forces 57
Based on Maximum Cask Dynamic Forces from the SSI Analysis 73
CONCLUSIONS 99
OVERTURNING STABILITY OF THE CASK STORAGE PADS 99
SLIDING STABILITY OF THE CASK STORAGE PADS 99
ALLOWABLE BEARING CAPACITY OF THE CASK STORAGE PADS 101
Static Bearing Capacity of the Cask Storage Pads 101
Dynamic Bearing Capacity of the Cask Storage Pads 101
REFERENCES: 103
TABLES 105
FIGURES 109

ATTACHMENT A	Telcon 6-19-97 SMM to PJT Dynamic Bearing Capacity of Pad	1 page
ATTACHMENT B	Pages from Calc 05996.02-G(PO17)-2, Rev. 3 providing maximum cask dynamic loads.	14 pages
ATTACHMENT C	Pages from Calc 05996.02-G(B)-05-2 providing basis for undrained strength used for dynamic bearing capacity analyses.	3 pages
ATTACHMENT D	Annotated Copies of Direct Shear Test Plots of Horizontal Displacement vs Shear Stress	3 pages
ATTACHMENT E	Triaxial Test Plot from Attachment 8 of Appendix 2A of SAR	1 page

CALCULATION IDENTIFICATION NUMBER				PAGE 5
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

RECORD OF REVISIONS

REVISION 0

Original Issue

REVISION 1

Revision 1 was prepared to incorporate the following:

- Revised cask weights and dimensions
- Revised earthquake accelerations
- Determine q_{all} as a function of the coefficient of friction between casks and pad.

REVISION 2

To add determination of dynamic bearing capacity of the pad for the loads and loading cases being analyzed by the pad designer. These include the 2-cask, 4-cask, and 8-cask cases. See Attachment A for background information, as well as bearing pressures for the 2-cask loading.

REVISION 3

The bearing pressures and the horizontal forces due to the design earthquake for the 2-cask case that are described in Attachment A are superseded by those included in Attachment B. Revision 3 also adds the calculation of the dynamic bearing capacity of the pad for the 4-cask and 8-cask cases and revises the cask weight to 356.5 K, which is based on Holtec HI-Storm Overpack with loaded MPC-32 (heaviest assembly weight shown on Table 3.2.1 of HI-Storm TSAR, Report HI-951312 Rev. 1 – p. C3, Calculation 05996.01-G(B)-05, Rev 0).

REVISION 4

Updated section on seismic sliding resistance of pads (pp 11-14F) using revised ground accelerations associated with the 2,000-yr return period design basis ground motion (horizontal = 0.528 g; vertical = 0.533 g) and revised soil parameters (c = 1,220 psf; ϕ = 24.9°, based on direct shear tests that are included in Attachments 7 and 8 of Appendix 2A of the SAR.). The horizontal driving forces used in this analysis (EQhc and EQhp) are based on the higher ground accelerations associated with the deterministic design basis ground motion (0.67g horizontal and 0.69g vertical). These forces were not revised for the lower ground accelerations associated with the 2,000-yr return period design basis ground motion (0.528g horizontal and 0.533g vertical) and, thus, this calculation will require confirmation at a later date.

Added a section on sliding resistance along a deeper slip plane (i.e., on cohesionless soils) beneath the pads.

Updated section on dynamic bearing capacity of pad for 8-cask case (pp 38-46). Inserted pp 46A and 46B. This case was examined because it previously yielded the lowest q_{all}

among the three loading cases (i.e., 2-cask, 4-cask, and 8-cask). The updated section shows a calculation of q_{all} based on revised soil parameters (c and ϕ). Note: this analysis will require confirmation and may be updated using revised vertical soil bearing pressures and horizontal shear forces, based on the lower ground accelerations associated with the 2,000-yr return period design basis ground motion (0.528g horizontal, and 0.533g vertical).

Modified/updated conclusions.

NOTE: SYBoakye prepared/DLAloysius reviewed pp 14 through 14F.

Remaining pages prepared by DLAloysius and reviewed by SYBoakye.

REVISION 5

Major re-write of the calculation.

1. Renumbered pages and figures to make the calculation easier to follow.
2. Incorporated dynamic loads due to revised design basis ground motion (PSHA 2,000-yr return period earthquake), as determined in CEC Calculation 05996.02-G(PO17)-2, Rev 0, and removed "Requires Confirmation".
3. Added overturning analysis.
4. Added analysis of sliding stability of cask storage pads founded on and within soil cement.
5. Revised dynamic bearing capacity analyses to utilize only total-stress strength parameters because these partially saturated soils will not have time to drain fully during the rapid cycling associated with the design basis ground motion. See Calculation 05996.02-G(B)-05-1 (SWEC, 2000a) for additional details.
6. Added reference to foundation profiles through pad emplacement area presented in SAR Figures 2.6-5, Sheets 1 through 14.
7. Changed "Load Combinations" to "Load Cases" and defined these cases to be consistent throughout the various stability analyses included herein. These are the same cases as are used in the stability analyses of the Canister Transfer Building, Calculation 05996.02-G(B)-13-2 (SWEC, 2000b).
8. Revised conclusions to reflect results of these changes.

REVISION 6

1. Added "References" section.
2. Revised shear strength used in the sliding stability analyses of the soil cement/silty clay interface to be the strength measured in the direct shear tests performed on samples obtained from depths of ~5.8 ft in the pad emplacement area. The shear strength equaled that measured for stresses corresponding to the vertical stresses at the bottom of the fully loaded cask storage pads.
3. Removed static and dynamic bearing capacity analyses based on total-stress strengths and added dynamic bearing capacity analyses based on c_u = 2.2 ksf..

Revised method of calculating the inclination factor in the bearing capacity analyses to that presented by Vesic in Chapter 3 of Winterkorn and Fang (1975). Vesic's method expands upon the theory developed by Hansen for plane strain analyses of footings with inclined loads. Vesic's method permits a more rigorous analysis of inclined loads acting in two directions on rectangular footings, which more closely represents the conditions applicable for the cask storage pads.

REVISION 7

1. Updated stability analyses to reflect revised design basis ground motions (a_H = 0.711g & a_V = 0.695g, per Table 1 of Geomatrix, 2001).
2. Resisting moment in overturning stability analysis calculated based on resultant of static and dynamic vertical forces.
3. Added analysis of sliding of an entire column of pads supported on at least 1' of soil cement, using an adhesion factor of 0.5 for the interface between the soil cement and the underlying silty clay layer.
4. Added discussion of strength limitations of the soil cement under the cask storage pads to comply with the maximum modulus of elasticity requirements of the materials supporting the pad in the hypothetical cask tipover analysis.
5. Changed pad length to 67 ft and pad embedment to 3 ft, in accordance with design change identified in Figure 4.2-7, "Cask Storage Pads," of SAR Revision 21.
6. Added definition of "m" used in the inclination factors for calculating allowable bearing capacity.
7. Updated references to supporting calculations.
8. Updated discussions and conclusions to incorporate revised results.

REVISION 8

1. Revised analyses of the stability of the storage pads to include a clear identification of the potential failure modes and failure surfaces and the material strengths required to satisfy the regulatory requirement, considering the critical failure modes and failure surfaces.
2. Added assessment of the edge effects of the last pad in the column of pads on the stability of the storage pads under the new seismic loads.
3. Horizontal cask earthquake forces in the dynamic bearing capacity calculations were changed to limit the resultant of the two horizontal components to the coefficient of friction between the cask and the top of the pad x the effective weight of the casks.
4. Reduced shear strength of clayey soils beneath the pads to 95% of peak shear strength measured in direct shear tests in analyses that included both shear resistance along base of sliding mass and passive resistance. This 5% reduction of peak strength to residual strengths is the maximum reduction measured in the three direct shear tests that were performed on these clayey soils for specimens confined at 2 ksf, which corresponds to the approximate final effective stress at the base of the pads.

REVISION 9

1. Revised unit weights of soil cement to reflect measured values obtained from ongoing laboratory testing program. Unit weight of soil cement adjacent to the pads exceeds 110 pcf and the cement-treated soil beneath the pads exceeds 100 pcf.
2. Added clarification of approximations used in calculation of K_{AE} and updated calculation of K_{AE} to remove excess conservatism inherent in the previous use of approximations "sin $(\phi - \theta) \approx 0$" and "cos $(\phi - \theta) \approx 1$".
3. Added inertial forces due to 2-ft thick layer of soil cement beneath pad to sliding stability analysis.
4. Added analysis of hypothetical case where resistance to sliding is comprised of frictional resistance along base of pads and soil cement + passive resistance. This analysis demonstrates that the factor of safety against sliding is less than 1.1. Also added analysis to estimate the maximum pad displacement for these very conservative assumptions. This analysis shows that the resulting maximum horizontal displacements, if they were to occur due to the earthquake, would be of no safety consequence to the pads or the casks.
5. Added Attachment E, plot of Total Stress Mohr's Circles from triaxial tests performed on samples from Boring B-1.

OBJECTIVE OF CALCULATION

Evaluate the static & seismic stability of the cask storage pad foundations at the proposed site. The failure modes investigated include overturning stability, sliding stability, and bearing capacity for static loads & for dynamic loads due to the design basis ground motion (PSHA 2,000-yr return period earthquake with peak horizontal ground acceleration of 0.711g).

Other potential failure modes are addressed elsewhere. Evaluation of static settlements are addressed in Calculation 05996.02-G(B)-3-3, which is supplemented by Calculation 05996.02-G(B)-21-0. Dynamic settlements are addressed in Calculation 05996.02-G(B)-11-3. The soils underlying the site are not susceptible to liquefaction, as documented in Calculation 05996.01-G(B)-6-1.

Evaluation of floatation of these pads is not required because they will never be submerged, since groundwater is approximately 125 ft below the ground surface at the site. In addition, as indicated in SAR Section 2.4.8, Flooding Protection Requirements,

> *"All Structures, Systems, and Components (SSCs) classified as being Important to Safety are protected from flooding by diversion berms to deflect potential flows generated by PMF from both the east mountain range (Basin A) and the west mountain range (Basin B) watersheds."*

The design of the concrete pad, to ensure that it will not suffer bending or shear failures due to static and dynamic loads, is addressed in Calculation 05996.02-G(PO17)-2-3 (CEC, 2001).

ASSUMPTIONS/DATA

The arrangement of the cask storage pads is shown on SAR Figure 1.2-1. The spacing of the pads is such that each N-S column of pads may be treated as one long strip footing with B/L ~ 0 & B=30 ft for the bearing capacity analyses.

The E-W spacing of the pads is great enough that adjacent pads will not significantly impact the bearing capacity of one another, as shown on Figure 1, "Foundation Plan & Profile."

The generalized soil profile, presented in Figure 1, indicates the soil profile consists of ~30 ft of silty clay/clayey silt with some sandy silt (Layer 1), overlying ~30 ft of very dense fine sand (Layer 2), overlying extremely dense silt (N ≥100 blows/ft, Layer 3). SAR Figures 2.6-5 (Sheets 1 through 14) present foundation profiles showing the relationship of the cask storage pads with respect to the underlying soils. These profiles, located as shown in SAR Figure 2.6-19, provide more detailed stratigraphic information, especially within the upper ~30-ft thick layer at the site.

Figure 1 also illustrates the coordinate system used in these analyses. Note, the X-direction is N-S, the Y-direction is vertical, and the Z-direction is E-W. This is the same coordinate system that is used in the stability analyses of the Canister Transfer Building (Calculation 05996.02-G(B)-13-2, SWEC, 2000b).

The bearing capacity analyses assume that Layer 1, which consists of silty clay/clayey silt with some sandy silt, is of infinite thickness and has strength properties based on those measured at depths of ~10 ft for the clayey soils within the upper layer. These assumptions simplify the analyses and they are very conservative. With respect to bearing capacity, the strength of the sandy silt in the upper layer is greater than that of the clayey soils, based on the increases in Standard Penetration Test (SPT) blow counts (N-values) and the increased tip resistance (see SAR Figures 2.6-5) in the cone penetration testing (ConeTec, 1999) noted in these soils. The underlying soils are even stronger, based on their SPT N-values, which generally exceed 100 blows/ft.

Based on probabilistic seismic hazard analysis, the peak acceleration levels of 0.711g for horizontal ground motion and 0.695g for the vertical ground motion were determined as the design bases of the PFSF for a 2,000-yr return period earthquake (Geomatrix Consultants, Inc, 2001).

GEOTECHNICAL PROPERTIES

Based on laboratory test results presented in Tables 2, 3, and 4 of Calculation 05996.02-G(B)-05-2 (SWEC, 2000a),

γ_{moist} = 80 pcf is a conservative lower-bound value of the unit weight for the soils underlying the pad emplacement area.

The bearing capacity of the structures are dependant primarily on the strength of the soils in the upper ~25 to ~30-ft layer at the site. All of the borings drilled at the site indicate that the soils underlying this upper layer are very dense fine sands overlying silts with standard penetration test blow counts that exceed 100 blows/ft. The results of the cone penetration testing, presented in ConeTec(1999) and plotted in SAR Figure 2.6-5, Sheets 1 to 14, illustrate that the strength of the soils in the upper layer are much greater at depths below ~10 ft than in the range of ~5 ft to ~10 ft, where most of the triaxial tests were performed.

In practice, the average shear strength along the anticipated slip surface of the failure mode should be used in the bearing capacity analysis. This slip surface is normally confined to within a depth below the footing equal to the minimum width of the footing. In this case, the effective width of the footing is decreased because of the large eccentricity of the load on the pads due to the seismic loading. As indicated in Table 2.6-7, the minimum effective width occurs for Load Cases II and IIIB, where B' ~15 ft. Figure 7 illustrates that the anticipated slip surface of the bearing capacity failure would be limited to the soils within the upper half of the upper layer. Therefore, in the bearing capacity analyses presented herein, the undrained strength measured in the UU triaxial tests was not increased to reflect the increase in strength observed for the deeper-lying soils in the cone penetration testing.

CALCULATION IDENTIFICATION NUMBER				PAGE 15
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

SLIDING STABILITY OF THE CASK STORAGE PADS

The factor of safety (FS) against sliding is defined as follows:

FS = resisting force ÷ driving force

For this analysis, ignoring passive resistance of the soil (soil cement) adjacent to the pad, the resisting, or tangential force (T), below the base of the pad is defined as follows:

$T = N \tan \phi + c\,B\,L$

where, N (normal force) = $\sum F_v = W_c + W_p + EQ_{vc} + EQ_{vp}$

$\phi = 0°$ (for Silty Clay/Clayey Silt)

c = 2.1 ksf, as indicated on p C-2.

B = 30 feet

L = 67 feet

DESIGN ISSUES RELATED TO SLIDING STABILITY OF THE CASK STORAGE PADS

Figure 3 presents a detail of the soil cement under and adjacent to the cask storage pads. Figure 8 presents an elevation view, looking east, that is annotated to facilitate discussion of potential sliding failure planes. The points referred to in the following discussion are shown on Figure 8.

1. Ignoring horizontal resistance to sliding due to passive pressures acting on the sides of the pad (i.e., Line AB or DC in Figure 8), the shear strength must be at least 1.60 ksf (11.10 psi) at the base of the cask storage pad (Line BC) to obtain the required minimum factor of safety against sliding of 1.1.
2. The static, undrained strength of the clayey soils exceeds 2.1 ksf (14.58 psi). This shear strength, acting only on the base of the pad, provides a factor of safety of 1.27 against sliding along the base (Line BC). This shear strength, therefore, is sufficient to resist sliding of the pads if the full strength can be engaged to resist sliding.
3. Ordinarily a foundation key would be used to ensure that the full strength of the soils beneath a foundation are engaged to resist sliding. However, the hypothetical cask tipover analysis imposes limitations on the thickness and stiffness of the concrete pad that preclude addition of a foundation key to ensure that the full strength of the underlying soils is engaged to resist sliding.
4. PFS will use a layer of soil cement beneath the pads (Area HITS) as an "engineered mechanism" to bond the pads to the underlying clayey soils.
5. The hypothetical cask tipover analysis imposes limitations on the stiffness of the materials underlying the pad. The thickness of the soil cement beneath the pads is limited to 2 ft and the static modulus of elasticity is limited to 75,000 psi.

6. The modulus of elasticity of the soil cement is directly related to its strength; therefore, its strength must be limited to values that will satisfy the modulus requirement. This criterion limits the unconfined compressive strength of the soil cement beneath the pads to 100 psi.

7. Therefore, the pads will be constructed on a layer of soil cement that is at least 1-ft thick, but no thicker than 2-ft, that extends over the entire pad emplacement area, as delineated by Area HITS.

8. The unconfined compressive strength of the soil cement beneath the pads is designed to provide sufficient shear strength to ensure that the bond between the concrete comprising the cask storage pad and the top of the soil cement (Line BC) and the bond between the soil cement and the underlying clayey soils (Line JK) will exceed the full, static, undrained strength of those soils. To ensure ample margin over the minimum shear strength required to obtain a factor of safety of 1.1, the unconfined compressive strength of the soil cement beneath the pads (Area HITS) will be at least 40 psi.

9. DeGroot (1976) indicates that this bond strength can be easily obtained between layers of soil cement, based on nearly 300 laboratory direct shear tests that he performed to determine the effect of numerous variables on the bond between layers of soil cement.

10. Soil cement also will be placed between the cask storage pads, above the base of the pads, in the areas labeled FGBM and NCQP. This soil cement is NOT required to resist sliding of the pads, because there is sufficient shear strength at the interfaces between the concrete pad and the underlying soil cement (Line BC) and between that soil-cement layer and the underlying clayey soils (Line JK) that the factor of safety against sliding exceeds the minimum required value.

11. The pads are being surrounded with soil cement so that PFS can effectively use the eolian silt found at the site to provide an adequate subbase for support of the cask transporter, as well as to provide additional margin against any potential sliding.

12. The actual unconfined compressive strength and mix requirements for the soil cement around the cask storage pads will be based on the results of standard soil-cement laboratory tests.

13. 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).

The analysis presented on the following pages demonstrates that the static, undrained strength of the in situ clayey soils is sufficient to preclude sliding (FS = 1.27 vs minimum required value of 1.1), provided that the full strength of the clayey soils is engaged. The soil-cement layer beneath the pads provides an "engineered mechanism" to ensure that the full, static, undrained strength of the clayey soils is engaged in resisting sliding forces. It also demonstrates that the bond between this soil-cement layer and the base of the concrete pad will be stronger than the static, undrained strength of the in situ clayey soils and, thus, the interface between the in situ soils and the bottom of the soil-cement layer is the weakest link in the system. Since this "weakest link" has an adequate factor of safety against sliding, the overlying interface between the soil cement and the base of the pad will have a greater factor of safety against sliding. Therefore, the factor of safety against sliding of the overall cask storage pad design is at least 1.27.

Sliding Stability at Interface Between In Situ Clayey Soils and Bottom of Soil Cement Beneath the Pads

Material under and around the pad will be soil cement. In this analysis, however, the presence of the soil cement adjacent to the sides of the pads is ignored to demonstrate that there is an acceptable factor of safety against sliding of the pads along the interface between in situ clayey soils and bottom of soil cement beneath the pads. The potential failure mode is sliding along the surface at the base of the pad. No credit is taken for the passive resistance acting on the sides of the pad above the base. This analysis is applicable for any of the pads at the site, including those at the ends of the rows or columns of pads, since it relies only on the strength of the material beneath the pads to resist sliding.

This analysis conservatively assumes that 100% of the dynamic forces due to the earthquake act in both the horizontal and vertical directions at the same time. The length of the pad in the N-S direction (67 ft) is greater than twice the width in the E-W direction (30 ft); therefore, the dynamic active earth pressures acting on the length of the pad will be greater than those acting on the width, and the critical direction for sliding will be E-W, since passive resistance is ignored.

The soil cement is assumed to have the following properties in calculation of the dynamic active earth pressure acting on the pad from the soil cement above the base of the pad:

γ = 100-110 pcf — Initial results of the soil-cement testing indicate that 110 pcf is a reasonable lower-bound value for the total unit weight of the soil cement adjacent to the pads and that 100 pcf is a reasonable lower-bound value for the total unit weight of the cement-treated soil to be placed beneath the pads.

ϕ = 40° — Tables 5 & 6 of Nussbaum & Colley (1971) indicate that ϕ exceeds 40° for all A-4 soils (CL & ML, similar to the eolian silts at the site) treated with cement; therefore, it is likely that ϕ will be higher than this value. This value also is used in this analysis only for determining upper-bound estimates of the active earth pressure acting on the pad due to the design basis ground motion. Because of the magnitude of the earthquake, this analysis is not sensitive to increases in this value.

H = 5 ft — As shown in SAR Figure 4.2-7, the pad is 3 ft thick, and it is constructed such that top of the pad is at the final ground surface (i.e., pads are embedded 3' below grade). Soil cement beneath the pad is 1-ft to 2-ft thick. The dynamic forces (active earth pressure + horizontal inertial forces) are greater for deeper depth of soil cement. Therefore, analyze for 2 ft of soil cement beneath the pad.

ACTIVE EARTH PRESSURE

$P_a = 0.5\,\gamma\,H^2\,K_a$

$K_a = (1 - \sin\phi)/(1 + \sin\phi) = 0.22$ for $\phi = 40°$ for the soil cement, ignoring cohesion (very conservative).

$P_{a\,E\text{-}W}$ = [0.5 x 0.11 kcf x (5 ft)^2 x 0.22] x 67 ft (length)/storage pad = 20.3 K E-W.

$P_{a\,N\text{-}S}$ = [0.5 x 0.11 kcf x (5 ft)^2 x 0.22] x 30 ft (width)/storage pad = 9.1 K N-S.

DYNAMIC EARTH PRESSURE

As indicated on p 11 of GTG 6.15-1 (SWEC, 1982), for active conditions, the combined static and dynamic lateral earth pressure coefficient is computed according to the analysis developed by Mononobe-Okabe and described in Seed and Whitman (1970) as:

$$K_{AE} = \frac{(1-\alpha_V)\cdot\cos^2(\phi-\theta-\alpha)}{\cos\theta\cdot\cos^2\alpha\cdot\cos(\delta+\alpha+\theta)\cdot\left[1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\theta-\beta)}{\cos(\delta+\alpha+\theta)\cdot\cos(\beta-\alpha)}}\right]^2}$$

whe re :

$$\theta = \tan^{-1}\left(\frac{\alpha_H}{1-\alpha_V}\right)$$

β = slope of ground behind wall,
α = slope of back of wall to vertical,
α_H = horizontal seismic coefficient, where a positive value corresponds to a horizontal inertial force directed toward the wall,
α_V = vertical seismic coefficient, where a positive value corresponds to a vertical inertial force directed upward,
δ = angle of wall friction,
ϕ = friction angle of the soil,
g = acceleration due to gravity.

The combined static and dynamic active earth pressure force, P_{AE}, is calculated as:

$$P_{AE} = \frac{1}{2}\gamma H^2 K_{AE}, \text{ where:}$$

γ = unit weight of soil,
H = wall height, and
K_{AE} is calculated as shown above.

To simplify the analysis, assume $\delta = 0$. This is conservative, as illustrated in Figure 12 of Seed and Whitman (1970), which indicates that K_{AE} decreases with increasing values of δ.

$\beta = \alpha = 0$

$$\theta = \tan^{-1}\left(\frac{0.711}{1-0.695}\right) = 66.8^\circ$$

$\phi = 40^\circ$

To obtain a real solution to the equation for calculating K_{AE}, the sin $(\phi - \theta - \beta)$ must be positive; i.e., the sin $(\phi - \theta - \beta)$ can vary from 0 to 1. Because it is in the denominator of K_{AE}, K_{AE} will be greatest when it = 0. Therefore, assume sin $(\phi - \theta - \beta) \approx 0$.

Similarly, approximate cos $(\phi - \theta - \alpha) \approx 1$. This term is in the numerator of K_{AE}, and K_{AE} will be maximum when cos $(\phi - \theta - \alpha) = 1$; therefore, approximating it equals 1 is conservative.

With these approximations,

$$K_{AE} = \frac{1 - \alpha_V}{\cos\theta \cdot \cos\theta}$$

$$\therefore \quad K_{AE} = \frac{1 - 0.695}{\cos^2 66.8^\circ} = 1.97$$

Therefore, the combined static and dynamic active lateral earth pressure force at the base of the 3 ft pad is:

$$F_{AE_{E\text{-}W}} = P_{AE} = \frac{1}{2} \times \underset{\gamma}{0.110\text{ kcf}} \times \underset{H^2}{(3\text{ ft})^2} \times \underset{K_{AE}}{1.97} \times \underset{L}{67\text{ ft}}\text{ / storage pad} = 65.3\text{ K in the E - W direction.}$$

$$F_{AE_{N\text{-}S}} = P_{AE} = \frac{1}{2} \times 0.110\text{ kcf} \times (3\text{ ft})^2 \times 1.97 \times 30\text{ ft / storage pad} = 29.3\text{ K in the N - S direction.}$$

The combined static and dynamic active lateral earth pressure force at the base of the 3 ft pad and underlying 2 ft of soil cement is:

$$F_{AE_{E\text{-}W}} = P_{AE} = \frac{1}{2} \times \underset{\gamma}{0.110\text{ kcf}} \times \underset{H^2}{(5\text{ ft})^2} \times \underset{K_{AE}}{1.97} \times \underset{L}{67\text{ ft}}\text{ / storage pad} = 181.5\text{ K in the E - W direction.}$$

$$F_{AE_{N\text{-}S}} = P_{AE} = \frac{1}{2} \times 0.110\text{ kcf} \times (5\text{ ft})^2 \times 1.97 \times 30\text{ ft / storage pad} = 81.3\text{ K in the N - S direction.}$$

WEIGHTS

Casks: $W_c = 8 \times 356.5$ K/cask = 2,852 K

Pad: $W_p = 3$ ft x 67 ft x 30 ft x 0.15 kips/ft^3 = 904.5 K

Soil Cement Beneath Pad: $W_{sc} = 2$ ft x 67 ft x 30 ft x 0.10 kips/ft^3 = 402 K

EARTHQUAKE ACCELERATIONS – PSHA 2,000-YR RETURN PERIOD

a_H = horizontal earthquake acceleration = 0.711g

a_V = vertical earthquake acceleration = 0.695g

CASK EARTHQUAKE LOADINGS

EQvc = -0.695 x 2,852 K = -1,982 K (minus sign signifies uplift force)

$EQhc_{E\text{-}W}$ = 2,212 K (acting short direction of pad, E-W) $Q_{xd\ max}$ in Table D-1(c) in Att B

$EQhc_{N\text{-}S}$ = 2,102 K (acting in long direction of pad, N-S) $Q_{yd\ max}$ in Table D-1(c) "

Note: These maximum horizontal dynamic cask driving forces are from Calc 05996.02-G(PO17)-2, (CEC, 2001), and they apply only when the dynamic forces due to the earthquake act downward and the coefficient of friction between the cask and the pad equals 0.8. $EQh_{c\ max}$ is limited to a maximum value of 696 K for Case III, based on the upper-bound value of μ = 0.8, as shown in the following table:

Cask Loads	WT K	EQ_{Vc} K	N K	0.2 x N K	0.8 x N K	$EQ_{hc\ max}$ K
Case III – Uplift	2,852	-1,982	870	174	696	**696**
Case IV - EQ_v Down	2,852	1,982	4,834	967	3,867	**2,212** E-W 2,102 N-S

Note:

Case III: 0% N-S, -100% Vertical, 100% E-W Earthquake Forces Act Upward

Case IV: 0% N-S, 100% Vertical, 100% E-W Earthquake Forces Act Downward

FOUNDATION PAD EARTHQUAKE LOADINGS	**SOIL CEMENT BENEATH PAD EARTHQUAKE LOADINGS**
EQvp = -0.695 x 904.5 K = -629 K	EQvsc = -0.695 x 402 K = -279.4 K
EQhp = 0.711 x 904.5 K = 643 K	EQhp = 0.711 x 402 K = 285.8 K

CASE III: 0% N-S, -100% VERTICAL, 100% E-W (EARTHQUAKE FORCES ACT UPWARD)

When EQvc and EQvp act in an upward direction (Case III), tending to unload the pad, sliding resistance is obtained as follows:

$$N = \underset{Wc}{2{,}852\text{ K}} + \underset{Wp}{904.5\text{ K}} + \underset{Wsc}{402\text{ K}} + \underset{EQvc}{(-1{,}982\text{ K})} + \underset{EQvp}{(-629\text{ K})}\,) + \underset{EQvsc}{(-279.4\text{ K})} = 1{,}268.6\text{ K}$$

$$T = \underset{N}{1{,}268.6\text{ K}} \times \tan \underset{\phi}{0°} + \underset{c}{2.1\text{ ksf}} \times \underset{B}{30\text{ ft}} \times \underset{L}{67\text{ ft}} = 4{,}221\text{ K}$$

The driving force, V, is defined as:

$$V = F_{AE} + EQhp + Eqhc + EQhsc$$

The factor of safety against sliding is calculated as follows:

$$FS = \underset{T}{4{,}221\text{ K}} \div (\underset{F_{AE\,E\text{-}W\,5'}}{181.5\text{ K}} + \underset{EQhp}{643\text{ K}} + \underset{EQhc}{696\text{ K}} + \underset{EQhsc}{285.8\text{ K}}) = \mathbf{2.34}$$

$$(1{,}806.3\text{ K})$$

For this analysis, the value of the horizontal driving force due to the earthquake, EQhc, is limited to the upper-bound value of the coefficient of friction, $\mu = 0.8$, x the cask normal load, because if EQhc exceeds this value, the cask will slide. The factor of safety exceeds the minimum allowable value of 1.1; therefore the pads plus 2-ft block of soil cement beneath them are stable with respect to sliding for this load case. The factor of safety against sliding is higher than this if the lower-bound value of μ is used (= 0.2), because the driving forces due to the casks would be reduced.

CASE IV: 0% N-S, 100% VERTICAL, 100% E-W (EARTHQUAKE FORCES ACT DOWNWARD)

When the earthquake forces act in the downward direction:

$$T = N \tan \phi + [c\, B\, L]$$

where, N (normal force) = Σ Fv = Wc + Wp + EQvc + Eqvp + EQvsc

$$N = \underset{Wc}{2{,}852\text{ K}} + \underset{Wp}{904.5\text{ K}} + \underset{EQvc}{1{,}982\text{ K}} + \underset{EQvp}{629\text{ K}} + \underset{Eqvsc}{279.4\text{ K}} = 6{,}647\text{ K}$$

$$T = \underset{N}{6{,}647\text{ K}} \times \tan \underset{\phi}{0°} + \underset{c}{2.1\text{ ksf}} \times \underset{B}{30\text{ ft}} \times \underset{L}{67\text{ ft}} = 4{,}221\text{ K}$$

The driving force, V, is defined as:

$$V = F_{AE} + EQhp + Eqhc + EQhsc$$

The factor of safety against sliding is calculated as follows:

$$\mathbf{FS}_{\textbf{Soil Cement to Clayey Soil}} = \overset{T}{4{,}221\text{ K}} \div \underbrace{(\overset{F_{AE\,E\text{-}W\,5'}}{181.5\text{K}} + \overset{EQhp}{643\text{ K}} + \overset{EQhc_{E\text{-}W}}{2{,}212\text{ K}} + \overset{EQh_{SC}}{285.8\text{ K}})}_{(3{,}322.3\text{ K})} = \underline{\mathbf{1.27\ (=Min)}}$$

The factor of safety against sliding is higher than this if the lower-bound value of μ is used (= 0.2), because the driving forces due to the casks would be reduced.

Ignoring the passive resistance acting on the sides of the pad, the resistance to sliding is the same in both directions; therefore, for this analysis, the larger value of EQhc (i.e., acting in the E-W direction) was used. Even with these conservative assumptions, the factor of safety exceeds the minimum allowable value of 1.1; therefore the pads overlying 2 ft of soil cement are stable with respect to sliding for this load case, assuming the strength of the cement-treated soils underlying the pad is at least as high as the undrained strength of the underlying soils.

MINIMUM SHEAR STRENGTH REQUIRED AT THE BASE OF THE PADS TO PROVIDE A FACTOR OF SAFETY OF 1.1

The minimum shear strength required at the base of the pads to provide a factor of safety of 1.1 is calculated as follows:

$$FS = \overset{T}{T} \div \underbrace{(\overset{F_{AE\,E\text{-}W\,3'}}{65.3\text{ K}} + \overset{EQhp}{643\text{ K}} + \overset{EQhc_{E\text{-}W}}{2{,}212\text{ K}})}_{(2{,}920.3\text{ K})} \geq 1.1$$

$$\rightarrow \quad T \geq 1.1 \times 2{,}920.3\text{ K} = 3{,}212.3\text{ K}$$

Dividing this by the area of the pad results in the minimum acceptable shear strength at the base of the pad:

$$\tau = \frac{3{,}212.3\text{ K}}{30\text{ ft} \times 67\text{ ft}} = 1.60\,\frac{\text{K}}{\text{ft}^2} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 \times \frac{1{,}000\text{ lbs}}{\text{K}} = \mathbf{11.10\ psi}$$

ADHESION BETWEEN THE BASE OF PAD AND UNDERLYING CLAYEY SOILS

The preceding analysis demonstrates that the static undrained strength of the soils underlying the pads is sufficient to preclude sliding of the cask storage pads over 2 ft of soil cement for the 2,000-yr return period earthquake with a peak horizontal ground acceleration of 0.711g, conservatively ignoring the passive resistance acting on the sides of the pads. This analysis assumes that the full static undrained strength of the clay is engaged to resist sliding. To obtain the minimum factor of safety required against sliding of 1.1, 76% (= 1.60 ksf (required for FS=1.1) ÷ 2.1 ksf available) of the undrained shear strength must be engaged, or in other words, the adhesion factor between the base of the concrete storage pads plus 2 ft of soil cement and the surface of the underlying clayey soils must be 0.76. This adhesion factor, c_a, is higher than would normally be used, considering disturbance that may occur to the surface of the subgrade during construction. Therefore, an "engineered mechanism" is required to ensure that the full strength of the clayey soils is available to resist sliding of these pads on 2 ft of soil cement.

Ordinarily, a foundation key would be added to extend the shear plane below the disturbed zone and to ensure that the full strength of the clayey soils are available to resist sliding forces. However, adding a key to the base of the storage pads would increase the stiffness of the foundation to such a degree that it would exceed the target hardness limitation of the hypothetical cask tipover analysis. Therefore, PFS decided to construct the cask storage pads on (and within) a layer of soil cement constructed throughout the entire pad emplacement area.

As shown in Figure 3, the soil cement will extend to the bottom of the eolian silt or a minimum of 1 ft below the base of the storage pads and up the vertical face at least 2 ft. In the sliding stability analysis, it is required that the following interfaces be strong enough to resist the sliding forces due to the design earthquake. Working from the bottom up, these include:

1. The interface between the in situ clayey soils and the bottom of the soil cement, and
2. The top of the soil cement and the bottom of the concrete storage pad.

The purpose of soil cement below the pads is to provide the "engineered mechanism" required to effectively transmit the sliding forces down into the underlying clayey soils. The techniques used to construct soil cement are such that the bond between the soil cement and the underlying clayey soils will exceed the undrained strength of the underlying clayey soils.

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 nearly always exceeded 11.10 psi, the minimum required value of shear strength of the bond between the base of the pads and the underlying material. The minimum bond strength he reports, other than for the asphalt and chlorinated rubber surface treatments identified above, is 7.7 psi. This value applied for only one test (Sample No. 15R-149, Series No. 3, Spec. No. 12) that was performed on a sample that had no special surface treatment along the lift line. This test, however, was anomalous, since all of the other specimens in this series had bond strengths in excess of 38.5 psi. 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 did not use the cement surface treatment, the minimum bond strength was 47.7 psi, which greatly exceeds the bond strength (11.10 psi) required to obtain an adequate factor of safety against sliding of the pads without including the passive resistance acting on the sides of the pads.

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 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 greatly exceeds the bond strength (11.10 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 between the top of the soil cement and 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.

The shear strength available at each of the interfaces applicable to resisting sliding of the cask storage pads will exceed the undrained strength of the underlying clayey soils. PFS has committed (SAR p. 2.6-113) to performing laboratory tests during the design of the soil cement to demonstrate that the required shear strengths can be achieved at the various interfaces, and PFS has committed (SAR p. 2.6-114) to performing field tests during construction to demonstrate that the required shear strengths at these interfaces have been achieved.

The soil cement beneath the pads is used as an "engineered mechanism" to ensure that the full static undrained shear strength of the underlying clayey soils is engaged to resist sliding and, as shown above, the minimum factor of safety against sliding of the pads is very conservatively calculated as 1.27 when the static undrained strength of the clayey soils is fully engaged. This value exceeds the minimum value required for the factor of safety against sliding (=1.1); therefore, the pads constructed on top of a layer of soil cement have an adequate factor of safety against sliding.

LIMITATION OF STRENGTH OF SOIL CEMENT BENEATH THE PADS

As indicated in Figure 3, the soil cement will extend at least 1 ft below all of the cask storage pads, and, as shown in SAR Figures 2.6-5, Pad Emplacement Area Foundation Profiles, it will typically extend ~2 ft below most of the pads. Thus, the area available to resist sliding will greatly exceed that of the pads alone. The hypothetical cask tipover analysis imposes limitations on the modulus of elasticity of the soils underlying the pad. The modulus of elasticity of the soil cement is directly related to its strength; therefore, its strength must be limited to values that will satisfy the modulus requirement, but it must still provide an adequate factor of safety with respect to sliding of the pads embedded within the soil cement.

Table 5-6 of Bowles (1996) indicates E = 1,500 s_u, where s_u = the undrained shear strength. Note, s_u is half of q_u, the unconfined compressive strength.

Based on this relationship, E = 750 q_u,

Where E = Young's modulus

q_u = Unconfined compressive strength

An unconfined compressive strength of 100 psi for the soil cement under the pad will limit the modulus value to 75,000 psi. Thus, designing the soil cement to have an unconfined compressive strength that ranges from 40 psi to 100 psi will provide an adequate factor of safety against sliding and will limit the modulus of the soil cement under the pads to an acceptable level for the hypothetical cask tipover considerations.

SLIDING ALONG CONTACT BETWEEN THE CONCRETE PAD AND THE UNDERLYING SOIL CEMENT

The soil cement will be designed to have an unconfined compressive strength of at least 40 psi to ensure that it will be stronger than required to provide a factor of safety against sliding that exceeds the required minimum value of 1.1. The shear strength equals half of the unconfined compressive strength, 20 psi, which equals 2.88 ksf. Therefore, the resistance to sliding between the concrete storage pad and the top of the soil cement layer beneath the pad will be greater than:

$$T = \overset{N}{6{,}368\ \text{K}} \times \tan \overset{\phi}{0°} + \overset{c}{2.88\ \text{ksf}} \times \overset{B}{30\ \text{ft}} \times \overset{L}{67\ \text{ft}} = \overset{T}{5{,}789\ \text{K}}$$

As indicated above, the driving force, V, is defined as: $V = F_{AE} + EQhp + EQhc$

The factor of safety against sliding between the pad and the surface of the underlying soil cement is calculated as the resisting force ÷ the driving force, as follows:

$$\mathbf{FS_{Pad\ to\ Soil\ Cement}} = \overset{T}{5{,}789\ \text{K}} \div (\overset{F_{AE\ E\text{-}W}}{65.3\ \text{K}} + \overset{EQhp}{643\ \text{K}} + \overset{EQhc_{E\text{-}W}}{2{,}212\ \text{K}}) = \mathbf{1.98}$$

$$(2{,}920.3\ \text{K})$$

Thus, designing the soil cement to have an unconfined compressive strength of at least 40 psi results in an acceptable factor of safety against sliding between the concrete at the base of the pad and the surface of the underlying soil cement that exceeds the factor of safety between the bottom of the soil cement and the underlying clayey soils. In other words, the soil cement will have higher strength than the underlying silty clay/clayey silt layer; therefore, the resistance to sliding on that interface will be limited by the strength of the silty clay/clayey silt.

Soil cement with strengths higher than this are readily achievable, as illustrated by the lowest curve in Figure 4.2 of ACI 230.1R-90, which applies for fine-grained soils similar to the eolian silt in the pad emplacement area. Note, f_c = 40C where C = percent cement in the soil cement. Therefore, to obtain f_c >40 psi, the percentage of cement required would be ~40/40 = 1%. This is even less cement than would typically be used in constructing soil cement for use as road base. The resulting material will more likely be properly classified as a cement-treated soil, rather than a true soil cement. Because this material is located below the frost zone (which is only 30" below grade at the site), it does not need to comply with the durability requirements of soil cement; i.e., ASTM freeze/thaw and wet/dry tests. The design of the mix for this material will require that the unconfined compressive strength of this layer of material will exceed 40 psi to ensure that the shear strength available to resist sliding of the concrete pads exceeds the shear strength of the in situ clayey soils.

Soil Cement Above the Base of the Pads

Soil cement also will be placed between the cask storage pads, above the base of the pads. Earlier versions of this calculation demonstrated that this soil cement could be designed such that its compressive strength alone would be sufficient to resist all of the sliding forces due to the design earthquake. However, as shown above, this soil cement is NOT required to resist sliding of the pads, because there is sufficient shear strength at the interfaces between the concrete pad and the underlying soil cement and between that soil cement and the underlying clayey soils that the factor of safety against sliding exceeds the minimum required value. The pads are being surrounded with soil cement so that PFS can effectively use the eolian silt found at the site to provide an adequate subbase for support of the cask transporter. The eolian silt, otherwise, would be inadequate for this purpose and would require replacement with imported structural fill. The soil cement surrounding the pad may also help to spread the seismic load into the clayey soil outside the pad area to engage additional resistance against sliding of the pad. This effect would result in an increase in the factor of safety against sliding.

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).

The beneficial effect of this soil cement on the factor of safety against sliding can be estimated by considering that the passive resistance provided by this soil cement is available to resist sliding before a sliding failure can occur. In this case, the shear strength of the clayey soils under the pad may be reduced to the residual strength, because of the horizontal displacement required to reach the full passive state. Note, the soil cement is much stiffer than normal soils; therefore, these horizontal displacements will not be as high as they typically are for soils to reach the full passive state.

The results of the direct shear tests, presented as plots of shear stress vs horizontal displacement in Attachment 7 of Appendix 2A of the SAR (copies included in Attachment D), illustrate that the residual strength of these soils is nearly equal to the peak strength. Looking at the test results for the specimens that were tested at confining stresses comparable to the loading at the base of the cask storage pads, σ_v ~2 ksf, at horizontal displacements of ~0.025" past the peak strength, there is ~1.5% reduction in the shear strength indicated for Sample U-1C from Boring C-2. Also note that Boring C-2 was drilled within the pad emplacement area. The results for Sample U-1AA from Boring CTB-S showed no decrease in shear strength following the peak at ~0.025" horizontal displacement, and Samples U-3B&C from Boring CTB-6 showed a decrease of ~5%.

Based of these results, conservatively assume that the strength of the clayey soils beneath the soil cement layer underlying the pads is reduced by 5% to account for horizontal straining required to reach the full passive resistance of the soil cement adjacent to the pad. This results in resisting forces acting on the base of the soil cement layer beneath each pad of 0.95 x 2.1 ksf x 30 ft x 67 ft = 4,010 K.

Assuming the soil cement adjacent to the pad is constructed such that its unconfined compressive strength is 250 psi, its passive resistance acting on the 2'-4" thickness of soil cement adjacent to the pad will provide an additional force resisting sliding in the N-S direction of:

$$T_{SC\ \text{Adjacent to Pad}\ @\ N\&S} = 250\,\frac{\text{lbs}}{\text{in.}^2} \times \left(\frac{12\ \text{in.}}{\text{ft}}\right)^2 \times \frac{\text{K}}{1{,}000\ \text{lbs}} \times 2.33\ \text{ft} \times 30\ \text{ft} = 2{,}516\ \text{K}$$

$$T_{N\text{-}S} = \underbrace{4{,}010\ \text{K}}_{\text{Clay}} + \underbrace{2{,}516\ \text{K}}_{\text{Soil Cement}} = 6{,}526\ \text{K}$$

The resulting FS against sliding in the N-S direction is calculated as:

$$\textbf{FS}_{\textbf{Pad to Clayey Soil N-S w/Passive}} = \underbrace{6{,}526\ \text{K}}_{T_{N\text{-}S}} \div \underbrace{(\underbrace{29.3\ \text{K}}_{F_{AE\ N\text{-}S}} + \underbrace{643\ \text{K}}_{EQhp} + \underbrace{2{,}102\ \text{K}}_{Eqhc_{N\text{-}S}})}_{(2{,}774.3\ \text{K})} = \mathbf{2.35}$$

Ignoring the passive resistance provided by the soil cement adjacent to the pads, it is appropriate to use the peak shear strength of the underlying clayey soils, and the resulting FS against sliding in the N-S direction is calculated as:

$$\textbf{FS}_{\textbf{Pad to Clayey Soil N-S w/o Passive}} = \underbrace{4{,}221\ \text{K}}_{T_{N\text{-}S}} \div \underbrace{(\underbrace{29.3\ \text{K}}_{F_{AE\ N\text{-}S}} + \underbrace{643\ \text{K}}_{EQhp} + \underbrace{2{,}102\ \text{K}}_{Eqhc_{N\text{-}S}})}_{(2{,}774.3\ \text{K})} = \mathbf{1.52}$$

The resulting FS against sliding in the E-W direction will be even higher, since there is much greater length available to resist sliding in that direction. It is calculated as:

$$T_{SC\ \text{Adjacent to Pad}\ @\ E\&W} = 250\,\frac{\text{lbs}}{\text{in.}^2} \times \left(\frac{12\ \text{in.}}{\text{ft}}\right)^2 \times \frac{\text{K}}{1{,}000\ \text{lbs}} \times 2.33\ \text{ft} \times 67\ \text{ft} = 5{,}620\ \text{K}$$

$$T_{E\text{-}W} = \underbrace{4{,}010\ \text{K}}_{\text{Clay}} + \underbrace{5{,}620\ \text{K}}_{\text{Soil Cement}} = 9{,}630\ \text{K}$$

$$\textbf{FS}_{\textbf{Pad to Clayey Soil E-W}} = \underbrace{9{,}630\ \text{K}}_{T_{E\text{-}W}} \div \underbrace{(\underbrace{65.3\ \text{K}}_{F_{AE\ E\text{-}W}} + \underbrace{643\ \text{K}}_{EQhp} + \underbrace{2{,}212\ \text{K}}_{EQhc_{E\text{-}W}})}_{(2{,}920.3\ \text{K})} = \mathbf{3.30}$$

These values are greater than the minimum value (1.1) required for factor of safety against sliding, and they ignore the beneficial effects of the 1 to 2-ft thick layer of soil cement underneath the concrete pad. Therefore, adding the soil cement adjacent to the pads does enhance the sliding stability of each pad.

SLIDING RESISTANCE OF ENTIRE N-S COLUMN OF PADS

The resistance to sliding of the entire column (running N-S) of pads exceeds that of each individual pad because there is more area available to engage more shearing resistance from the underlying soils than just the area directly beneath the individual pads. The extra area is provided by the 5-ft long x 30-ft wide plug of soil cement that exists between each of the pads in the north-south direction. This analysis assumes that the soil cement east and west of the long column of pads provides no resistance to sliding, conservatively assuming that the soil cement somehow shears along a vertical plane at the eastern and western sides of the column of 10 pads running north-south.

Consider a column of 10 pads with 2'-4" of soil cement in between the pads and at least 1' of soil cement under the pads:

Cask Earthquake Loads$_{N-S}$ = 10 x 2,102 K = 21,020 K

Inertial forces due to Pads + Soil Cement:

Weight of Pads	=	10 x 904.5 K	=	9,045 K
Weight of Soil Cement	=	9 x 3.33 ft x 30 ft x 5 ft x 0.11 kips/ft^3	=	495 K
		+10 x 30 ft x 67 ft x 1 ft x 0.11 kips/ft^3	=	2,211 K
		Total Weight	=	11,751 K

Inertial forces due to Pads + Soil Cement = 0.711 x 11,751 K = 8,355 K

Dynamic active earth pressure acting in the N-S direction on pads + 2 ft (more conservative than using 1 ft, since it results in higher driving forces) of soil cement beneath the pads = 81.3 K

Total driving force in N-S direction = 21,020 K + 8,355 K + 81.3 K = 29,456 K

Ignoring Passive Resistance at End of N-S Column of Pads

This analysis conservatively ignores the passive resistance of the soil cement adjacent to the northern or southern end of the N-S column of pads. The resistance to sliding in the N-S direction is provided only by the shear strength of the soils underlying the soil cement layer beneath the pads (i.e., along Line IT in Figure 8). This case uses the soil cement beneath the pads as the engineered mechanism to bond the pads to the underlying clayey soils so that their peak shear strength can be engaged to resist sliding. As shown in Figure 7 on p. C2 of Attachment 2, the shear strength of the clayey soils under the pads is 2.1 ksf. The effective stresses under the soil cement between the pads is less than that directly under the pads; therefore, the shear strength available to resist sliding is lower. As shown in this figure, the shear strength available to resist sliding of the soil cement between the pads is 1.4 ksf. Using these strengths, the total resisting force is calculated as follows:

Soil cement

$T_{N\text{-}S}$ = 10 pads x 30 ft x 67 ft x 2.1 ksf + 9 zones between the pads x 30 ft x 5 ft x 1.4 ksf,

or $T_{N\text{-}S}$ = 42,210 K + 1,890 K = 44,100 K

Total driving force in N-S direction = 21,020 K + 8,355 + 81.3 K = 29,456 K, as calculated above.

The resulting FS against sliding in the N-S direction is calculated as:

$$FS_{\text{Pad to Clayey Soil N-S}} = \overset{T_{N\text{-}S}}{44{,}100\text{ K}} \div \overset{\text{Driving Force}_{N\text{-}S}}{29{,}456} = \underline{\mathbf{1.50}}$$

Ignoring Passive Resistance at End of E-W Row of Pads

The resulting FS against sliding in the E-W direction will be even higher, because the soil cement zone between the pads is much wider (35 ft vs 5 ft) and longer (67 ft vs 30 ft) between the pads in the E-W direction than those in the N-S direction. The cask driving forces in the E-W direction are slightly higher than in the N-S direction, 10 pads x 2,212 K = 22,120 K vs 10 pads x 2,102 K = 21,020 K, resulting in an increased driving force of 22,120 K – 21,020 K = 1,100 K. The resistance to sliding in the E-W direction is increased much more than this, however. The increased resistance to sliding E-W = 35 ft x 67 ft x 1.4 ksf = 3,283 K / area between pads in the E-W row, compared to 5 ft x 30 ft x 1.4 ksf = 210 K / area between pads in the N-S column. Thus, the factor of safety against sliding of a row of pads in the E-W is much greater than that shown above for sliding of a column of pads in the N-S direction.

Including Passive Resistance at End of N-S Column of Pads

In this analysis, the resistance to sliding in the N-S direction includes the full passive resistance at the far end of the column of pads, which acts on the 2'-4" height of soil cement along the 30-ft width of the pad in the E-W direction.

Assuming the soil cement adjacent to the pad is constructed such that its unconfined compressive strength is 250 psi, its full passive resistance acting on the 2'-4" thickness of soil cement adjacent to the pad will provide a force resisting sliding in the N-S direction of:

$$T_{\text{SC Adjacent to Pad @ N\&S}} = 250\,\frac{\text{lbs}}{\text{in.}^2} \times \left(\frac{12\text{ in.}}{\text{ft}}\right)^2 \times \frac{\text{K}}{1{,}000\text{ lbs}} \times 2.33\text{ ft} \times 30\text{ ft} = 2{,}516\text{ K}$$

The total resistance based on the peak shear strength of the underlying clayey soil is

Soil cement

$T_{N\text{-}S}$ = 10 pads x 30 ft x 67 ft x 2.1 ksf + 9 zones between the pads x 30 ft x 5 ft x 1.4 ksf, or

$T_{N\text{-}S}$ = 42,210 K + 1,890 K = 44,100 K

As discussed above, conservatively assume that the strength of the clayey soils beneath the soil cement layer underlying the pads is reduced to its residual strength (i.e., by 5%) to account for horizontal straining required to reach a strain that will result in the full passive resistance of the soil cement adjacent to the pad.

$T_{\text{N-S Residual Strength}} = 0.95 \times 44{,}100 \text{ K} = 41{,}895 \text{ K}$

$$T_{N\text{-}S} = \underset{\text{Clay}}{41{,}895 \text{ K}} + \underset{\text{Soil Cement}}{2{,}516 \text{ K}} = 44{,}411 \text{ K}$$

The resulting FS against sliding in the N-S direction is calculated as:

$$FS_{\text{Pad to Clayey Soil N-S}} = \underset{T_{N\text{-}S}}{44{,}411 \text{ K}} \div \underset{\text{Driving Force}_{N\text{-}S}}{29{,}456 \text{ K}} = \underline{\mathbf{1.51}}$$

Including Passive Resistance at End of E-W Row of Pads

The resulting FS against sliding in the E-W direction will be even higher, since there is much greater length available to resist sliding in that direction. The cask driving forces in the E-W direction are slightly higher than in the N-S direction, 10 pads x 2,212 K = 22,120 K vs 10 pads x 2,102 K = 21,020 K, resulting in an increased driving force of 22,120 K – 21,020 K = 1,100 K. The resistance to sliding in the E-W direction is increased more than this, including only the difference between the length vs the width of the pad. The soil cement adjacent to the pad provides (67 ft ÷ 30 ft) x 2,516 K, or 5,619 K of resistance based on the full passive pressure acting on the length of the pad, which is an increase of 5,619 K – 2,516 K = 3,103 K compared to the resistance provided by the soil cement to sliding in the N-S direction. This is greater than the increase in driving forces in the E-W direction; therefore, the factor of safety against sliding will be higher in the E-W direction. The soil cement zone between the pads also is much wider and longer between the pads in the E-W direction; therefore, there will be even more resistance to sliding E-W than N-S.

DETERMINE RESIDUAL STRENGTH REQUIRED ALONG BASE OF ENTIRE COLUMN OF PADS IN N-S DIRECTION, ASSUMING FULL PASSIVE RESISTANCE IS PROVIDED BY 250 PSI SOIL CEMENT ADJACENT TO LAST PAD IN COLUMN

To obtain FS = 1.1, the total resisting force, T, must =

1.1 x [Cask Earthquake Loads + (Wt of Pads + Wt of Soil Cement) x 0.711 + $F_{AE\ N\text{-}S}$]

= 1.1 x [21,020 K + (11,751 K x 0.711) + 81.3 K]

Therefore, $T_{FS=1.1}$ = 32,402 K

In this case, the resisting forces to sliding in the N-S direction include all of the passive resistance at the far end of the column of pads, which acts on the 2'-4" height of soil cement along the 30' width of the pad in the E-W direction + the 1' minimum thickness of soil cement under the pads.

Assuming the soil cement adjacent to the pad is constructed such that its unconfined compressive strength is 250 psi, the passive resistance acting on the 2'-4" thickness of soil cement adjacent to the pad + a minimum of 1' below the pad will provide a force resisting sliding in the N-S direction of:

$$T_{\text{SC Adjacent to Pad @ N\&S}} = 250\,\frac{\text{lbs}}{\text{in.}^2} \times \left(\frac{12\text{ in.}}{\text{ft}}\right)^2 \times \frac{\text{K}}{1{,}000\text{ lbs}} \times 3.33\text{ ft} \times 30\text{ ft} = 3{,}596\text{ K}$$

Base area, A, of a column of 10 pads is given by

$A = 10 \times 30\text{ ft} \times 67\text{ ft} + 9 \times 30\text{ ft} \times 5\text{ ft}$

$A = 20{,}100\text{ ft}^2 + 1{,}350\text{ ft}^2 = 21{,}450\text{ ft}^2$

Therefore the minimum shear strength required to provide the resisting force T is given by

$T_{N\text{-}S} = \tau \times \text{area (A)}$

$T_{N\text{-}S} = \tau_{Pad} \times 20{,}100\text{ ft}^2 + \tau_{Soil\ Cement} \times 1{,}350\text{ ft}^2 = 32{,}402\text{ K} - 3{,}596\text{ K} = 28{,}806\text{ K}$

$\tau_{Pad} = 2.1$ ksf & $\tau_{Soil\ Cement} = 1.4$ ksf; thus, $\tau_{Soil\ Cement} = (1.4 \div 2.1) \times \tau_{Pad} = 0.67 \times \tau_{Pad}$

$T_{N\text{-}S} = \tau_{Pad} \times 20{,}100\text{ ft}^2 + 0.67 \times \tau_{Pad} \times 1{,}350\text{ ft}^2 = \tau_{Pad} \times 21{,}000\text{ ft}^2$

$\tau_{Pad} \times 21{,}000\text{ ft}^2 = 28{,}806\text{ K}$

$\tau_{Pad} = 28{,}806\text{ K} \div 21{,}000\text{ ft}^2 = 1.37\text{ ksf}$

The peak shear strength of the clayey soils is 2.1 ksf. Therefore, the maximum reduction in peak strength permitted to obtain a factor of safety of 1.1 is calculated as:

$\Delta\tau = 1.37 \div 2.1 = 0.65$

In other words, the residual strength of the underlying clayey soils must drop below 65% of the peak shear strength before the factor of safety against sliding in the N-S direction of an entire column of pads will drop below 1.1.

Repeating this analysis, but ignoring the passive resistance of the soil cement adjacent to the pads at the northern or southern end of the column of pads,

$T_{N\text{-}S} = \tau_{Pad} \times 20{,}100\text{ ft}^2 + \tau_{Soil\ Cement} \times 1{,}350\text{ ft}^2 = 32{,}402\text{ K}$

$\tau_{Pad} = 2.1$ ksf & $\tau_{Soil\ Cement} = 1.4$ ksf; thus, $\tau_{Soil\ Cement} = (1.4 \div 2.1) \times \tau_{Pad} = 0.67 \times \tau_{Pad}$

$T_{N\text{-}S} = \tau_{Pad} \times 20{,}100\text{ ft}^2 + 0.67 \times \tau_{Pad} \times 1{,}350\text{ ft}^2 = \tau_{Pad} \times 21{,}000\text{ ft}^2$

$\tau_{Pad} \times 21{,}000\text{ ft}^2 = 32{,}402\text{ K}$

$\tau_{Pad} = 32{,}402\text{ K} \div 21{,}000\text{ ft}^2 = 1.54\text{ ksf}$

The peak shear strength of the underlying clayey soils is 2.1 ksf. Therefore, the maximum reduction in peak strength permitted to obtain a factor of safety of 1.1 is calculated as:

$\Delta\tau = 1.54 \div 2.1 = 0.73.$

In other words, even if the beneficial effects of the soil cement adjacent to the last pad in the N-S column of pads is ignored, the residual strength only needs to exceed 73% of the peak strength of the clayey soils to obtain a factor of safety against sliding in the N-S direction of an entire column of pads that is greater than 1.1.

As discussed above, the direct shear test results indicate that the greatest reduction between the peak shear strength and the residual shear strength is less than 5% for the specimens tested at effective stresses of 2 ksf, which are comparable to the final stresses under the fully loaded pads. The average reduction from peak stress is only ~20% for the specimens tested at effective vertical stresses of 1 ksf. Therefore, there is ample margin against sliding of an entire column of pads in the N-S direction.

SLIDING RESISTANCE OF LAST PAD IN COLUMN OF PADS ("EDGE EFFECTS")

Since the resistance to sliding of the cask storage pads is provided by the strength of the bond at the interface between the concrete pad and the underlying soil cement and by the bond between the soil cement under the pad and the in situ clayey soils, the sliding stability of the pads at the end of each column or row of pads are no different than that of the other pads. Therefore, the pads along the perimeter of the pad emplacement area also have an adequate factor of safety against sliding.

WIDTH OF SOIL CEMENT ADJACENT TO LAST PAD TO PROVIDE FULL PASSIVE RESISTANCE

As discussed above, the resisting force provided by the full passive resistance of the soil cement with an unconfined compressive strength of 250 psi acting on the last pad in the column of pads + a 1-ft thick layer of soil cement under the pad is:

$$T_{SC\ \text{Adjacent to Pad}\ @\ N\&S} = 250\,\frac{\text{lbs}}{\text{in.}^2} \times \left(\frac{12\ \text{in.}}{\text{ft}}\right)^2 \times \frac{\text{K}}{1{,}000\ \text{lbs}} \times 3.33\ \text{ft} \times 30\ \text{ft} = 3{,}596\ \text{K}$$

The base area required to provide this shear resistance = 30 ft x $L_{N\text{-}S}$ x 1.4 ksf, where 1.4 ksf is the shear strength of the underlying clayey soil for the effective vertical stress (~0.4 ksf) at the base of the soil cement layer beyond the end of the column of pads – See p C2.

$L_{N\text{-}S}$ = 3,596 K ÷ (30 ft x 1.4 ksf) = 85.62 ft.

Less than half of this amount is actually required due to 3D effects, similar to analysis of laterally loaded piles. Further, as shown above, the factor of safety against sliding of these pads exceeds the minimum allowable value without taking credit for the passive resistance provided by the soil cement adjacent to the pads. Therefore, this soil cement is not required for resisting sliding. However, the soil cement will be constructed adjacent to the pads, and it will extend further than this from the pads at the perimeter of the pad emplacement area. This soil cement will enhance the factor of safety against sliding, providing defense in depth against sliding of these pads due to the design ground motion.

SLIDING STABILITY OF THE PADS ASSUMING RESISTANCE IS BASED ON ONLY FRICTIONAL RESISTANCE ALONG BASE PLUS PASSIVE RESISTANCE

The design basis for the sliding stability of the cask storage pads relies on:

1. the assumption that sufficient "bonding" can be achieved at the interfaces between (a) the concrete comprising the pad and the soil cement beneath the pads, (b) soil cement lifts, and (c) soil cement and the underlying clayey soils such that the shear strength at these interfaces will be at least as high as the undrained strength measured in direct shear tests performed on samples of the underlying soils, and
2. the commitment to perform testing in the laboratory during the soil cement design phase to demonstrate that this "bonding" can be achieved, as well as during construction to demonstrate that this "bonding" has been achieved.

Laboratory testing to demonstrate the validity of this assumption are expected to be performed in the second half of 2001. Prior to completion of these tests, it is recognized that the resistance along the base of the pads + soil cement beneath the pads will be at least equal to the frictional resistance of the underlying soils, ignoring any contribution from the cohesive portion of the strength of these soils. Therefore, the purpose of this analysis is to demonstrate that even if the cohesion of the underlying soils is ignored along the interface between the soil cement and those soils, the resulting displacements of the pads would be minimal, and since there are no safety-related connections to these pads or casks, such displacements would have no safety consequence.

This hypothetical case assumes resistance to sliding is comprised of only frictional resistance along base of pads and soil cement + passive resistance, using obviously conservative values of the friction angle for the underlying soils. Although the resulting factor of safety is less than 1.1, the resulting maximum horizontal displacements, if they were to occur due to the earthquake, would be of no safety consequence to the pads or the casks.

Considering a single pad, assume that the shear strength available on the base of the pad to resist sliding is limited to that provided by friction alone. For this case, conservatively assume that friction is based on Table 1 of DM-7 (p. 7.2-63, NAVFAC, 1986), "Ultimate Friction Factors and Adhesion for Dissimilar Materials." This table indicates that an obviously conservative value of the friction angle for these clayey soils is 17 degrees. This is the lowest friction angle reported for the interface between mass concrete on any of the materials, and it applies for mass concrete on either "Fine sandy silt, nonplastic silt" or "Medium stiff and stiff clay and silty clay." Without including the cohesion, the resulting shear strength available to resist sliding of the pad is calculated as N tan ϕ. N = 1,146 K, as shown on p. 21:

SLIDING STABILITY OF THE PADS ASSUMING RESISTANCE IS BASED ON ONLY FRICTIONAL RESISTANCE ALONG BASE PLUS PASSIVE RESISTANCE

$$N = \underset{Wc}{2{,}852\text{ K}} + \underset{Wp}{904.5\text{ K}} + (\underset{EQvc}{-1{,}982\text{ K}}) + (\underset{EQvp}{-629\text{ K}}) = 1{,}146\text{ K}$$

$$T = \underset{N}{1{,}146\text{ K}} \times \tan \underset{\phi}{17^\circ} + \underset{c}{0\text{ ksf}} \times \underset{B}{30\text{ ft}} \times \underset{L}{67\text{ ft}} = 350.4\text{ K}$$

The driving force, V, is defined as: $V = F_{AE} + EQhp + EQhc$

The factor of safety against sliding is calculated as follows:

$$FS = \underset{T}{350.4\text{ K}} \div (\underset{F_{AE\ N\text{-}S}}{29.3\text{ K}} + \underset{EQhp}{643\text{ K}} + \underset{EQhc}{696\text{ K}}) = 0.26$$
$$(1{,}368.3\text{ K})$$

This analysis assumes that the maximum forces due to the earthquake act in both the north-south and vertical directions at the same time, which is not the case, and, thus, is overly conservative. Combining the effects of the earthquake components in accordance with ASCE 4-86, 100% of the vertical forces are assumed to act at the same time that 40% of the maximum forces act in the other two orthogonal directions. This results in the following, for a single pad:

Case IIIA: 40% N-S, -100% Vertical, 40% E-W (Earthquake Forces Act Upward)

$$N = \underset{Wc}{2{,}852\text{ K}} + \underset{Wp}{904.5\text{ K}} + (\underset{EQvc}{-1{,}982\text{ K}}) + (\underset{EQvp}{-629\text{ K}}) = 1{,}146\text{ K}$$

$$T = \underset{N}{1{,}146\text{ K}} \times \tan \underset{\phi}{17^\circ} + \underset{c}{0\text{ ksf}} \times \underset{B}{30\text{ ft}} \times \underset{L}{67\text{ ft}} = 350.4\text{ K}$$

The driving force, V, is defined as $V = F_{AE} + EQhp + Eqhc$, and using 40% in the north-south direction for this case (Case IIIA), the factor of safety against sliding is calculated as follows:

$$FS = \underset{T}{350.4\text{ K}} \div [\underset{40\%\text{ of }[F_{AE\ N\text{-}S}}{0.4 \times (29.3\text{ K}} + \underset{EQhp}{643\text{ K})} + \underset{Eqhc]}{696\text{ K}}] = 0.36$$
$$(964.9\text{ K})$$

In this case, note that $Eqhc_{N\text{-}S}$ = the minimum of $0.4 \times Eq_{hc\ max\ N\text{-}S}$ and $0.8 \times N_{Casks}$.

$Eq_{hc\ max\ N\text{-}S}$ = 2,101 K, as shown in the table on p. 20; thus, 40% of it = 841K.

$0.8 \times N_{Casks}$ = 696 K, as shown in the table on p. 20; therefore, $Eqhc_{N\text{-}S}$ equals 696 K. This is the maximum horizontal force that can be transmitted from the casks to the top of the pad due to friction.

To ensure the pad does not slide, the factor of safety should be greater than 1.1. Therefore, the resistance to sliding must be increased by 1.1 x 965 K – 350 K, or 615 K.

The soil cement adjacent to the pad is 2'-4" deep and 30' wide. The resisting force provided by the soil cement adjacent to the pad is calculated as the unconfined compressive strength, q_u, of the soil cement, multiplied by the area of the end of the pad, which equals 2.33' x 30'. Therefore,

$$q_u = \frac{615\,\text{K}}{2.33\,\text{ft} \times 30\,\text{ft}} = 8.8\,\frac{\text{K}}{\text{ft}^2} \times \frac{\text{ft}^2}{(12\,\text{in.})^2} \times \frac{1{,}000\,\text{lbs}}{\text{K}} = 61.1\,\text{psi}$$

As indicated above, in the section titled " Soil Cement Above the Base of the Pads":

> *"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)."*

Therefore, the resistance required to prevent an individual pad from sliding can readily be provided by passive resistance from the soil cement adjacent to the pad, ***if the soil cement can be demonstrated to stay in place*** to provide that resistance. Sliding of the soil cement is resisted by the shear strength along the base of the soil cement layer and the passive resistance of the in situ soils at the edge of the soil cement away from the pad, where the soil cement bears against the existing soils. The shear resistance available at the bottom of the soil cement is insignificant if we include only the frictional portion of the strength of the underlying clayey soils, ignoring the cohesive portion of the strength.

The following hypothetical analysis demonstrates that, even without imposing the horizontal loads from the pads, the frictional resistance along the base of the soil cement layer is not sufficient to preclude sliding of the soil cement block itself due to the earthquake loads.

The soil cement layer will be approximately 5-ft thick over most of the pad emplacement area; therefore, consider the sliding stability of a block of soil cement adjacent to the pads that is 5-ft thick. For Case IIIA, where 100% of the vertical earthquake forces act upward, tending to unload the soil cement, the normal stress at the base of the soil cement is very small. Preliminary results of the moisture-density tests that have been performed to-date on the soil-cement specimens indicate that 110 pcf is a reasonable unit weight to use for the soil cement adjacent to the pads. Without the earthquake loading, the normal stress at the base of the 5-ft deep soil cement layer is 5' x 0.110 kcf = 0.55 ksf. Subtracting the uplift forces, the normal stress is reduced to (1 – 0.695) x 0.55 ksf = 0.168 ksf. The shear resistance available due to friction at the base of the soil cement overlying the clayey soils is calculated as N tan ϕ, or 0.168 ksf x tan 17° = 0.051 ksf.

Assume there are no external forces acting on this block of soil cement, other than the horizontal and vertical dynamic forces due to the earthquake. In reality, there will be large horizontal forces imposed on the soil cement block from the pad, but these are ignored in this example to demonstrate the point that the soil cement cannot preclude sliding of the soil cement block itself during the earthquake ***based only on the frictional resistance*** along its base.

In this hypothetical case, the driving forces are due to the horizontal inertia of the soil-cement block. The maximum horizontal driving force is calculated as the mass of the block x the peak horizontal acceleration, 0.711g, which equals 0.711g x 5' x 0.110 kcf/g x the width and length of the block of soil cement. The resulting horizontal shear stress at the base of the block = 0.39 ksf. In this case (Case IIIA) only 40% of this value is considered to act horizontally at the same time as the full uplift force, resulting in a maximum horizontal shear stress due to the driving force of 0.4 x 0.39 ksf = 0.156 ksf.

The factor of safety against sliding is calculated as the resisting forces ÷ the driving forces, or, since the area of the base of the block is the same for resisting and driving forces,

$$FS_{\text{Soil-cement Block}}\text{Case IIIA} = \frac{\text{Shear Strength Due to Friction}}{\text{Shear Stress Due to Horiz Inertia}} = \frac{0.051\text{ ksf}}{0.156\text{ ksf}} = 0.33$$

Similar results apply for Loading Case IIIC, where 100% of the earthquake forces are assumed to act in the north-south direction when 40% act in the other two orthogonal directions; e.g.,

$$FS_{\text{Soil-cement Block}}\text{Case IIIC} = \frac{(1-0.4\times 0.695)\times 5\text{ ft}\times 0.11\text{ kcf}\times\tan 17^\circ}{100\%\times 0.711\times 5\text{ ft}\times 0.11\text{ kcf}} = \frac{0.121\text{ ksf}}{0.391\text{ ksf}} = 0.31$$

Thus, the soil cement cannot provide adequate resistance ***based solely on the friction acting along its base*** to preclude sliding of the pad. As a matter of fact, the soil cement cannot even resist sliding of itself during the earthquake ***if only the frictional portion of the strength is assumed to be available*** along its base. Even using an unreasonably high value of the friction angle in this calculation, say 40°, the factor of safety against sliding of the soil-cement block is still not adequate to preclude sliding of the block due to only the inertia forces of the block itself; e.g.,

$$FS_{\text{Soil-cement Block}}\frac{\text{Case IIIA}}{\text{w}/\phi = 40^\circ} = \frac{(1-0.695)\times 5\text{ ft}\times 0.11\text{ kcf}\times\tan 40^\circ}{40\%\times 0.711\times 5\text{ ft}\times 0.11\text{ kcf}} = \frac{0.141\text{ ksf}}{0.156\text{ ksf}} = 0.90$$

Therefore, the effects of the frictional resistance acting on the base of the soil-cement block are ignored in the following hypothetical analysis of the factor of safety against sliding of a single pad.

The passive resistance at the edge of the soil cement, where it bears against the existing soil, is included, however. The soil cement layer is 5-ft deep at the edge away from the end of the pad. The passive resistance of the soils at this edge is calculated as follows. In this case, assume the strength of the soil is based on the triaxial test results presented in

Attachment 8 of Appendix 2A of the SAR. A copy of the summary plot of these test results is included in Attachment E of this calculation, and it indicates c = 1.4 ksf and ϕ = 21.3°.

Equation 23.7 of Lambe and Whitman (1969) indicates that the passive resisting force, P_p, is calculated as:

$$P_p = \frac{1}{2}\gamma_b \times H^2 \times N_\phi + 2c \times H \times \sqrt{N_\phi},$$

where $N_\phi = \dfrac{1+\sin\phi}{1-\sin\phi} = \dfrac{1+\sin 21.3^\circ}{1-\sin 21.3^\circ} = 2.14$ Eq 23.2 Lambe & Whitman (1969)

and H = 5 ft

$$\therefore\ P_p = \frac{1}{2}0.080\ \text{kcf} \times (5\ \text{ft})^2 \times 2.14 + 2 \times 1.4\ \text{ksf} \times 5\ \text{ft} \times \sqrt{2.14} = 20.91\ \text{K/LF}$$

For the 30 ft width of the pad, full passive resistance of the in situ soils = 30 ft × 20.91 K/LF = 627.3 K.

Thus, for a single pad, the factor of safety against sliding based on friction acting on the base of the pad and the full passive resistance of the existing soils is calculated as follows:

$$FS = (\underbrace{\overset{T}{350.4\ K} + \overset{P_p}{627.3\ K}}_{(977.7\ K)}) \div \underbrace{[0.4 \times (\overset{40\%\ \text{of}\ [F_{AE\ N\text{-}S}}{29.3\ K} + \overset{EQhp}{643\ K}) + \overset{Eqhc]}{696\ K}]}_{(964.9\ K)} = 1.01$$

This is less than 1.1, the minimum acceptable factor of safety to preclude sliding of the pads. Therefore, a single pad is not stable for the loads associated with Case IIIA, ***assuming that resistance to sliding is provided only by friction acting on the base*** of the pads and the full passive resistance of the site soils.

Check Sliding of an Entire Row of Pads in the North-South Direction for the Hypothetical Case Where Resistance Along the Base Is Due Solely to Frictional Resistance

Note, the length of the pads, 67 ft in the north-south direction, is more than twice the width, 30 ft in the east-west direction; therefore, the resistance to sliding is greater in the east-west direction when passive resistance is considered. Thus, these analyses are performed for sliding in the north-south direction.

Considering one north-south row of pads, assume that the shear strength available on the base of the pads to resist sliding is limited to that provided by friction alone. As discussed above, the resulting shear strength available to resist sliding of each pad is calculated as N tan ϕ. N = 1,146 K, calculated as follows:

$$N = \underset{Wc}{2{,}852\ K} + \underset{Wp}{904.5\ K} + \underset{EQvc}{(-1{,}982\ K)} + \underset{EQvp}{(-629\ K)} = 1{,}146\ K$$

$$T = \underset{N}{1{,}146\ K} \times \tan \underset{\phi}{17°} + \underset{c}{0\ ksf} \times \underset{B}{30\ ft} \times \underset{L}{67\ ft} = 350.4\ K$$

Therefore, the total resistance due to friction acting on the base of 20 pads in the row is 20 x 350.4 K = 7,008 K. Note, ϕ is assumed to be 17°, an obviously conservative value based on Table 1 on p. 7.2-63 of DM-7 (NAVFAC, 1986), as discussed above.

The passive resistance of the soils at the edge of the 5-ft deep layer of soil cement away from the end of the pad is available to resist sliding of the entire row of pads. It is calculated, as shown above, and it equals 20.91 K/LF of width of the 5-ft deep soil cement layer surrounding the pad emplacement area. For a strip 30-ft wide at either the northern or southern end of the row of pads, this provides an additional resistance to sliding of 627.3 K. It is reasonable to expect that, due to 3D effects, the soil cement will distribute the horizontal loads from the row of pads over more than just the 30-ft width of the pad. This passive resistance would be limited, however, to the width of the pad, 30 ft, + the width of the aisle between the rows of pads north-south, 35 ft. Thus, the maximum credible contribution of the passive resistance of the existing soils at the edge of the soil-cement layer north or south of the entire row of pads is 20.91 K/LF x (30' + 35'), which equals 1,359 K.

As shown above, the shear strength available due to friction along the base of the soil cement between the pads and at the end of the row of pads (0.051 ksf) is not sufficient to resist the inertial forces of the soil cement (0.156 ksf) and, thus, is ignored in this analysis. It is recognized that the forces due to the difference between this frictional shear strength along the base of the soil cement and the horizontal shear stresses due to the inertial forces should be accounted for in the analysis of sliding, but it is ignored in this example to demonstrate the point that the soil cement cannot preclude sliding of the entire row of pads if the resistance along the base of the soil cement ***is limited to only the frictional component.***

Therefore, the total resisting force available for the entire row of 20 pads due to only friction along the base of the row + passive resistance of the existing soils at the edge of the soil cement = 7,008 K + 627.3 K = 7,635.3 K. If 3D effects are included to distribute the horizontal loads beyond the 30-ft width of the pad, the maximum credible resisting force is 7,008 K + 1,359 K = 8,367 K.

The driving force, V, is defined as V = F_{AE} + EQhp + EQhc. For the entire row of 20 pads, the maximum horizontal driving force is calculated as:

$$V = \underbrace{29.3\text{ K}}_{F_{AE\ N\text{-}S}} + 20\text{ pads} \times [\underbrace{643\text{ K}}_{EQhp} + \underbrace{696\text{ K}}_{EQhc}] = 26{,}809\text{ K}.$$

For Case IIIA, 40% of the horizontal driving force is assumed to act in the north-south direction at the same time as 100% of the uplift force due to the earthquake. Thus, the driving force for Case $IIIA_{N\text{-}S}$ is:

$$V_{IIIA\ N\text{-}S} = 0.4 \times (\underbrace{29.3\text{ K}}_{F_{AE\ N\text{-}S}} + 20\text{ pads} \times \underbrace{643\text{ K}}_{EQhp}) + 20\text{ pads} \times \underbrace{696\text{ K}}_{EQhc} = 19{,}076\text{ K}.$$

And the factor of safety against sliding of the entire row for Case IIIA is calculated as follows:

$$FS = \underbrace{7{,}635.3\text{ K}}_{T} \div \underbrace{19{,}076\text{ K}}_{40\%\text{ of }F_{AE\ N\text{-}S}+EQhp+EQhc} = 0.40$$

or, for the maximum credible passive resistance, relying on distribution of the horizontal loads through the soil cement in to the soils due to 3D effects, the factor of safety against sliding is calculated as follows:

$$FS = \underbrace{8{,}367\text{ K}}_{T} \div \underbrace{19{,}076\text{ K}}_{40\%\text{ of }F_{AE\ N\text{-}S}+EQhp+EQhc} = 0.44$$

These values are less than 1.1; therefore, assuming the resistance to sliding is provided only by frictional resistance along the base of the row of pads and soil cement + passive resistance available at the edge of the soil cement, the pads might slide due to the design earthquake. As indicated in Section 4.4.2 of the Storage Facility Design Criteria (Stone & Webster, 2000),

> *"Where the factor of safety against sliding is less than 1 due to the design basis ground motion, the displacements the structure may experience are calculated using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes. The magnitude of these displacements are evaluated to assess the impact on the performance of the structure."*

The following analyses estimate the horizontal displacement of the pads, assuming they are supported directly on frictional soils with ϕ = 17°. These analyses are based on the method proposed by Newmark (1965) to estimate the displacement of the pads, which is described in the section titled " Evaluation of Sliding on Deep Slip Surface Beneath Pads."

Load Case IIIA: 40% N-S direction, -100% Vertical direction, 40% E-W direction.

20 Pads in N-S Row

Static Vertical Force, F_v = W = Weight of casks, pads, and soil cement in the row

$$\text{Pads + Casks} = 20 \times [904.5\text{ K} + 2{,}852\text{ K}] = \mathbf{75{,}130\text{ K}}$$

Soil cement adjacent to pads is 30 ft wide and 3 ft deep =

$$30\text{ ft width} \times 3\text{ ft deep} \times \left[9\,\frac{\text{gaps}}{\text{area}} \times 5\text{ ft}\,\frac{\text{length}}{\text{gap}} \times 2\text{ areas} + 90\text{ ft between areas}\right] \times 0.110\text{ kcf} = \mathbf{1{,}782\text{ K}}$$

Soil cement 2 ft deep beneath the pads, which are 30 ft wide =

$$30\text{ ft} \times 2\text{ ft} \times \left[20\text{ pads} \times 67\,\frac{\text{ft}}{\text{pad}} + 9\,\frac{\text{gaps}}{\text{area}} \times 5\text{ ft}\,\frac{\text{length}}{\text{gap}} \times 2\text{ areas} + 90\text{ ft between areas}\right]$$

$$\times 0.100\text{ kcf} = \mathbf{9{,}120\text{ K}}$$

$$\Rightarrow \text{Fv} = 75{,}130\text{ K} + 1{,}728\text{ K} + 9{,}120\text{ K} = \mathbf{86{,}032\text{ K}}$$

Earthquake Vertical Force, $F_{v\,Eqk} = a_V \times W/g = 0.695g \times 86{,}032\text{ K}/g = 59{,}792\text{ K}$

$$\phi = 17°$$

For Case IIIA, 100% of vertical earthquake force is applied upward and, thus, must be subtracted to obtain the normal force; thus, Newmark's maximum resistance coefficient is

$$\mathbf{N} = [(\underset{F_v}{86{,}032} - \underset{F_{v\,Eqk}}{59{,}792}) \tan \underset{\phi}{17°} + \underset{Pp}{627.3\text{ K}}] / \underset{W}{86{,}032} = 0.101$$

Acceleration in N-S direction, A = 0.284g

Velocity in N-S direction, V = 13.7 in./sec

$$\Rightarrow \quad \mathbf{N} / A = 0.101 / 0.284 = 0.354$$

The maximum displacement of the pad relative to the ground, u_m, calculated based on Newmark (1965) is

$$u_m = [V^2 (1 - \mathbf{N}/A)] / (2g\mathbf{N})$$

where g is in units of inches/sec^2.

$$\Rightarrow \quad u_m = \left(\frac{(13.7\text{ in./sec})^2 \cdot (1 - 0.354)}{2 \cdot 386.4\text{ in./sec}^2 \cdot 0.101}\right) = 1.55''$$

The above expression for the relative displacement is an upper bound for all the data points for **N** /A less than 0.15 and greater than 0.5, as shown in Figure 5. For N/A values between 0.15 and 0.5 the data in Figure 5 is bounded by the expression

$$u_m = [V^2] / (2g\mathbf{N})$$

$$\Rightarrow \quad u_m = \left(\frac{(13.7 \text{ in./sec})^2}{2 \cdot 386.4 \text{ in./sec}^2 \cdot 0.101} \right) = 2.40''$$

In this case, N /A is = 0.354. As shown in Figure 5, at this value of N/A, the data points for actual earthquake records are between the two curves, and the maximum displacement is closer to the average of these two curves. Therefore, use the average of the maximum displacements calculated above, or the maximum displacement is 1.98 inches.

Load Case IIIB: 40% N-S direction, -40% Vertical direction, 100% E-W direction.

Since the pads are longer in the north-south direction than in the east-west direction, the passive resistance available to resist sliding in the east-west direction will be greater than that resisting sliding in the north-south direction. Thus, sliding in the north-south direction is more critical than sliding east-west. See Load Case IIIC for estimate of displacement in the north-south direction.

Load Case IIIC: 100% N-S direction, -40% Vertical direction, 40% E-W direction.

Static Vertical Force, F_v = W = 86,032 K

Earthquake Vertical Force, $F_{v(Eqk)}$ = 59,792 K x 0.40 = 23,917 K

$$\phi = 17°$$

$$\mathbf{N} = [(\underset{F_v}{86{,}032} - \underset{F_{v\,Eqk}}{23{,}917}) \tan \underset{\phi}{17°} + \underset{Pp}{627.3\text{ K}}] / \underset{W}{86{,}032} = 0.228$$

Acceleration in N-S direction, A = 0.711g

Velocity in N-S direction, V = 34.1 in./sec

$$\Rightarrow \quad \mathbf{N} / A = 0.228 / 0.711 = 0.321$$

The maximum displacement of the pad relative to the ground, u_m, calculated based on Newmark (1965) is

$$u_m = [V^2 (1 - \mathbf{N}/A)] / (2g\,\mathbf{N})$$

$$\Rightarrow \quad u_m = \left(\frac{(34.1 \text{ in./sec})^2 \cdot (1 - 0.321)}{2 \cdot 386.4 \text{ in./sec}^2 \cdot 0.228} \right) = 4.48''$$

The above expression for the relative displacement is an upper bound for all the data points for **N** /A less than 0.15 and greater than 0.5, as shown in Figure 5. For N/A values between 0.15 and 0.5 the data in Figure 5 is bounded by the expression

$$u_m = [V^2] / (2g\mathbf{N})$$

$$\Rightarrow \quad u_m = \left(\frac{(34.1\,\text{in./sec})^2}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.228} \right) = 6.60"$$

In this case, N /A is = 0.321. As shown in Figure 5, at this value of N/A, the data points for actual earthquake records are between the two curves; the data points for actual earthquake records are between the two curves, and the maximum displacement is closer to the upper curve. Therefore, the maximum displacement is ~6 inches.

SUMMARY OF HORIZONTAL DISPLACEMENTS CALCULATED BASED ON NEWMARK'S METHOD FOR ASSUMPTION THAT CASK STORAGE PADS ARE FOUNDED DIRECTLY ON COHESIONLESS SOILS WITH ϕ = 17° AND PASSIVE PRESSURE DUE TO SITE SOILS ACTS ON 5-FT THICK LAYER OF SOIL CEMENT AT END OF ROW OF 20 PADS

LOAD COMBINATION				DISPLACEMENT
Case IIIA	40% N-S	-100% Vert	40% E-W	~2 inches
Case IIIB	40% N-S	-40% Vert	100% E-W	< Case IIIC
Case IIIC	100% N-S	-40% Vert	40% E-W	~6 inches

Assuming the cask storage pads are founded directly on a layer of cohesionless soils with ϕ = 17°, the estimated relative displacement of the pads due to the design basis ground motion based on Newmark's method of estimating displacements of embankments and dams due to earthquakes ranges from ~2 inches to ~6 inches. There are several conservative assumptions that were made in determining these values for this hypothetical case, and, therefore, the estimated displacements represent upper-bound values. Even if the maximum horizontal displacement were to occur from an earthquake, there would be no safety consequence to the pads or the casks, since the pads and casks do not rely on any external "Important to Safety" connections.

CALCULATION IDENTIFICATION NUMBER				PAGE 46
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

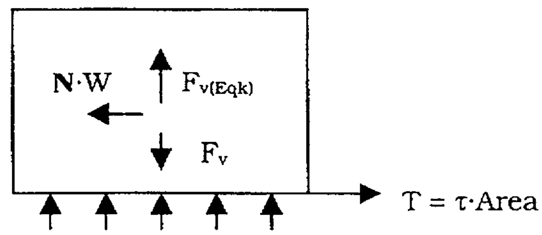
EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

Adequate factors of safety against sliding due to maximum forces from the design basis ground motion have been obtained for the storage pads founded directly on the silty clay/clayey silt layer, conservatively ignoring the presence of the soil cement that will surround the pads. The shearing resistance is provided by the undrained shear strength of the silty clay/clayey silt layer, which is not affected by upward earthquake loads. As shown in SAR Figures 2.6-5, Pad Emplacement Area – Foundation Profiles, a layer, composed in part of sandy silt, underlies the clayey layer at a depth of about 10 ft below the cask storage pads. Sandy silts oftentimes are cohesionless; therefore, to be conservative, this portion of the sliding stability analysis assumes that the soils in this layer are cohesionless, ignoring the effects of cementation that were observed on many of the split-spoon and thin-walled tube samples obtained in the drilling programs.

The shearing resistance of cohesionless soils is directly related to the normal stress. Earthquake motions resulting in upward forces reduce the normal stress and, consequently, the shearing resistance, for purely cohesionless (frictional) soils. Factors of safety against sliding in such soils are low if the maximum components of the design basis ground motion are combined. The effects of such motions are evaluated by estimating the displacements the structure will undergo when the factor of safety against sliding is less than 1 to demonstrate that the displacements are sufficiently small that, should they occur, they will not adversely impact the performance of the pads.

The method proposed by Newmark (1965) is used to estimate the displacement of the pads, assuming they are founded directly on a layer of cohesionless soils. This simplification produces an upper-bound estimate of the displacement that the pads might see if a cohesionless layer was continuous beneath the pads. For motion to occur on a slip surface along the top of a cohesionless layer at a depth of 10 ft below the pads, the slip surface would have to pass through the overlying clayey layer, which, as shown above, is strong enough to resist sliding due to the earthquake forces. In this analysis, a friction angle of 30° is used to define the strength of the soils to conservatively model a loose cohesionless layer. The soils in the layer in question have a much higher friction angle, generally greater than 35°, as indicated in the plots of "Phi" interpreted from the cone penetration testing, which are presented in Appendix D of ConeTec (1999).

ESTIMATION OF HORIZONTAL DISPLACEMENT USING NEWMARK'S METHOD



Newmark (1965) defines "**N**·W" as the steady force applied at the center of gravity of the sliding mass in the direction which the force can have its lowest value to just overcome the stabilizing forces and keep the mass moving. Note, Newmark defines "**N**" as the "Maximum Resistance Coefficient," and it is an acceleration coefficient in this case, not the normal force.

For a block sliding on a horizontal surface, $\mathbf{N}\cdot W = T$,

where T is the shearing resistance of the block on the sliding surface.

Shearing resistance, $T = \tau\cdot\text{Area}$

where $\tau = \sigma_n \tan\phi$

$\sigma_n =$ Normal Stress

$\phi =$ Friction angle of cohesionless layer

$\sigma_n =$ Net Vertical Force/Area

$= (F_v - F_{v\,Eqk})/\text{Area}$

$T = (F_v - F_{v\,Eqk}) \tan\phi$

$\mathbf{N}\,W = T$

$\Rightarrow \quad \mathbf{N} = [(F_v - F_{v\,Eqk}) \tan\phi] / W$

The maximum relative displacement of the pad relative to the ground, u_m, is calculated as

$$u_m = [V^2 (1 - \mathbf{N}/A)] / (2g\mathbf{N})$$

The above expression for the relative displacement is an upper bound for all of the data points for **N**/A less than 0.15 and greater than 0.5, as shown in Figure 5 , which is a copy of Figure 41 of Newmark (1965). Within the range of 0.5 to 0.15, the following expression gives an upper bound of the maximum relative displacement for all data.

$$u_m = V^2 /(2g\mathbf{N})$$

MAXIMUM GROUND MOTIONS

The maximum ground accelerations used to estimate displacements of the cask storage pads were those due to the PSHA 2,000-yr return period earthquake; i.e., $a_H = 0.711g$ and $a_V = 0.695g$. The maximum horizontal ground velocities required as input in Newmark's method of analysis of displacements due to earthquakes were estimated for the cask storage pads assuming that the ratio of the maximum ground velocity to the maximum ground acceleration equaled 48 (i.e., 48 in./sec per g). Thus, the estimated maximum velocities applicable for the Newmark's analysis of displacements of the cask storage pads = 0.711 x 48 = 34.1 in./sec. Since the peak ground accelerations are the same in both horizontal directions, the velocities are the same as well.

Evaluation of Sliding on Deep Slip Surface Beneath Pads

Load Cases

The resistance to sliding on cohesionless materials is lowest when the dynamic forces due to the design basis ground motion act in the upward direction, which reduces the normal forces and, hence, the shearing resistance, at the base of the foundations. Thus, the following analyses are performed for Load Cases IIIA, IIIB, and IIIC, in which the pads are unloaded due to uplift from the earthquake forces.

Case IIIA 40% N-S direction,-100% Vertical direction, 40% E-W direction.

Case IIIB 40% N-S direction, -40% Vertical direction, 100% E-W direction.

Case IIIC 100% N-S direction, -40% Vertical direction, 40% E-W direction.

Ground Motions for Analysis

Load Case	North-South		Vertical	East-West	
	Accel g	Velocity in./sec	Accel g	Accel g	Velocity in./sec
IIIA	0.284g	13.7	0.695g	0.284g	13.7
IIIB	0.284g	13.7	0.278g	0.711g	34.1
IIIC	0.711g	34.1	0.278g	0.284g	13.7

Load Case IIIA: 40% N-S direction, -100% Vertical direction, 40% E-W direction.

Static Vertical Force, F_v = W = Weight of casks and pad = 2,852 K + 904.5 K = 3,757 K

Earthquake Vertical Force, $F_{v\,Eqk} = a_v \times W/g = 0.695g \times 3{,}757\ K/g = 2{,}611\ K$

$$\phi = 30°$$

For Case IIIA, 100% of vertical earthquake force is applied upward and, thus, must be subtracted to obtain the normal force; thus, Newmark's maximum resistance coefficient is

$$\mathbf{N} = [(\underset{F_v}{3{,}757} - \underset{F_{v\,Eqk}}{2{,}611}) \tan \underset{\phi}{30°}] / \underset{W}{3{,}757} = 0.176$$

$$\text{Resultant acceleration in horizontal direction, } A = \sqrt{(\underset{40\%\ N\text{-}S}{0.284^2} + \underset{40\%\ E\text{-}W}{0.284^2})} = 0.402g$$

$$\text{Resultant velocity in horizontal direction, } V = \sqrt{(\underset{40\%\ N\text{-}S}{13.7^2} + \underset{40\%\ E\text{-}W}{13.7^2})} = 19.4\ \text{in./sec}$$

$$\Rightarrow \quad \mathbf{N} / A = 0.176 / 0.402 = 0.438$$

The maximum displacement of the pad relative to the ground, u_m, calculated based on Newmark (1965) is

$$u_m = [V^2 (1 - \mathbf{N}/A)] / (2g\mathbf{N})$$

where g is in units of inches/sec².

$$\Rightarrow \quad u_m = \left(\frac{(19.4\ \text{in./sec})^2 \cdot (1 - 0.438)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.176} \right) = 1.56''$$

The above expression for the relative displacement is an upper bound for all the data points for **N** /A less than 0.15 and greater than 0.5, as shown in Figure 5. For N/A values between 0.15 and 0.5 the data in Figure 5 is bounded by the expression

$$u_m = [V^2] / (2g\mathbf{N})$$

$$\Rightarrow \quad u_m = \left(\frac{(19.4\ \text{in./sec})^2}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.176} \right) = 2.77''$$

In this case, **N** /A is = 0.438; therefore, use the average of the maximum displacements; i.e., 0.5 (1.56 + 2.77) = 2.2" . Thus the maximum displacement is ~2.2 inches.

Load Case IIIB: 40% N-S direction, -40% Vertical direction, 100% E-W direction.

Static Vertical Force, $F_v = W = 3{,}757$ K

Earthquake Vertical Force, $F_{v(Eqk)} = 2{,}611$ K x 0.40 = 1,044 K

$$\phi = 30°$$

$$\mathbf{N} = [(\underset{F_v}{3{,}757} - \underset{F_{v\,Eqk}}{1{,}044}) \tan \underset{\phi}{30°}] / \underset{W}{3{,}757} = 0.417$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\underset{40\%\ N\text{-}S}{0.284^2} + \underset{100\%\ E\text{-}W}{0.711^2})}\, g = 0.766g$

Resultant velocity in horizontal direction, $V = \sqrt{(\underset{40\%\ N\text{-}S}{13.7^2} + \underset{100\%\ E\text{-}W}{34.1^2})} =$ 36.7 in./sec

$$\Rightarrow \quad \mathbf{N} / A = 0.417 / 0.766 = 0.544$$

The maximum displacement of the pad relative to the ground, u_m, calculated based on Newmark (1965) is

$$u_m = [V^2 (1 - \mathbf{N}/A)] / (2g\,\mathbf{N})$$

$$\Rightarrow \quad u_m = \left(\frac{(36.7\,\text{in./sec})^2 \cdot (1 - 0.544)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.417} \right) = 1.91''$$

The above expression for the relative displacement is an upper bound for all the data points for **N** /A less than 0.15 and greater than 0.5, as shown in Figure 5. In this case, **N** /A is > 0.5; therefore, this equation is applicable for calculating the maximum relative displacement. Thus the maximum displacement is ~1.9 inches.

Load Case IIIC: 100% N-S direction, -40% Vertical direction, 40% E-W direction.

Since the horizontal accelerations and velocities are the same in the orthogonal directions, the result for Case IIIC is the same as those for Case IIIB.

SUMMARY OF HORIZONTAL DISPLACEMENTS CALCULATED BASED ON NEWMARK'S METHOD FOR ASSUMPTION THAT CASK STORAGE PADS ARE FOUNDED DIRECTLY ON COHESIONLESS SOILS WITH ϕ = 30° AND NO SOIL CEMENT

LOAD COMBINATION				DISPLACEMENT
Case IIIA	40% N-S	-100% Vert	40% E-W	2.2 inches
Case IIIB	40% N-S	-40% Vert	100% E-W	1.9 inches
Case IIIC	100% N-S	-40% Vert	40% E-W	1.9 inches

Evaluation of Sliding on Deep Slip Surface Beneath Pads

Assuming the cask storage pads are founded directly on a layer of cohesionless soils with $\phi = 30°$, the estimated relative displacement of the pads due to the design basis ground motion based on Newmark's method of estimating displacements of embankments and dams due to earthquakes ranges from ~1.9 inches to 2.2 inches. Because there are no connections between the pads or between the pads and other structures, displacements of this magnitude, were they to occur, would not adversely impact the performance of the cask storage pads. There are several conservative assumptions that were made in determining these values and, therefore, the estimated displacements represent upper-bound values.

The soils in the layer that are assumed to be cohesionless, the one ~10 ft below the pads that is labeled "Clayey Silt/Silt & Some Sandy Silt" in the foundation profiles in the pad emplacement area (SAR Figures 2.6-5, Sheets 1 through 14), are clayey silts and silts, with some sandy silt. To be conservative in this analysis, these soils are assumed to have a friction angle of 30°. However, the results of the cone penetration testing (ConeTec, 1999) indicate that these soils have ϕ values that generally exceed 35 to 40°, as shown in Appendices D & F of ConeTec (1999). These high friction angles likely are the manifestation of cementation that was observed in many of the specimens obtained in split-barrel sampling and in the undisturbed tubes that were obtained for testing in the laboratory. Possible cementation of these soils is also ignored in this analysis, adding to the conservatism.

In addition, this analysis postulates that cohesionless soils exist directly at the base of the pads. In reality, the surface of these soils is 10 ft or more below the pads, and it is not likely to be continuous, as the soils in this layer are intermixed. For the pads to slide, a surface of sliding must be established between the horizontal surface of the "cohesionless" layer at a depth of at least 10 ft below the pads, through the overlying clayey layer, and daylighting at grade. As shown in the analysis preceding this section, the overlying clayey layer is strong enough to resist sliding due to the earthquake forces. The contribution of the shear strength of the soils along this failure plane rising from the horizontal surface of the "cohesionless" layer at a depth of at least 10 ft to the resistance to sliding is ignored in the simplified model used to estimate the relative displacement, further adding to the conservatism.

These analyses also conservatively ignore the presence of the soil cement under and adjacent to the cask storage pads. As shown above, this soil cement can easily be designed to provide all of the sliding resistance necessary to provide an adequate factor of safety, considering only the passive resistance acting on the sides of the pads, without relying on friction or cohesion along the base of the pads. Adding friction and cohesion along the base of the pads will increase the factor of safety against sliding.

CALCULATION IDENTIFICATION NUMBER				PAGE 111
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 9		

FIGURE 3

DETAIL OF SOIL CEMENT UNDER & ADJACENT TO CASK STORAGE PADS

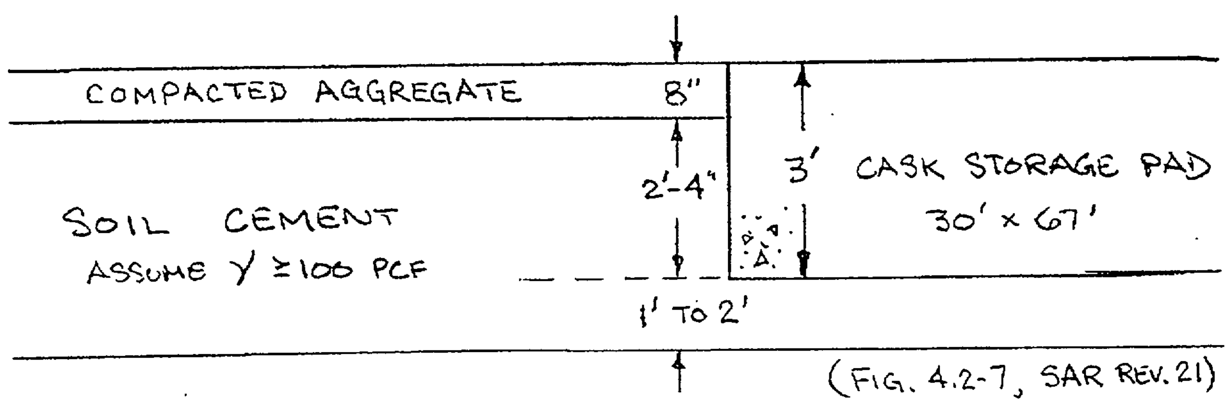
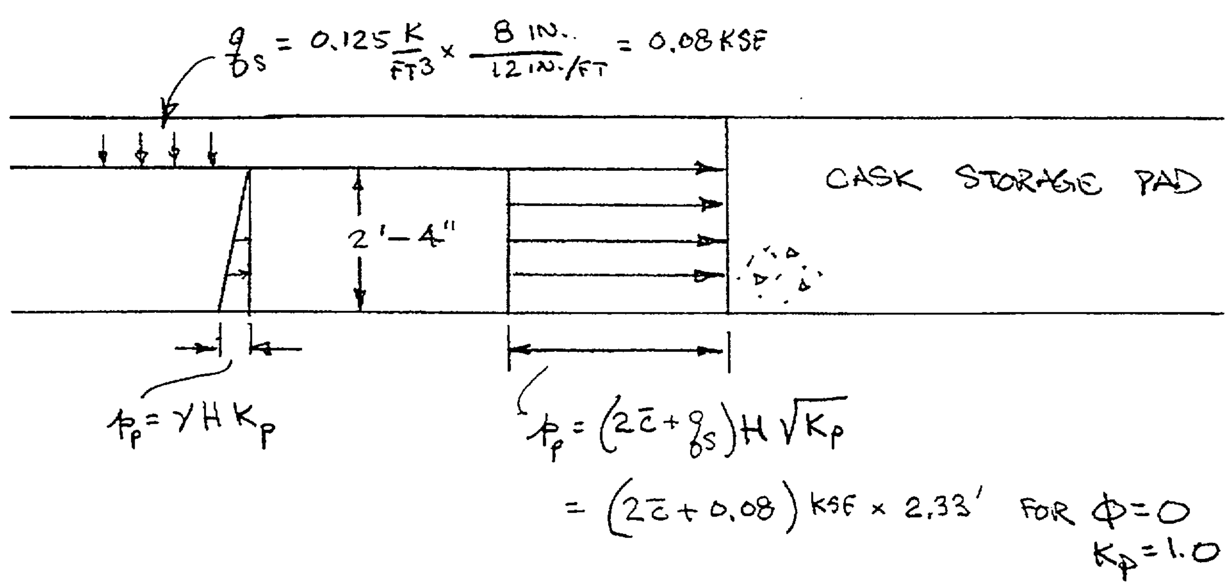


FIGURE 4

PASSIVE PRESSURE ACTING ON CASK STORAGE PADS



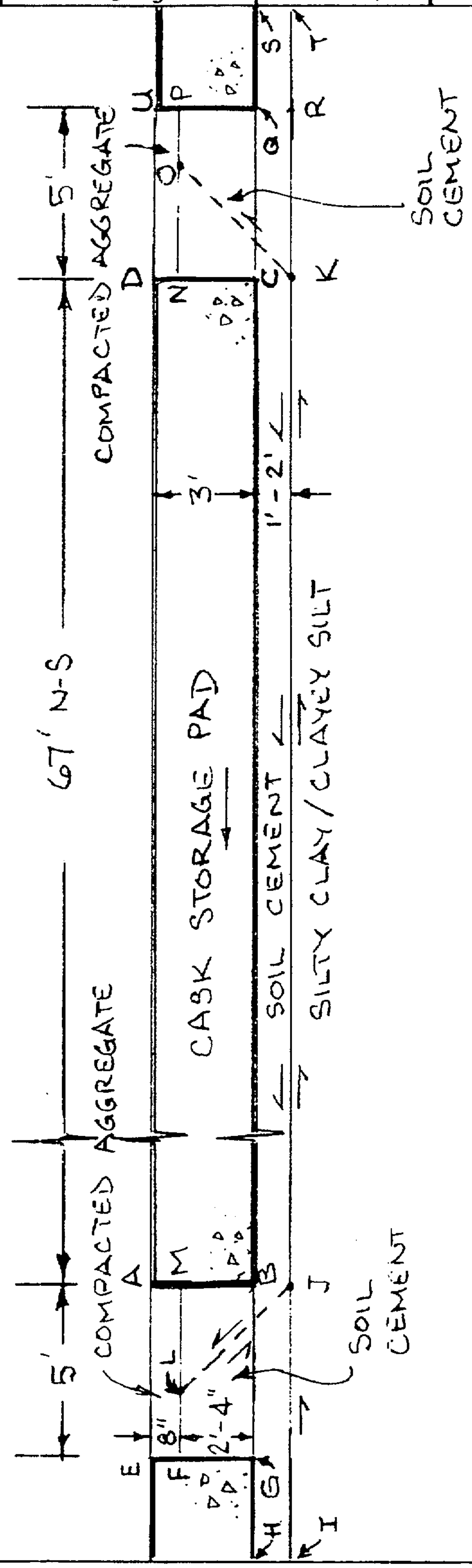
STONE & WEBSTER, INC.
CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 115
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04-9		

FIGURE 8

ELEVATION VIEW OF COLUMN OF CASK STORAGE PADS - LOOKING EAST



TRUDEAU SECOND DECLARATION

EXHIBIT 2

Calculation No. 05996.02-G(B)-13, Rev. 6

STONE & WEBSTER, INC.

5010.64

CALCULATION SHEET

CLIENT & PROJECT **PRIVATE FUEL STORAGE, LLC – PFSF**	PAGE 1 OF 59 + 6 pp of ATTACHMENTS
CALCULATION TITLE **STABILITY ANALYSES OF CANISTER TRANSFER BUILDING**	QA CATEGORY (✓) ☑ I NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ (other)

CALCULATION IDENTIFICATION NUMBER				
JOB ORDER NO.	DISCIPLINE	CURRENT CALC NO	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.02	**G(B)**	**13**		

APPROVALS - SIGNATURE & DATE					CONFIRMATION REQUIRED ☑	
PREPARER(S)/DATE(S)	REVIEWER(S)/DATES(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES CALC NO. OR REV NO.	YES	NO
Original Signed By: LPSingh / 12-9-98	Original Signed By: DLAloysius / 12-10-98	Original Signed By: DLAloysiu / 12-10-98	0	N/A		✓
Original Signed By: DLAloysius / 9-3-99 SYBoakye / 9-3-99 *See page 2-1 for ID of*	Original Signed By: SYBoakye / 9-3-99 DLAloysius / 9-3-99 *Prepared / Reviewed By*	Original Signed By: TYChang / 9-3-99 TYChang / 9-3-99	1	G(C)-13 Rev. 0		✓
Original Signed By: PJTrudeau / 1-21-00	Original Signed By: TYChang / 1-21-00	Original Signed By: TYChang / 1-21-00	2	1		✓
Original Signed By: PJTrudeau / 6-19-00	Original Signed By: TYChang / 6-19-00	Original Signed By: TYChang / 6-19-00	3	2		✓
Original Signed By: SYBoakye / 3-30-01	Original Signed By: TYChang / 3-30-01	Original Signed By: TYChang / 3-30-01	4	3		✓
Original Signed By: SYBoakye / 3-30-01	Original Signed By: TYChang / 3-30-01	Original Signed By: TYChang / 3-30-01	5	4		✓
PJTrudeau / 7-26-01 Paul J. Trudeau	TYChang / 7-26-01 Thomas Y. Chang	TYChang / 7-26-01 Thomas Y. Chang	6	5		✓

DISTRIBUTION

GROUP	NAME & LOCATION	COPY SENT (✓)	GROUP	NAME & LOCATION	COPY SENT (✓)
RECORDS MGT. FILES (OR FIRE FILE IF NONE)	JOB BOOK R4.2G	ORIG			
	FIRE FILE - Denver	☑			
Geotechnical	PJTrudeau – Stoughton/3	☑			

CALCULATION IDENTIFICATION NUMBER				PAGE 2
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

TABLE OF CONTENTS

TABLE OF CONTENTS 2
RECORD OF REVISIONS 3
OBJECTIVE 7
ASSUMPTIONS/DATA 7
GEOTECHNICAL PROPERTIES 8
METHOD OF ANALYSIS 13
ANALYSIS OF OVERTURNING STABILITY 14
Checking overturning about the N-S axis 14
Checking overturning about the E-W axis 16
ANALYSIS OF SLIDING STABILITY 18
Sliding Stability of the Canister Transfer Building on In Situ Clayey Soils 18
Sliding Stability of the Canister Transfer Building on Cohesionless Soils 24
ALLOWABLE BEARING CAPACITY 30
Bearing Capacity Factors 30
Shape Factors 31
Depth Factors 31
Inclination Factors 31
Static Bearing Capacity of the Canister Transfer Building 32
Dynamic Bearing Capacity of the Canister Transfer Building 35
CONCLUSIONS 43
OVERTURNING STABILITY OF THE CANISTER TRANSFER BUILDING 43
SLIDING STABILITY OF THE CANISTER TRANSFER BUILDING 43
BEARING CAPACITY 44
Static Bearing Capacity of the Canister Transfer Building 44
Dynamic Bearing Capacity of the Canister Transfer Building 44
REFERENCES 46
TABLES 47
FIGURES 52
ATTACHMENTS
A Table 6 from Calc 05996.02-G(B)-05-2 re Strength of Clayey Silt 1 page
B Annotated copies of CPT-37 & CPT-38 Showing Relative Difference Between Deeper Lying Soils and Those Tested in UU & CU Triaxial Tests At Depths ~10 Ft 2 pages
C Annotated Copies of Direct Shear Test Plots of Horizontal Displacement vs Shear Stress 3 pages

RECORD OF REVISIONS

REVISION 0

Original Issue

REVISION 1

Page count increased from 37 to 63.

- Revised seismic loadings to correspond to the PSHA 2,000-yr return period earthquake (p. 9-1)
- Added section on dynamic strength of soils (p. 9-3)
- Added section on seismic sliding resistance of the mat foundation (p. 9-5)
- Added section on evaluation of sliding on a deep slip surface (p. 9-8)
- Updated bearing capacity analysis using revised seismic loadings (p. 34-1)
    - Added additional loading combination: static + 40% seismic uplift + 100% in x (N-S) direction + 40% in z (E-W) direction
- Added additional references (p. 36-1)

NOTE:

SYBoakye prepared/DLAloysius reviewed pp. 9-8 through 9-12. Remaining pages prepared by DLAloysius and reviewed by SYBoakye.

REVISION 2

Major re-write of the calculation.

1. Renumbered pages and figures to make the calculation easier to follow.
2. Changed effective length of mat to 265 ft to make it consistent with Calculation 05996.02-SC-4, Rev 1 (SWEC, 1999a).
3. Added overturning analysis.
4. Corrected calculation of moments for joints 3 and 6 in Table 2.6-11 and incorporated revised seismic loads in calculations of overturning stability and dynamic bearing capacity.
5. Revised dynamic bearing capacity analyses to utilize only total strength parameters because these partially saturated soils will not have time to drain fully during the rapid cycling associated with the design basis ground motion. See Calculation 05996.02-G(B)-05-1 (SWEC, 1999b) for additional details.
6. Updated references to current issues of drawings.

7. Added references to foundation profiles through Canister Transfer Building area presented in SAR Figures 2.6-21 through 23.
8. Deleted analyses of bearing capacity on layered profile, as adequate factors of safety are obtained conservatively assuming that the total strengths measured for the clayey soils in the upper ~25' to 30' layer apply for the entire profile under the Canister Transfer Building and revised all of the detailed bearing capacity analyses.
9. Changed "Load Combinations" to "Load Cases" and defined these cases to be consistent throughout the various stability analyses included herein. These are the same cases as are used in the stability analyses of the cask storage pads, Calculation 05996.02-G(B)-04-5 (SWEC, 2000).
10. Added analysis of sliding on a deep plane at the top of silty sand/sandy silt layer, incorporating passive resistance acting on the block of clayey soil and the foundation mat overlying this interface.
11. Revised Conclusions to reflect results of these changes.

REVISION 3

1. Added a 1-ft deep key around the perimeter of the Canister Transfer Building mat to permit use of the cohesive strength of the in situ silty clay/clayey silt in resisting sliding due to loads from the design basis ground motion.
2. Revised shear strength used in the sliding stability analyses of the Canister Transfer Building mat supported on the in situ silty clay to be the strength measured in the direct shear tests performed on samples obtained from elevations approximately at the bottom of the 1-ft deep perimeter key. The shear strength used in this analysis equaled that measured for stresses corresponding to the vertical stresses at the bottom of the mat following completion of construction.
3. Removed static and dynamic bearing capacity analyses based on total-stress strengths.
4. The relative strength increase noted for the deeper lying soils in the cone penetration testing that was performed within the Canister Transfer Building footprint was used to determine a weighted average undrained strength of the soils in the entire upper layer for use in the bearing capacity analyses, since the soils within a depth equal to approximately the width of the foundation are effective in resisting bearing failures. This resulted in the average undrained strength for the bearing capacity analyses of the upper layer equal to 3.18 ksf.
5. Removed dynamic analyses based on increasing strengths of the cohesive soils that were measured in static tests to reflect well known phenomenon that the strength of cohesive soils increases as the rate of loading decreases.
6. Revised undrained shear strength of the clay block overlying the cohesionless layer to 2.2 ksf, based on the UU tests that were performed at confining pressures of 1.3 ksf (reported in Attachment 2 of Appendix 2A of the SAR) in the analysis of sliding of the Canister Transfer Building on deep plane of cohesionless soils.

7. Added shearing resistance available on the ends of the block of clay, since this soil must be sheared along these planes in order for the Canister Transfer Building to slide on a deep plane of cohesionless soils.
8. Revised method of calculating the inclination factor in the bearing capacity analyses to that presented by Vesic in Chapter 3 of Winterkorn and Fang (1975). Vesic's method expands upon the theory developed by Hansen for plane strain analyses of footings with inclined loads. 0Vesic's method permits a more rigorous analysis of inclined loads acting in two directions on rectangular footings, which more closely represents the conditions applicable for the Canister Transfer Building.
9. Replaced Tables 2, 2.6-9, and 2.6-10 with revised results for the changes in shear strength of the in situ soils noted above and deleted Table 3.

REVISION 4

1. Updated stability analyses to reflect revised design basis ground motions (a_H = 0.711g & a_V = 0.695g, per Table 1 of Geomatrix, 2001).
2. Resisting moment in overturning stability analysis calculated based on resultant of static and dynamic vertical forces.
3. Updated dimensions of foundation mat to 240 ft (E-W) x 279.5 ft (N-S), and changed the depth of the perimeter key to 1.5 ft, in accordance with design change identified in Figure 4.7-1 (3 sheets), "Canister Transfer Building," of SAR Revision 21 (based on S&W Drawings 0599602-EC-404A-B & 404B-B).
4. Added definition of "m" used in the inclination factors for calculating allowable bearing capacity.
5. Updated references to supporting calculations.
6. Updated discussions and conclusions to incorporate revised results.

REVISION 5

1. Shear strength of clayey soils beneath the building for resisting sliding was changed from 1.8 ksf to 1.7 ksf to reflect lower final effective stresses under the mat after changing size of mat to 240 ft x 279.5 ft.
2. Added sliding analysis that includes both shear resistance along bottom of the plane of the clayey soils enclosed within the perimeter key at the base of the mat and the full passive resistance from the soil cement placed adjacent to the mat. Used residual strength measured in the direct shear tests that were performed on these clayey soils for this case.

CALCULATION IDENTIFICATION NUMBER				PAGE 6
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-6	N/A	

REVISION 6

1. Expanded description of soil cement properties.
2. Added discussion to clarify use of peak strengths measured in the direct shear tests along with one-half of passive resistance and residual strengths along with full passive resistance in sliding stability analysis.
3. Added calculation of horizontal displacement of the building due to elastic theory.
4. Expanded discussion of residual strengths of the clayey soils underlying the building.

OBJECTIVE

To determine the stability against overturning, sliding, and static and dynamic bearing capacity failure of the Canister Transfer Building supported on a mat foundation.

ASSUMPTIONS/DATA

The footprint of the Canister Transfer Building foundation mat is shown on SAR Figure 4.7-1, "Canister Transfer Building," and S&W Drawing 0599602-EC-404A-B & 404B-B, Canister Transfer Building – Conc Mat Foundation Plan, Sheets 1 & 2. The elevation view of the structure is shown on Sheets 2 & 3 of SAR Figure 4.7-1. The foundation mat is 240 ft (E-W) x 279.5 ft (N-S) x 5 ft thick, with a 6.5-ft wide x 1.5-ft deep foundation key along the perimeter of the mat.

Figure 1 presents a schematic view of the foundation and identifies the coordinate system used in these analyses. Figure 2 presents the stick model used in the structural analysis of the Canister Transfer Building.

The various static and dynamic loads and load combinations used in these analyses were obtained from Calculation 05996.02-SC-5-2 (S&W, 2001). All loads are transferred to the bottom of the mat. Moments, when transferred to the bottom of the mat, result in eccentricity of the applied load with respect to the center of gravity of the mat. Lateral loads, when combined with the vertical load, result in inclination of the vertical load, which decreases the allowable bearing capacity.

The generalized soil profile at the site is shown on Figure 3. The soil profile consists of ~30 ft of silty clay/clayey silt with sandy silt/silty sand layers (Layer 1), overlying ~30 ft of very dense fine sand (Layer 2), overlying extremely dense silt (N ≥100 blows/ft, Layer 3). SAR Figures 2.6-21 through 23 present foundation profiles showing the relationship of the Canister Transfer Building with respect to the underlying soils. These profiles, located as shown in SAR Figure 2.6-18, provide more detailed stratigraphic information, especially within the upper ~30-ft thick layer at the site.

The bearing capacity analyses assume that Layer 1, which consists of silty clay/clayey silt with some sandy silt/silty sand, is of infinite thickness and has strength properties based on those measured for the clayey soils within the upper layer. These assumptions simplify the analyses and they are very conservative. The strength of the sandy silt/silty sand in the upper layer is greater than that of the clayey soils, based on the increases in Standard Penetration Test (SPT) blow counts (N-values) and the increased tip resistance (see SAR Figure 2.6-5, Sheet 1) in the cone penetration testing (ConeTec, 1999) measured for these soils. The underlying soils are even stronger, based on their SPT N-values, which generally exceed 100 blows/ft.

GEOTECHNICAL PROPERTIES

Based on laboratory test results presented in Table 3 of Calculation 05996.02-G(B)-5-2 (SWEC, 2000a), γ_{moist} = 80 pcf above the bottom of the mat and 90 pcf below the mat.

Table 6 of Calc 05996.02-G(B)-05-2 (copy included in Attachment A) summarizes the results of the triaxial tests that were performed within depths of ~10 ft. The undrained shear strengths (s_u) measured in these tests are plotted vs confining pressure in Figure 6. This figure is annotated to indicate the vertical stresses existing prior to construction and following completion of construction.

The undrained shear strengths measured in the triaxial tests are used for the dynamic bearing capacity analyses because the partially saturated, fine-grained soils will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. As indicated in Figure 6, the undrained strength of the soils within ~10 ft of grade is assumed to be 2.2 ksf. This value is the lowest strength measured in the UU tests, which were performed at confining stresses of 1.3 ksf. This confining stress corresponds to the in situ vertical stress existing near the middle of the upper layer, prior to construction of these structures. It is much less than the final stresses that will exist under the cask storage pads and the Canister Transfer Building following completion of construction. Figure 6 illustrates that the undrained strength of these soils increase as the loadings of the structures are applied; therefore, 2.2 ksf is a very conservative value for use in the bearing capacity analyses of these structures.

The bearing capacity of the structures are dependant primarily on the strength of the soils in the upper ~25 to ~30-ft layer at the site. All of the borings drilled at the site indicate that the soils underlying this upper layer are very dense fine sands overlying silts with standard penetration test blow counts that exceed 100 blows/ft. The results of the cone penetration testing, presented in ConeTec(1999) and plotted in SAR Figure 2.6-5, Sheets 1 to 14, illustrate that the strength of the soils in the upper layer are much greater at depths below ~10 ft than in the range of ~5 ft to ~10 ft, where most of the triaxial test specimens were obtained.

In determining the bearing capacity of the foundation, the average shear strength of the soils along the anticipated bearing capacity failure slip surface should be used. This slip surface is normally confined to the zone within a depth below the footing equal to the minimum width of the footing. For the Canister Transfer Building, the effective width of the footing is decreased because of the large eccentricity of the load on the mat due to the seismic loading. As indicated in Table 2.6-10, the minimum effective width of the Canister Transfer Building occurs for Load Case IIIA, where B' = 119.5 ft. This is greater than the depth of the upper layer (~30 ft). Therefore, it is conservative to use the average strength of the soils in the upper layer in the bearing capacity analyses, since all of the soils in the upper layer will be effective in resisting failure along the anticipated bearing capacity slip surface.

The undrained strength used in the bearing capacity analyses presented herein is a weighted average strength that is applicable for the soils in the upper layer. This value is determined using the value of undrained shear strength of 2.2 ksf noted above for the soils tested at depths of ~10 ft and the relative strength increase measured for the soils below depths of ~12 ft in the cone penetration tests that were performed within the Canister Transfer Building footprint. As indicated on SAR Figure 2.6-18, these included CPT-37 and CPT-38. Similar increases in undrained strength for the deeper lying soils were also noted in all of the other CPTs performed in the pad emplacement area.

Attachment B presents copies of the plots of s_u vs depth for CPT-37 and CPT-38, which are included in Appendix D of ConeTec(1999). These plots are annotated to identify the average undrained strength of the cohesive soils measured with respect to depth. As shown by the plot of s_u for CPT-37, the weakest zone exists between depths of ~5 ft and ~12 ft. The results for CPT-38 are similar, but the bottom of the weakest zone is at a depth of ~11 ft. The underlying soils are all much stronger. The average value of s_u of the cohesive soils for the depth range from ~18 ft to ~28 ft is ~2.20 tsf, compared to s_u ~1.34 tsf for the zone between ~5 ft and ~12 ft. Therefore, the undrained strength of the deeper soils in the upper layer was ~64% (Δs_u = 100% x [(2.20 tsf – 1.34 tsf) / 1.34 tsf] higher than the strength measured for the soils within the depth range of ~5 ft to ~12 ft. The relative strength increase was even greater than this in CPT-38.

Using 2.2 ksf, as measured in the UU triaxial tests performed on specimens obtained from depths of ~10 ft, as the undrained strength applicable for the weakest soils (i.e., those in the depth range of ~5 ft to ~12 ft), the average strength for the soils in the entire upper layer is calculated as shown in Figure 4. The resulting average value, weighted as a function of the depth, is s_u ~3.18 ksf. This value would be much higher if the results from CPT-38 were used; therefore, this is considered to be a reasonable lower-bound value of the average strength applicable for the soils in the upper layer that underlie the Canister Transfer Building.

Further evidence that this is a conservative value of s_u for the soils in the upper layer is presented in Figure 6. This plot of s_u vs confining pressure illustrates that this value is slightly less than the average value of s_u measured in the CU triaxial tests that were performed on specimens obtained from depths of ~10 ft at confining stresses of 2.1 ksf. As indicated in this figure, the confining stress of 2.1 ksf used to test these specimens is comparable to the vertical stress that will exist ~7 ft [(2.1 ksf – 1.46 ksf) ÷ 0.09 kcf] below the Canister Transfer Building mat following completion of construction. Since these tests were performed on specimens of the weakest soils underlying the Canister Transfer Building mat (the deeper lying soils are stronger based on the SPT and the cone penetration test data), it is conservative to use the weighted average value of s_u of 3.18 ksf for the soils in the entire upper layer of the profile in the bearing capacity analyses.

Direct shear tests were performed on undisturbed specimens of the silty clay/clayey silt obtained from Borings CTB-6 and CTB-S, which were drilled in the locations shown in SAR Figure 2.6-18. These specimens were obtained from Elevation ~4469, approximately the elevation of the bottom of the perimeter key proposed at the base of Canister Transfer

Building mat. Note, this key is being constructed around the perimeter of the mat to ensure that the full shear strength of the clayey soils is available to resist sliding of the structure due to loads from the design basis ground motion. These direct shear tests were performed at normal stresses that ranged from 0.25 ksf to 3.0 ksf. This range of normal stresses bounds the ranges of stresses expected for static and dynamic loadings from the design basis ground motion.

The results of these tests are presented in Attachments 7 and 8 of the Appendix 2A of the SAR and they are plotted in Figures 7 and 8. Because of the fine grained nature of these soils, they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. Therefore, sliding stability analyses included below of the Canister Transfer Building constructed directly on the silty clay are performed using the average shear strength measured in these direct shear tests for a normal stress equal to the vertical stress under the building following completion of construction, but prior to imposition of the dynamic loading due to the earthquake. As shown in Figures 7 and 8, this average shear strength is 1.7 ksf and the friction angle is set equal to 0°.

Effective-stress strength parameters are estimated to be $\phi = 30°$ and c = 0 ksf, even though these soils may be somewhat cemented. This value of ϕ is based on the PI values for these soils, which ranged between 5% and 23% (SWEC, 2000a), and the relationship between ϕ and PI presented in Figure 18.1 of Terzaghi & Peck (1967).

Therefore, static bearing capacity analyses are performed using the following soil strengths:

Case IA Static using undrained strength parameters: $\phi = 0°$ & c = 3.18 ksf.

Case IB Static using effective-stress strength parameters: $\phi = 30°$ & c = 0.

and dynamic bearing capacity analyses are performed using $\phi = 0°$ & c = 3.18 ksf.

Soil Cement Properties:

The unit weight of the soil cement is assumed to be 100 pcf in the analyses included herein and the unconfined compressive strength is 250 psi. (Initial results of the soil-cement testing indicate that 110 pcf is a reasonable lower-bound value for the total unit weight of the soil cement adjacent to the Canister Transfer Building foundation.) This strength is consistent with the soil-cement mix proposed for use within the frost zone adjacent to the cask storage pads and is based on the assumption that the strength will be at least this value to obtain a soil cement mix design that will satisfy the durability requirements of the ASTM wet/dry and freeze/thaw tests.

PFS is developing the soil-cement mix design using standard industry practice, in accordance with the criteria specified by the Portland Cement Association. 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, Rev. 0. 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)-04-8:

> *"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 PCA[1]:

> *"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.*

[1] Portland Cement Association, "Soil-Cement Laboratory Handbook," Skokie, IL, 1971.

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 lifts of 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[2] in his testing of bond along soil-cement interfaces. 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 (p. 2.6-113 of SAR) and during construction to demonstrate that the required interface strengths are achieved (p. 2.6-114 of SAR). 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.

[2] DeGroot, G., 1976, "Bonding Study on Layered Soil Cement", REC-ERC-76-16, U.S. Bureau of Reclamation, Denver, CO, September 1976.

METHOD OF ANALYSIS

Load cases analyzed consist of combinations of vertical static, vertical dynamic (compression and uplift, Y-direction), and horizontal dynamic (in X and Z-directions) loads.

The following load combinations are analyzed:

Case I	Static
Case II	Static + dynamic horizontal forces due to the earthquake
Case III	Static + dynamic horizontal + vertical uplift forces due to the earthquake
Case IV	Static + dynamic horizontal + vertical compression forces due to the earthquake

For Case II, 100% of the dynamic lateral forces in both X and Z directions are combined. For Cases III and IV, 100% of the dynamic loading in one direction is assumed to act at the same time that 40% of the dynamic loading acts in the other two directions. For these cases, the suffix "A" is used to designate 40% in the X direction (N-S for the Canister Transfer Building, as shown in Figure 1), 100% in the Y direction (vertical), and 40% in the Z direction (E-W). Similarly, the suffix "B" is used to designate 40% in the X direction, 40% in the Y, and 100% in the Z, and the suffix "C" is used to designate 100% in the X direction and 40% in the other two directions. Thus,

Case IIIA	40% N-S direction,	-100% Vertical direction,	40%	E-W direction.
Case IIIB	40% N-S direction,	-40% Vertical direction,	100%	E-W direction.
Case IIIC	100% N-S direction,	-40% Vertical direction,	40%	E-W direction.

The negative sign for the vertical direction in Case III indicates uplift forces due to the earthquake. Case IV is the same as Case III, but the vertical forces due to the earthquake act downward in compression; therefore, the signs on the vertical components are positive.

Combining the effects of the three components of the design basis ground motion in this manner is in accordance with ASCE-4 (1986).

ANALYSIS OF OVERTURNING STABILITY

The factor of safety against overturning is defined as:

$FS_{OT} = \Sigma M_{Resisting} \div \Sigma M_{Driving}$

The overturning stability of the Canister Transfer Building is determined using the dynamic loads for the building due to the PSHA 2,000-yr return period earthquake. These loads are listed in Table 2.6-11, and they were developed based on the dynamic analysis performed in Calculation 05996.02-SC-5 (S&W, 2001) and described in SAR Section 4.7.1.5.3. The masses and accelerations of the joints (see Figure 2 for locations of the joints) used in the model of the Canister Transfer Building in Calculation 05996.02-SC-5 are listed on the left side of Table 2.6-11, and the resulting inertial forces and associated moments are listed on the right. Based on building geometry shown schematically in Figure 1 and the forces and moments shown in Table 2.6-11, overturning is more critical about the N-S axis (279.5 ft) than about the E-W axis (240 ft). Page 37 of Calculation 05996.02-SC-5 indicates that the moment due to angular (rotational) acceleration of the structure is 465,729 ft-K about the N-S axis and 1,004,332 ft-K about the E-W axis.

The vertical force due to the earthquake can act upward or downward. However, when it acts downward, it acts in the same direction as the weight, tending to stabilize the structure with respect to overturning stability. The minimum factor of safety against overturning will occur when the maximum dynamic vertical force acts in the upward direction, tending to unload the mat and reduce the resisting moment. Therefore, calculate the factor of safety for Case III.

CHECKING OVERTURNING ABOUT THE N-S AXIS

For Case IIIA, where 40% of the horizontal force due to the earthquake act in the N-S and E-W directions and 100% acts vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 79,779 K, (i.e., Weight – Total $F_{V\,Dyn}$), as shown in Table 2.6-11. For overturning about the N-S axis, the moment arm for the resisting moment equals ½ of 240 ft, or 120 ft. Therefore,

$\Sigma M_{Resisting} = (97{,}749 - 79{,}779)\text{ K} \times 120\text{ ft} = 2{,}156{,}400\text{ ft-K}.$

This ignores the eccentricities of the vertical masses with respect to the center of the mat. Incorporating these eccentricities, which are included in Attachment A of Calc 05996.02-SC-5, Rev. 2, the resulting resisting moment is calculated as follows:

JOINT	EL.	MASS Y k-sec²/ft	AY g's	Z (E-W) ft	Moment Arm E-W ft	$SM_{@N\text{-}S}$ ft-K
0	94.25	260.1	0.783	0	120.00	218,002
1	95	1,908.0	0.783	-0.73	119.27	1,589,353
2	130	420.4	0.821	-2.02	117.98	285,292
3	170	304.3	0.913	-3.14	116.86	99,412
4	190	117.1	0.928	0	120.00	32,638
5	190	27.6	1.840	0	120.00	-89,478
6	170	1.0	0	0	120.00	3,860
					Total =	**2,139,080**

The driving moments include 40% of the ΣM acting about the N-S axis, $\Sigma M_{@X}$ in Table 2.6-11, which is 0.4 x 2,706,961.4 = 1,082,785 ft-K, and 40% of the moment about the N-S axis due to angular (rotational) acceleration of the structure, which is 0.4 x 465,729 = 186,292 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the E-W axis do not contribute to overturning about the N-S axis; therefore,

$$\sum M_{Driving} = \sqrt{1,082,785^2 + (186,292)^2} = 1,098,694 \quad \text{ft} - \text{K}$$

and **FS_{OT}** = 2,156,400 ÷ 1,098,694 = **1.96**

about the N-S axis for Case IIIA without including eccentricities of vertical masses.

Including the effect of the eccentricities of the vertical masses, the resulting factor of safety against overturning is:

FS_{OT} = 2,139,080 ÷ 1,098,694 = **1.95** **<u>(Minimum)</u>**

For Case IIIB, where 100% of the horizontal force due to the earthquake acts in the E-W direction and 40% acts in the N-S direction and vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 40% of 79,779 K, (i.e., Weight – Total $F_{V\ Dyn}$), as shown in Table 2.6-11. For overturning about the N-S axis, the moment arm for the resisting moment equals ½ of 240 ft, or 120 ft. Therefore,

$\Sigma M_{Resisting}$ = (97,749 – 0.4 x 79,779) K x 120 ft = 7,900,488 ft-K.

The driving moments include 100% of the ΣM acting about the N-S axis, $\Sigma M_{@X}$ in Table 2.6-11, which is 2,706,961.4 ft-K, and 100% of the moment about the N-S axis due to angular (rotational) acceleration of the structure, which is 465,729 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the E-W axis do not contribute to overturning about the N-S axis; therefore,

$$\sum M_{Driving} = \sqrt{2{,}706{,}961.4^2 + 465{,}729^2} = 2{,}746{,}733 \quad \text{ft} - \text{K}$$

and FS_{OT} = 7,900,488 ÷ 2,746,733 = **2.88** about the N-S axis for Case IIIB.

Case IIIC, where 100% of the horizontal force due to the earthquake acts in the N-S direction and 40% acts in the E-W direction and vertically upward, **is less critical** for overturning about the N-S axis than Case IIIB.

CHECKING OVERTURNING ABOUT THE E-W AXIS

For Case IIIA, where 40% of the horizontal force due to the earthquake act in the N-S and E-W directions and 100% acts vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 79,779 K, (i.e., Weight – Total $F_{V\,Dyn}$), as shown in Table 2.6-11. For overturning about the E-W axis, the moment arm for the resisting moment equals ½ of 279.5 ft, or 139.75 ft. Therefore,

$\Sigma M_{Resisting}$ = (97,749 – 79,779) K x 139.75 ft = 2,511,308 ft-K.

This ignores the eccentricities of the vertical masses with respect to the center of the mat. Incorporating these eccentricities, the resulting resisting moment is calculated as follows:

JOINT	EL.	MASS Y k-sec²/ft	AY g's	Moment Arm N-S ft	$SM_{@E\text{-}W}$ ft-K
0	94.25	260.1	0.783	139.75	253,882
1	95	1,908.0	0.783	138.08	1,840,009
2	130	420.4	0.821	131.46	317,889
3	170	304.3	0.913	143.18	121,802
4	190	117.1	0.928	139.75	38,010
5	190	27.6	1.840	139.75	-104,205
6	170	1.0	0	139.75	4,496
				Total =	**2,471,883**

The driving moments include 40% of the ΣM acting about the E-W axis, $\Sigma M_{@z}$ in Table 2.6-11, which is 0.4 x 2,849,703 = 1,139,881 ft-K, and 40% of the moment about the E-W axis due to angular (rotational) acceleration of the structure, which is 0.4 x 1,004,322 = 401,729 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the N-S axis do not contribute to overturning about the E-W axis; therefore,

$$\sum M_{Driving} = \sqrt{1{,}139{,}881^2 + 401{,}729^2} = 1{,}208{,}601 \quad ft-K$$

and FS_{OT} = 2,511,308 ÷ 1,208,601 = **2.07**

about the E-W axis for Case IIIA without including eccentricities of vertical masses.

Including the effect of the eccentricities of the vertical masses, the resulting factor of safety against overturning is:

FS_{OT} = **2,471,883** ÷ 1,208,601 = **2.05 (Minimum @ E-W Axis)**

For Case IIIC, where 100% of the horizontal force due to the earthquake acts in the N-S direction and 40% acts in the E-W direction and vertically upward, the resisting moment is calculated as the net effective weight of the building x the distance from one edge of the mat to the center of the mat. The net effective weight of the building is 97,749 – 40% of 79,779 K, (i.e., Weight – Total Fv_{Dyn}), as shown in Table 2.6-11. For overturning about the E-W axis, the moment arm for the resisting moment equals ½ of 279.5 ft, or 139.75 ft. Therefore,

$\Sigma M_{Resisting}$ = (97,749 – 0.4 x 79,779) K x 139.75 ft = 9,200,777 ft-K.

The driving moments include 100% of the ΣM acting about the E-W axis, $\Sigma M_{@z}$ in Table 2.6-11, which is 2,849,703.4 ft-K, and 100% of the moment about the E-W axis due to angular (rotational) acceleration of the structure, which is 1,004,322 ft-K.

The square root of the sum of the squares (SRSS) is used to combine the moments to account for the fact that the maximum responses of earthquake do not act in all three orthogonal directions and angular rotations at the same time. The moments acting about the N-S axis do not contribute to overturning about the E-W axis; therefore,

$$\sum M_{Driving} = \sqrt{2{,}849{,}703^2 + 1{,}004{,}322^2} = 3{,}021{,}501 \quad ft-K$$

and FS_{OT} = 9,200,777 ÷ 3,021,501 = **3.05** about the E-W axis for Case IIIC.

Case IIIB is less critical for overturning about the N-S axis than Case IIIC.

ANALYSIS OF SLIDING STABILITY

The factor of safety (FS) against sliding is defined as follows:

$$FS = \text{Resisting Force} \div \text{Driving Force} = T \div V$$

For this analysis, ignoring passive resistance of the soil adjacent to the mat, the resisting, or tangential shear force, T, below the base of the pad is defined as follows:

$$T = N \tan \phi + c\,B\,L$$

where, N (normal force) = $\Sigma F_v = F_{v\,Static} + F_{v\,Eqk}$

$\phi = 0°$ (for Silty Clay/Clayey Silt)

c = 1.7 ksf, as discussed above under "Geotechnical Properties."

B = 240 feet

L = 279.5 feet

The driving force, V, is calculated as follows:

$$V = \sqrt{F^2_{HN-S} + F^2_{HE-W}}$$

SLIDING STABILITY OF THE CANISTER TRANSFER BUILDING ON IN SITU CLAYEY SOILS

Based on Half of the Passive Resistance of the Soil Cement and the Peak Strength of the Clayey Soils Under the Building

The sliding stability of the CTB was evaluated using the foundation loadings developed in the soil-structure interaction analyses (Calculation 05996.02-SC-5, S&W, 2001). In this case, the strength of the clayey soils at the bottom of the 1.5-ft deep key around the CTB mat was based on the average of the two sets of direct shear tests performed on samples of soils obtained from beneath the CTB, approximately at the elevation proposed for founding the structure. The results of these tests are included in Attachments 7 and 8 of Appendix 2A of the SAR, and Figures 7 and 8 present plots of peak shear stress vs normal stress measured in these tests. As discussed above under Geotechnical Properties, $\phi = 0°$ and a shear strength of 1.7 ksf were used for the clayey soils underlying the Canister Transfer Building in determining resisting forces for the earthquake loading combinations.

The unconfined compressive strength of the soil cement adjacent to the Canister Transfer Building will be at least 250 psi. These analyses assume that the peak shear strength of the clayey soils under the Canister Transfer Building are available to resist sliding along with up to half of the passive resistance of the soil cement.

The backfill to be placed around the Canister Transfer Building mat and 1.5-ft deep key will be soil cement, constructed from the eolian silt and silty clay that was excavated from the area. For soil cement constructed using these soils, it is reasonable to assume the lower bound value of γ is 100 pcf, $\phi = 0°$ & c = 125 psi.

For the soil cement, $P_p = 2c \times D_f \times (B \text{ or } L)$

For 5' of soil cement, using a factor of safety of 2 applied to the passive resistance,

$$P_p = \frac{2 \times c \times D_f \times w}{FS} = \frac{2 \times 125 \frac{\#}{in.^2} \times \frac{144 \cdot in.^2}{ft^2} \times \frac{K}{1{,}000\#} \times 5\text{ ft} \times 1\frac{ft}{LF}}{2} = 90\frac{K}{LF}$$

The CTB mat is 240' wide in the E-W direction and 279.5' long in the N-S direction; therefore, the passive force available to resist sliding is at least 240' x 90 K/LF = 21,600 K acting in the N-S direction in the analyses that use half of the passive resistance of the soil cement adjacent to the mat.

The effects of wall movement on wall pressure are defined in DM-7[3] (p. 7.2-60) as the ratio of horizontal displacement to the height of the wall. For stiff cohesive soils, the wall rotation or yield ratio, y/H, required to fully mobilize passive resistance is 0.02, or 2%. For dense cohesionless soils, even less movement is required to reach full passive, ~0.2%. Lambe & Whitman (1969, p 166) also indicates that little horizontal compression, ~0.5%, is required to reach half of full passive resistance for dense sands. The soil cement will be compacted to a dense state, and once it cures, it is expected to be stiffer than dense sand, requiring less displacement to reach full passive resistance. Therefore, it is conservative to assume that half of the total passive resistance is available to resist sliding of the building.

Note, if we assume that the soil cement is comparable in stiffness to stiff cohesive soil, the figure from DM-7 cited above indicates that yield ratio, y/H, required to fully mobilize passive resistance is 2%. It is reasonable to use a yield ratio of half of this, or ~1% of the 5 ft height of the mat + 1.5-ft deep key, to reach half of passive resistance for the soil cement adjacent to the mat. This indicates that a horizontal displacement of the mat = 0.01 x 6.5 ft x 12 in./ft = 0.78 in. would be sufficient to reach half of the passive resistance. Since there are no safety-related systems that would be severed or otherwise impacted by movements of this small magnitude, it is reasonable to use this passive thrust to resist sliding. The following analysis demonstrates that it is also reasonable to use the resistance provided by the peak shear strength of the clayey soils enclosed within the perimeter key at the base of the mat to resist sliding in this case, because this amount of horizontal displacement can be obtained from elastic deformation of the clayey soils underlying the building.

The horizontal displacement of the Canister Transfer Building is estimated using elastic theory, as described in Section 4.3, "Rectangles Subjected to Shear Loading," of Poulos and Davis[4].

$$\rho = \frac{q \times a \times I}{E} \quad \text{Eq. 4.9 Poulos \& Davis}$$

---

3 NAVFAC (1986), DM 7.2, "Foundations and Earth Structures," Dept of the Navy, Naval Facilities Eng'g. Command, Alexandria, VA.

4 Poulos, H. G., and Davis, E. H., Elastic Solutions for Soil and Rock Mechanics, John Wiley & Sons, New York, NY, 1974.

$$G_s = \rho \times V_s^2 = \frac{80\text{ pcf}}{32.2\,\frac{\text{ft./sec}^2}{g}} \times (540\text{ ft/sec})^2 = 724{,}472\text{ psf} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 = 5{,}031\text{ psi}$$

$$E_s = 2 \times (1 + \nu) \times G_s = 2 \times (1 + 0.4) \times 5{,}031\text{ psi} = 14{,}087\text{ psi}$$

In the E-W direction (See Table 2.6-11 for horizontal shear values):

$$q = \frac{99{,}997\text{ K}}{240\text{ ft} \times 279.5\text{ ft}} = 1.49\text{ ksf} \times \frac{1{,}000\text{ lbs}}{\text{K}} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 = 10.4\text{ psi}$$

$$\frac{h}{b} = \frac{6.5\text{ ft}}{279.5\text{ ft}} = 0.023$$

$$\frac{b}{a} = \frac{279.5\text{ ft}}{240\text{ ft}} = 1.17$$

In the N-S direction:

$$q = \frac{111{,}108\text{ K}}{240\text{ ft} \times 279.5\text{ ft}} = 1.66\text{ ksf} \times \frac{1{,}000\text{ lbs}}{\text{K}} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 = 11.5\text{ psi}$$

$$\frac{h}{b} = \frac{6.5\text{ ft}}{240\text{ ft}} = 0.027$$

$$\frac{b}{a} = \frac{240\text{ ft}}{279.5\text{ ft}} = 0.859$$

From Figure 4.17 of Poulos & Davis, estimate the horizontal displacement factor for the corners for horizontal shear of a horizontal rectangle. For the h/b and b/a values shown above, $I_{E\text{-}W} = 0.62$ and $I_{N\text{-}S} = 0.59$.

$$\rho_{E-W} = \frac{10.4\text{ psi} \times 240\text{ ft} \times 12\,\frac{\text{in.}}{\text{ft}} \times 0.62}{14{,}087\text{ psi}} = 1.32\text{ inches} \quad \text{Eq. 4.9 Poulos \& Davis}$$

$$\text{Yield Ratio} = \frac{\rho}{H} = \frac{1.32\text{ in.}}{6.5\text{ ft} \times 12\,\frac{\text{in.}}{\text{ft}}} = 0.017\text{, or }1.7\%$$

$$\rho_{N-S} = \frac{11.5\ \text{psi} \times 279.5\ \text{ft} \times 12\ \frac{\text{in.}}{\text{ft}} \times 0.59}{14{,}087\ \text{psi}} = 1.62\ \text{inches} \quad \text{Eq. 4.9 Poulos \& Davis}$$

$$\text{Yield Ratio} = \frac{\rho}{H} = \frac{1.62\ \text{in.}}{6.5\ \text{ft} \times 12\ \frac{\text{in.}}{\text{ft}}} = 0.021,\ \text{or}\ 2.1\%$$

Thus, based on the shear modulus estimated from the shear wave velocity of the surficial silty clay/clayey silt, the horizontal displacement of the CTB subjected to the full horizontal earthquake load is calculated to be about 1.3 to 1.6 inches using the elastic solution of a buried horizontal rectangle subjected to shear in an elastic half-space. This horizontal displacement corresponds to a yield ratio, defined as horizontal displacement ÷ height of wall, of 2% from translation of the 6.5 ft height of the CTB foundation mat adjacent to the soil cement. This yield ratio is larger than the yield ratio required to mobilize one half of full passive resistance for dense sand or stiff cohesive soils. This displacement is sufficient to develop full passive resistance in the soil cement adjacent tot he mat; therefore, it is conservative to use one-half of the passive resistance in these analyses

The results of the sliding stability analysis of the Canister Transfer Building for this case are presented in Table 2.6-13. In this table, the components of the driving and resisting forces are combined using the SRSS rule. All of these factors of safety are greater than 1.1, the minimum required value. These results indicate that the factors of safety are acceptable for all load combinations examined. The lowest factor of safety is 1.15, which applies for Cases IIIC and IVC, where 100% of the dynamic earthquake forces act in the N-S direction and 40% act in the other two directions.

These results are conservative, because they assume that only one-half of the passive pressures are available to resist sliding and no credit is taken for the fact that the strength of cohesive soils increases as the rate of loading increases. Note, Newmark and Rosenblueth (1973) indicate:

> *"In all cohesive soils reported to date, strength and stiffness increase markedly with strain rate (Figs. 13.6 and 13.7). An increase of the order of 40 percent is common for the usual strain rates of earthquakes, above the strength and stiffness of static tests."*

Schimming et al, (1966), Casagrande and Shannon (1948, and Das (1993) all report similar increases in strength of cohesive soils due to rapid loading. Therefore, since these results are based on static shear strengths, they represent conservative lower-bound values of the factor of safety against sliding of the Canister Transfer Building founded on in situ silty clay/clayey silt with soil-cement backfill around the mat.

Based on the Full Passive Resistance of the Soil Cement and the Residual Strength of the Clayey Soils Under the Building

Before a complete sliding failure can occur, the full passive resistance of the soil cement must be engaged. Because the horizontal displacements associated with reaching the full passive state typically are large for soils, in the analyses where the full passive resistance of the soil cement adjacent to the mat is used, the shear strength of the clayey soils under the building is reduced to a conservative estimate of the residual shear strength based on the results of the direct shear tests.

The results of the direct shear tests, presented as plots of shear stress vs horizontal displacement in Attachment 7 of Appendix 2A of the SAR (annotated copies are included in Attachment C of this calculation), illustrate that the residual strength of these soils is nearly equal to the peak strength for those specimens that were tested at confining stresses of 2 ksf. For example, for Sample U-1C from Boring C-2, at horizontal displacements of ~0.025" past the peak strength, there is ~1.5% reduction in the shear strength indicated. The results for Sample U-1AA from Boring CTB-S showed no decrease in shear strength following the peak at ~0.025" horizontal displacement, and Samples U-3B&C from Boring CTB-6 showed a decrease of ~5%. The specimens that were tested at confining stresses of 1 ksf all show reductions of ~20% at horizontal displacements of ~0.025" past the peak.

The final effective vertical stresses at the base of the Canister Transfer Building, σ'_v, are ~1.5 ksf, now that the mat has been changed to 240 ft x 279.5 ft. This value is approximately half-way between the confining stresses of 1 and 2 ksf used for several of the direct shear tests. The residual strength of the clayey soils beneath the building are expected to show reductions from the peak strength of ~10% to ~12.5%; i.e., approximately half-way between the reductions observed for the specimens tested at confining stresses of 1 ksf and 2 ksf, since the final effective stresses under the building are ~1.5 ksf; i.e., approximately half-way between confining stresses used in these tests (1 ksf and 2 ksf). Therefore, it is reasonable to assume that the peak strength of the clayey soils enclosed within the perimeter key at the base of the Canister Transfer Building mat should be reduced to account for horizontal displacement required to reach full passive resistance of the soil cement adjacent to the mat. Based of the results of the direct shear tests performed on samples of the site soils, it would be reasonable to use a reduction of ~10% to ~12.5% to obtain the residual strength applicable for the final vertical stresses at the base of the Canister Transfer Building. The analyses that follow, however, reduce the peak strength even more than this, by a total of 20%, to provide additional conservatism.

The following table illustrates further that using a reduction of the peak strength equal to 20% provides a conservative estimation of the residual strength of these soils. This table presents the peak strengths measured in the direct shear tests at normal stresses of 1 ksf and 2 ksf. It also lists the final shear strengths measured in these tests, which were generally obtained at horizontal displacements of 0.25 inches or 0.30 inches. The table also lists the calculated post-peak strength reduction for these test results, as well as the average post-peak strength reduction for normal stress of 1.5 ksf, which is applicable for the state of stress existing under the Canister Transfer Building mat. Note, that the average post-peak strength reduction for normal stress of 1.5 ksf for the three direct shear tests is only 15.6% for these very high shear displacements in the direct shear tests. The maximum value of the average the post-peak strength reductions for normal stress of 1.5 ksf occurred for Sample U-3B&C in CTB-6, and it equaled 20.8%. If the results of this test were used to define the residual strength of these soils, the analyses would be performed at c = 1.5 ksf, the average of the post-peak strengths measured at the maximum shear displacements in these tests for normal stresses of 1 ksf and 2 ksf. This would result in higher factors of safety than are calculated and presented in Table 2.6-14, based on c = 1.36 ksf.

CALCULATION OF AVERAGE POST-PEAK STRENGTH REDUCTION FOR NORMAL STRESS APPLICABLE TO FINAL TRESSES UNDER THE CANISTER TRANSFER BUILDING

Boring	Sample	Normal Stress = 1 ksf			Normal Stress = 2 ksf			Average Post-Peak Strength Reduction for Normal Stress = 1.5 ksf
		Peak Strength	Strength at Maximum Shear Displacement	Post-Peak Strength Reduction	Peak Strength	Strength at Maximum Shear Displacement	Post-Peak Strength Reduction	
		ksf	ksf	%	ksf	ksf	%	%
C-2	**U-1C**	1.67	1.2	28.1	2.13	2.1	1.4	**14.8**
CTB-6	**U-3B&C**	1.57	1.1	29.9	2.15	1.9	11.6	**20.8**
CTB-S	**U-1AA**	1.42	1.1	22.5	1.58	1.7	~0.0	**11.3**
							Average	**= 15.6**

The results of the sliding stability analysis of the Canister Transfer Building for this case are presented in Table 2.6-14. In this table, the components of the driving and resisting forces are combined using the SRSS rule. All of these factors of safety are greater than 1.1, the minimum required value. These results indicate that the factors of safety are acceptable for all load combinations examined. The lowest factor of safety is 1.26, which applies for Cases IIIC and IVC, where 100% of the dynamic earthquake forces act in the N-S direction and 40% act in the other two directions. These results demonstrate that there is additional margin available to resist sliding of the building due to the earthquake loads, even when very conservative estimates of the residual shear strength of the clayey soils are used.

Sliding Stability of the Canister Transfer Building on Cohesionless Soils

The Canister Transfer Building will be founded on clayey soils that have an adequate amount of cohesive strength to resist sliding due to the dynamic forces from the design basis ground motion. As shown in SAR Figures 2.6-21 through 2.6-23, however, some of the soils underlying the building are cohesionless within the depth zone of about 10 to 20 ft, especially near the southern portion of the building. Analyses presented on the next six pages address the possibility that sliding may occur along a deeper slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces.

The resistance to sliding is greatly reduced for frictional materials when the dynamic forces due to the earthquake act upward. The normal forces act downward for Case IV loadings and, hence, the resisting forces will be much greater than those for Case III. Therefore, these analyses are performed only for Load Cases IIIA, IIIB, and IIIC. As described above, these load cases are defined as follows:

Case IIIA	40%	N-S direction,	-100%	Vertical direction,	40%	E-W direction.
Case IIIB	40%	N-S direction,	-40%	Vertical direction,	100%	E-W direction.
Case IIIC	100%	N-S direction,	-40%	Vertical direction,	40%	E-W direction.

As shown in SAR Figures 2.6-21 through 2.6-23, the top of the cohesionless layer varies from about 5 ft to about 9 ft below the mat, and it generally is at a depth of about 6 ft below the mat. These analyses include the passive resistance acting on a plane extending from grade down to the top of the cohesionless layer, plus the shear strength available at the ends of the silty clay block under the mat, plus the frictional resistance available along the top of the cohesionless layer. The weight of the clayey soils existing between the top of the cohesionless soils and the bottom of the mat is included in the normal force used to calculate the frictional resistance acting along the top of the cohesionless layer.

A review of the cone penetration test results (ConeTec, 1999) obtained within the top 2 ft of the layer of nonplastic silt/silty sand/sandy silt underlying the Canister Transfer Building indicated that $\phi = 38°$ is a reasonable minimum value for these soils. This review is presented on the next page.

The next five pages illustrate that the factor of safety against sliding along the top of this layer is >1.1 for all load cases (i.e., Load Cases IIIA, IIIB, and IIIC). These analyses include several conservative assumptions. They are based on static strengths of the silty clay block under the Canister Transfer Building mat, even though, as reported in Das (1993), experimental results indicate that the strength of cohesive soils increases as the rate of loading increases. For rates of strain applicable for the cyclic loading due to the design basis ground motion, Das indicates that for most practical cases, one can assume that $c_{u\ dynamic} \sim 1.5 \times c_{u\ static}$. In addition, the silty sand/sandy silt layer is not continuous under the Canister Transfer Building mat, and this analysis neglects cementation of these soils that was observed in the samples obtained in the borings. Therefore, sliding is not expected to occur along the surface of the cohesionless soils underlying the Canister Transfer Building.

NOTED JAN 2 1 2000 P. J. Trulear

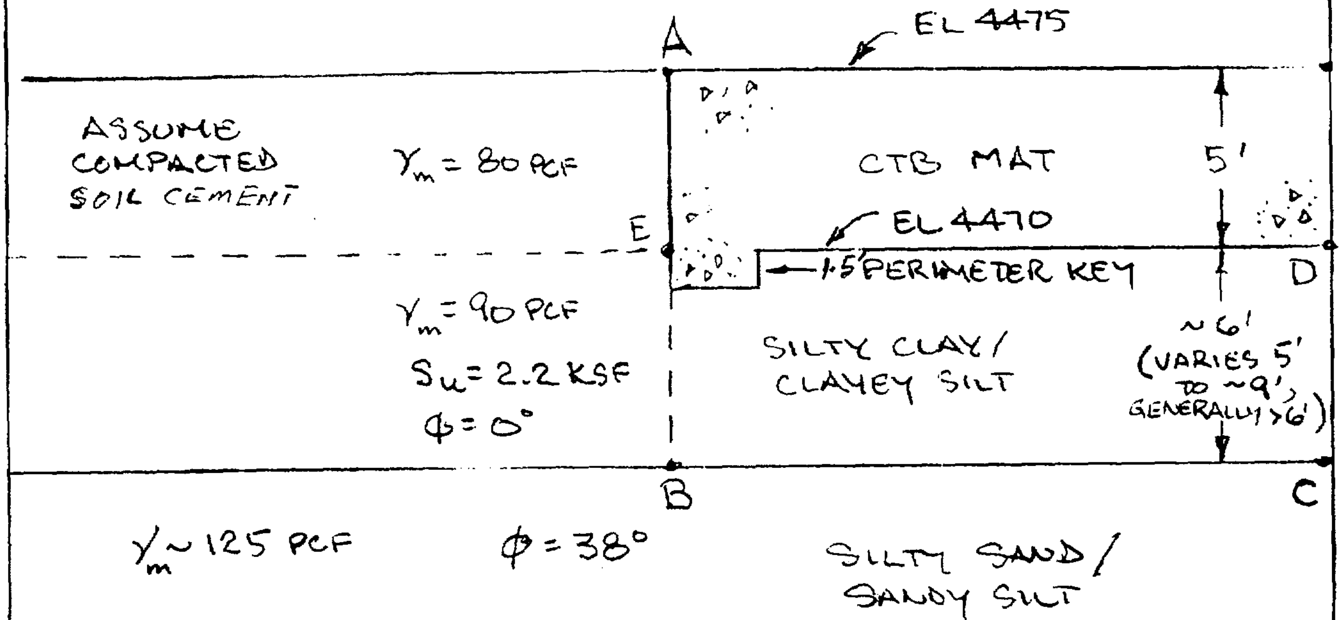
STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 25
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 13-6	OPTIONAL TASK CODE	

SLIDING ON DEEP PLANE AT TOP OF SILTY SAND/ SANDY SILT LAYER



NOTE: VALUE OF ϕ BASED ON ϕ DATA FROM CPT-37 & 38 PRESENTED IN CONETEC (1999)

ID	~DEPTH OF SILTY SAND	MIN ϕ	MAX ϕ	AVG ϕ	MEDIAN ϕ	ϕ IN TOP 2'
CPT-37	~11.6' TO ~18.7'	36*	44	40	40	~38
CPT-38	~11' TO ~18'	38	46	43	44	~38

PASSIVE PRESSURES ACTING ON PLANE AB WILL INCREASE AS B GETS DEEPER IN THE SILTY SAND / SANDY SILT LAYER; ∴ USE ϕ NEAR THE TOP OF THE LAYER. ⇒ $\phi = 38°$.

N VALUES ARE HIGH, GENERALLY ≫ 20 BL/FT; ∴ $\phi = 38°$ IS REASONABLE

* EXCLUDING SINGLE VALUE OF $\phi = 34°$ AT $z = 13.8'$

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 13-6	OPTIONAL TASK CODE	PAGE 26

SLIDING ON DEEP COHESIONLESS PLANE

$$FS_{SLIDING} = \frac{\Sigma \text{ RESISTING FORCES}}{\Sigma \text{ DRIVING FORCES}}$$

RESISTING FORCES INCLUDE PASSIVE RESISTANCE AVAILABLE ALONG AB + SHEAR RESISTANCE ALONG ENDS OF BLOCK BCDE + FRICTION ALONG BC.

1. PASSIVE RESISTANCE AVAILABLE ALONG AB

INCLUDES $(2 \times 5 \times 125 \times .144 \frac{K}{FT^2}) \times (5') = 180$ K/LF FOR COMPACTED 5' SOIL-CEMENT ADJACENT TO 5' MAT

$+ \frac{1}{2} \gamma H^2 K_P + q_S H K_P + 2cH\sqrt{K_P}$ FOR 5' BLOCK OF SILTY CLAY UNDERLYING THE COMPACTED SOIL-CEMENT

$$\frac{1}{2}\left(0.090 \frac{K}{FT^3}\right) \times (5\,FT)^2 \times \overset{K_P}{1.0} + \overset{H}{6\,FT} \times \overset{\gamma}{0.080 \frac{K}{FT^3}} \times 5\,FT \times \overset{K_P}{1.0}$$

$$+ 2 \times \overset{c}{2.2} \frac{K}{FT^2} \times 5\,FT \times \sqrt{1.0} = 1.125 + 2.40 + 22.0 = 25.52 \frac{K}{FT}$$

∴ TOTAL PASSIVE RESISTANCE AVAILABLE ALONG AB

$= 180 + 25.52 = 205.52$ K/LF

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 27
05996.62	G(B)	13-6		

(2) ESTIMATE ADDITIONAL RESISTANCE TO SLIDING AVAILABLE AT THE ENDS OF THE BLOCK OF SILTY CLAY THAT MUST SHEARED BEFORE THE CTB CAN SLIDE. INCLUDE ONLY THE PORTION BELOW THE CTB MAT; i.e., BCDE SHOWN ON PAGE 19.

$S_u = 2.2$ KSF, = MINIMUM S_u MEASURED IN UU TRIAXIAL TESTS AT $\sigma_c = 1.3$ KSF

AREA BCDE $= 6\text{ FT} \times 240\text{ FT}_{E\text{-}W} = 1440 \dfrac{\text{FT}^2}{\text{END}}$

$\therefore \Delta T_{ENDS_{E\text{-}W}} = 2\text{ ENDS} \times 1440 \dfrac{\text{FT}^2}{\text{END}} \times 2.2 \dfrac{K}{\text{FT}^2} = 6{,}336\ K_{E\text{-}W}$

$\Delta T_{END_{N\text{-}S}} = 2\text{ ENDS} \times 6' \times \underset{L_{N\text{-}S}}{279.5'} \times 2.2 \dfrac{K}{\text{FT}^2} = 7{,}379\ K_{N\text{-}S}$

(3) FRICTIONAL RESISTANCE ALONG PLANE BC:

ADD WEIGHT OF SILTY CLAY BLOCK BETWEEN BOTTOM OF MAT & TOP OF SILTY SAND / SANDY SILT TO THE NORMAL FORCE AT BOTTOM OF THE MAT.

$\Delta N_{CLAY} = \underset{\Delta H}{6'} \times \underset{\gamma}{0.090 \dfrac{K}{\text{FT}^3}} \times \underset{B}{240'} \times \underset{L}{279.5'} = 36{,}223\ K$

(4) SINCE THE MATERIAL FROM GROUND SURFACE TO THE TOP COHESIONLESS SILTY SAND/SANDY SILT ARE ALL COHESIVE (SOIL CEMENT, SILTY CLAY), THE ACTIVE EARTHQUAKE PRESSURE IS SMALL AND NEGLIGIBLE.

NOTE: FRICTIONAL RESISTANCE WILL BE LOWER WHEN VERT EARTHQUAKE FORCES ACT UPWARD. $\therefore$ CHECK CASES III A, B & C

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

NOTED JAN 2 1 2000 P. I. Tradeau

▲5010.65

3 CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 28
05996.02	G(B)	13-6		

SLIDING ON DEEP PLANE

	N-S	VERT	E-W
CASE IIIA:	40% IN X	−100% IN Y	40% IN Z
FROM TABLE 1	0.4×111,108^K	−79,779K	$\underbrace{0.4 \times 99,997}_{L = V_{E-W}}$ = 39,999K

$$\therefore N = \overset{\text{CTB DL}}{97,749} - \overset{F_{VD} \text{ or } EQ_V}{79,779^K} + \overset{\Delta N_{CLAY}}{36,223^K} = 54,193\,K$$

$$N \tan\phi = 54,193^K \tan 38° = 42,340\,K$$

$$\therefore FS_{SLIDING_{N-S}} = \frac{205.52^K/_{LF} \times \underset{\sigma_{E-W}}{240'} + 7,379^K + 42,340^K}{0.4 \times 111,108\,K} = 1.78$$

$$FS_{SLIDING_{E-W}} = \frac{205.52\frac{K}{LF} \times \overset{L}{279.5} + 6336^K + 42,340\,K}{0.4 \times 99,997\,K} = 1.65 > 1.1$$

$\therefore$ OK

	N-S	VERT	E-W
CASE IIIB	40% IN X	−40% IN Y	100% IN Z
FROM TABLE 1	0.4×111,108^K	−0.4×79,779K	99,997^K
			L = V_{E-W}

$$\therefore N = \overset{\text{CTB DL}}{97,749\,K} - \overset{F_{VD}}{0.4 \times 79,779K} + \overset{\Delta N_{CLAY}}{36,223^K} = 102,060\,K$$

$$\Rightarrow T = \underbrace{\left(\overset{P_P}{180\frac{K}{L.F.}} + 25.52\frac{K}{L.F.}\right) \times \overset{L_{N-S}}{279.5'}}_{57,443\ K.} + 6,336\,K_{E-W}$$

$$+ \underbrace{\overset{N}{102,060}\,K \tan \overset{\phi}{38°}}_{79,738} = 143,517\,K$$

$$FS = \frac{\text{RESISTING}}{\text{DRIVING}} = \frac{143,517^K}{99,997\,K} = 1.44 > 1.1$$

$\therefore$ O.K

NOTED JAN 2 1 2000 P. J. Trudeau

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

▲ 5010.65

4 CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO. 05996.02	DIVISION & GROUP G(B)	CALCULATION NO. 13-6	OPTIONAL TASK CODE	PAGE 29

SLIDING ON DEEP PLANE

	N-S	VERT	E-W
<u>CASE III C</u>	100% IN X	−40% IN Y	40% IN Z
FROM TABLE 1	111,108 K	−0.4 × 79,779 K	0.4 × 99,997

$$\therefore N = \underset{CTB+LL}{97,749} - \underset{31,912}{\underset{F_{VD}}{0.4 \times 79,779^{K}}} + \underset{\Delta N_{CLAY}}{36,223} = 102,060\,K$$

$$T_{N\text{-}S} = 205.52\,\frac{K}{LF} \times \underset{B}{240'} + \underset{\Delta T_{N\text{-}S}}{7,379^{K}} + 102,060 \tan 38^{\circ} = 136,442\,K$$

$$FS_{SLIDING} = \frac{T}{V_{N\text{-}S}} = \frac{136,442^{K}}{111,108\,K} = 1.23 \quad > 1.1 \;\therefore OK$$

THE FACTOR OF SAFETY AGAINST SLIDING ON A DEEP PLANE OF COHESIONLESS SOIL IS > 1.1 FOR LOAD CASES III A, III B, & III C. THEREFORE THERE IS NO SLIDING ON A DEEP PLANE OF COHESIONLESS SOIL.

Table 2.6-13

Sliding Stability of Canister Transfer Building Using Shear Strength Along Bottom of Plane Formed by 1.5-ft Deep Perimeter Key and Half of Resistance from Soil Cement Using Peak Strength of Clay

Joint	MASS X $k\text{-}sec^2/ft$	MASS Y $k\text{-}sec^2/ft$	MASS Z $k\text{-}sec^2/ft$	N-S a_x g	Vert a_y g	E-W a_z g	Static F_v k	Earthquake $Shear_{N-S}$ k	Earthquake F_v k	Earthquake $Shear_{E-W}$ k
0	260.1	260.1	260.1	1.047	0.783	0.920	8,368	8,761	6,551	7,699
1	1,908.0	1,908.0	1,908.0	1.047	0.783	0.920	61,380	64,265	48,055	56,470
2	420.4	420.4	420.4	1.111	0.821	0.994	13,524	15,023	11,106	13,446
3	304.3	304.3	170.3	1.778	0.913	1.185	9,789	17,402	8,939	6,493
4	144.7	117.1	144.7	1.215	0.928	1.408	3,767	5,656	3,495	6,554
5	1.0	27.6	1.0	0.000	1.840	0.000	888	0	1,634	0
6	1.0	1.0	134.0	0.000	0.000	2.166	32	0	0	9,336
CTB Mat Dimensions:			**B =**	**240.0**	**ft (E-W)**	***Totals =***	**97,749**	**111,108**	**79,779**	**99,997**
Depth =	**5**	**ft**	**L =**	**279.5**	**ft (N-S)**			***Resisting***	***Driving***	

For ϕ =		**0.0**	**degrees**	**c =**	**1.70**	**N (k)**	**T (k)**	**V (k)**	**FS**
Earthquake Vertical Forces Acting Up	**IIIA**	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	100% $F_{v(Eqk)}$ -79,779	40% $F_{H(EW)}$ 39,999	17,970	135,999	59,792	**2.27**
	IIIB	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	40% $F_{v(Eqk)}$ -31,912	100% $F_{H(EW)}$ 99,997	65,837	135,999	109,429	**1.24**
	IIIC	$F_{v(Static)}$ 97,749	100% $F_{H(NS)}$ 111,108	40% $F_{v(Eqk)}$ -31,912	40% $F_{H(EW)}$ 39,999	65,837	135,999	118,088	**1.15**
Earthquake Vertical Forces Acting Down	**IVA**	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	100% $F_{v(Eqk)}$ 79,779	40% $F_{H(EW)}$ 39,999	177,529	135,999	59,792	**2.27**
	IVB	$F_{v(Static)}$ 97,749	40% $F_{H(NS)}$ 44,443	40% $F_{v(Eqk)}$ 31,912	100% $F_{H(EW)}$ 99,997	129,661	135,999	109,429	**1.24**
	IVC	$F_{v(Static)}$ 97,749	100% $F_{H(NS)}$ 111,108	40% $F_{v(Eqk)}$ 31,912	40% $F_{H(EW)}$ 39,999	129,661	135,999	118,088	**1.15**
Soil Cement ΔF_H for q_u (psi) = 250			21,600	N/A	25,155	**for FS_{SC} = 2.0**			

here is the first resume you did for me. If you get a chance, will you
update it for me, and then pass it out!!
I am currently an administrative assistant. I answer the phone, do expense
reports in excel, make travel arrangements, keep the office calendar for
entire staff of 21 people.(meetings, appt. travel) and I also set of
meetings for 4 partners, including the president. I sort and distribute
mail, assembly company brochures, meet and greet clients and host in house
conferences. (provide breafast, lunch etc., materials)
I have basic knowledge of word, outlook and excel.

>From: "Sharon" <Sharonl@olg.com>
>To: "Rena Johnson" <hearditb4@hotmail.com>
>Subject: RE:
>Date: Thu, 30 Nov 2000 22:48:00 -0500
>
>Attach is a copy of your resume!!
>
>-----Original Message-----
>From: Rena Johnson [mailto:hearditb4@hotmail.com]
>Sent: Thursday, November 30, 2000 3:48 PM
>To: sharonl@olg.com
>Subject:
>
>
>Company Name: Technology Strategies + Alliances
>5242 Lyngate Court, Burke Va 22015
>703-425-1210 Fax 703-425-8839 Attn: Vicky Laperle
>
>Work Experience:
>
>1976-1983 FBI: I did background checks for security clearances for various
>government agencies. at the time, I held a top secret clearance
>
>1983-1985 Temporaries
>Receptionist, Customer Service
>
>1985-1989- Homemaker
>
>1989-1991 Talent Tree
>Receptionist/Amindistrative Assitant
>
>1991-1995 Interstate Vanlines (ADS 1001 service devision)
>customer service/ data entry for Washington Nat'l and Dulles Airports
>
>1995-Present Compex Corporation
>Administrative Assistant/Receptionist
>Currently assisting at front desk, but duties have included timesheet entry
>in Deltek. Travel scheduling, filing, phone system changes, market research
>for new business ventures, new hire packages, assisting in HR
>
>References:
>Dave Smith, Comptroller 703-642-5910
>Tammy Kitchen, Payroll Administrator 703-642-5910
>Debra Gibbs, Fairfax Co Social Services 703-838-0715
>
>Rena Johnson

TRUDEAU SECOND DECLARATION

EXHIBIT 3

Calculation No. 05996.02-G(B)-04, Rev. 4

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

▲ 5010.64 (FRONT)

CLIENT & PROJECT	PAGE 1 OF 55 + 9 INSERTS + Attachments (9 pp.)
PRIVATE FUEL STORAGE, LLC - PRIVATE FUEL STORAGE FACILITY	
CALCULATION TITLE (Indicative of the Objective): STABILITY ANALYSES OF STORAGE PADS	QA CATEGORY (✓) ☑ I – NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ____ OTHER

CALCULATION IDENTIFICATION NUMBER

J.O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.01	G(B)	04		119

* APPROVALS – SIGNATURE & DATE						
PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES * CALC. NO. OR REV. NO.	CONFIRMATION *REQUIRED (✓) YES	NO
TE SPONSELLER 2-18-97 Tom Sponseller; PJ TRUDEAU 2-24-97 Paul J. Trudeau	PAUL J. TRUDEAU 2-24-97 Paul J. Trudeau; TE SPONSELLER 2-24-97 Tom Sponseller	NURI T. GEORGES 2-27-97 Nuri T. Georges	0		✓	
T.E. SPONSELLER 4/30/97 Tom Sponseller; PAUL J. TRUDEAU 4/30/97 Paul J. Trudeau	PAUL J. TRUDEAU 4/30/97 Paul J. Trudeau; TE SPONSELLER 4/30/97 Tom Sponseller	Alan F Brown 5/8/97 Alan Brown	1	0		✓
PAUL J. TRUDEAU 6/20/97 Paul J. Trudeau	NURI T. GEORGES 6/20/97 Nuri T. Georges	ALAN F BROWN 6/20/97 Alan Brown	2 CONTINUED ON p 1A	1		✓ 1A

DISTRIBUTION*

GROUP	NAME & LOCATION	COPY SENT (✓)	GROUP	NAME & LOCATION	COPY SENT (✓)
RECORDS MGT. FILES (OR FIRE FILE IF NONE)	SWEC-DENVER	ORIG			
	SWEC-BOSTON	X			
	SWEC-DENVER FIRE FILE	X			

~~00844~~

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

▲ 5010.64 (FRONT)

CLIENT & PROJECT	PAGE 1A OF 55+9 INSERTS +Attachments (9pp)
PRIVATE FUEL STORAGE, LLC - PRIVATE FUEL STORAGE FACILITY	
CALCULATION TITLE (Indicative of the Objective): STABILITY ANALYSES OF STORAGE PADS	QA CATEGORY (✓) ☑ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ____ OTHER

CALCULATION IDENTIFICATION NUMBER				
J. O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.01	G(B)	04		

* APPROVALS - SIGNATURE & DATE			REV. NO. OR NEW CALC NO.	SUPERSEDES * CALC. NO. OR REV. NO.	CONFIRMATION *REQUIRED (✓)	
PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)			YES	NO
—— CONTINUED FROM PAGE 1 ——						
PAUL J. TRUDEAU 6/27/97 Paul J. Trudeau	LP SINGH [signature] 7-1-97	LP SINGH [signature] 7-1-97	3	2		✓
ΔL ALOYSIUS [signature] S. Y. BOAKYE 9/3/99 [signature]	S. Y. BOAKYE 9/3/99 [signature] ΔL ALOYSIUS [signature]	T. Y. Chang 9/3/99 Thomas Y. Chang T. Y. Chang 9/3/99 Thomas Y. Chang	4	3	✓	
SEE PAGE 3 FOR IDENTIFICATION OF PREPARED/REVIEWED BY						

DISTRIBUTION*

GROUP	NAME & LOCATION	COPY SENT (✓)	GROUP	NAME & LOCATION	COPY SENT (✓)
RECORDS MGT. FILES (OR FIRE FILE IF NONE)	SWEC - DENVER	ORIG			
	SWEC - BOSTON	X			
	SWEC - DENVER FIRE FILE	X			

~~008 45~~

STONE & WEBSTER ENGINEERING CORPORATION

5010.65

CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER				PAGE 2
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 4	N/A	

TABLE OF CONTENTS

TITLE PAGE 1, 1A

TABLE OF CONTENTS 2

RECORD OF REVISIONS 3

OBJECTIVE OF CALCULATION 4

DATA/ASSUMPTIONS 4

FOOTING PLAN AND PROFILE 6

DETAIL OF FOOTING CONFIGURATIONS 7

GEOTECHNICAL PROPERTIES 8

STATIC BEARING CAPACITY TOTAL STRESS ANALYSIS 9

STATIC BEARING CAPACITY EFFECTIVE STRESS ANALYSIS 10

SEISMIC ANALYSIS: CASE 1 - SEISMIC SLIDING RESISTANCE ANALYSIS .... 11-14 + 14A To F

SEISMIC ANALYSIS: CASE 2 - BEARING CAPACITY (w/ EQv and EQh) 15

SEISMIC ANALYSIS: CASE 3 - BEARING CAPACITY (w/ EQh) 18

DYNAMIC BEARING CAPACITY OF PAD: 2 CASK CASE 20

DYNAMIC BEARING CAPACITY OF PAD: 4 CASK CASE 28

DYNAMIC BEARING CAPACITY OF PAD: 8 CASK CASE 38

DYNAMIC BEARING CAPACITY OF PAD: SUMMARY 46, 46A, 46B

CONCLUSIONS 48

REFERENCES 49

TABLES 50

FIGURES 53

<u>ATTACHMENTS</u> — <u>No. of Pages</u>

A: Telecon 6-19-97 between SMMacie and WTseng and PJTrudeau 1

B: pp. 174-181 of ICEC calc. SC(PO17)-1, Rev 0, "Storage Pad Analysis and Design" 8

RECORD OF REVISIONS

REVISION 0
Original Issue

REVISION 1
Revision 1 was prepared to incorporate the following:

- Revised cask weights and dimensions
- Revised earthquake accelerations
- Determine q_{all} as a function of the coefficient of friction between casks and pad.

REVISION 2
To add determination of dynamic bearing capacity of the pad for the loads and loading cases being analyzed by the pad designer. These include the 2-cask, 4-cask, and 8-cask cases. See Attachment A for background information as well as bearing pressures for the 2-cask loading.

REVISION 3
The bearing pressures and the horizontal forces due to the design earthquake for the 2-cask case, which are described in Attachment A, are superseded by those included in Attachment B. Revision 3 also adds the calculation of the dynamic bearing capacity of the pad for the 4-cask and 8-cask cases and revises the cask weight to 356.5 K, which is based on Holtec HI-Storm Overpack with loaded MPC-32 (heaviest assembly weight shown on Table 3.2.1 of HI-Storm TSAR, Report HI-951312 Rev. 1 – p. C3, calculation 05996.01-G(B)-05-0).

REVISION 4
Updated section on seismic sliding resistance of pads (pp. 11-14F) using revised ground accelerations (horizontal = 0.528 g; vertical = 0.533 g) and revised soil parameters (c = 1220 psf; ϕ = 24.9 °). The driving forces used in this analysis (EQhc and EQhp) are based on higher ground accelerations (0.67g horizontal and 0.69g vertical). These forces were not revised for the lower ground accelerations (0.528g horizontal and 0.533g vertical) and will require confirmation at a later date.

Added a section on sliding resistance along a deeper slip plane (i.e., on cohesionless soils) beneath the pads.

Updated section on dynamic bearing capacity of pad for 8-cask case (pp. 38-46). Inserted pp. 46A and 46B. This case was examined because it previously yielded the lowest q_{all} among the three loading cases (i.e., 2-cask, 4-cask, and 8-cask). The updated section shows a calculation of q_{all} based on revised soil parameters (c and ϕ). Note: this analysis will require confirmation and may be updated using revised vertical soil bearing pressures and horizontal shear forces, based on the lower ground accelerations of 0.528g horizontal, and 0.533g vertical.

Modified/updated conclusions.

NOTE:
SYBoakye prepared/DLAloysius reviewed pp. 14 through 14F.
Remaining pages prepared by DLAloysius and reviewed by SYBoakye.

N (normal force) = $\sum$ Fv = Wc + Wf + EQvc + EQvp
ϕ = 24.9° (for Layer 1 silty clay)
c = 1220 psf (1.22 ksf)
B = 30 feet
L = 64 feet

Minimum sliding resistance exists when EQvc and EQvp act in an upward direction. For upward force:

N = 2852 K + 864 K + (- 1520 K) + (- 461 K) = 1735 K

T = [1735 K x tan (24.9°)] + [1.22 ksf x 30 ft x 64 ft] = 3148 K

The driving force is defined as:

$$F_A + EQhp + EQhc$$

F_A , EQhp, and EQhc have been defined above.

The equation used for calculating factor of safety is as follows:

$$FS = T \div (F_A + EQhp + EQhc)$$

For this analysis, the larger value of EQhc (i.e., acting in the short direction of the pad) was used because it produces a lower and thus, more conservative factor of safety.

$$FS = 3148\ K \div (69.1\ K + 579\ K + 2030\ K) = \mathbf{1.18}$$

The above analysis provides a factor of safety > 1.1, which is a minimum value that is considered to be "safe" against sliding. The driving forces used in this analysis (EQhc and EQhp) are based on higher ground accelerations (0.67g horizontal and 0.69g vertical). These values were not calculated for the lower ground accelerations (0.528g horizontal and 0.533g vertical) considered in this calculation and will require confirmation at a later date. At present, however, it is assumed that these forces will yield what could be considered worst-case factors of safety against sliding.

<u>Analysis 2: Evaluation of Sliding on Deep Slip Surface Beneath the Pads</u>

Adequate factors of safety against sliding have been obtained with the storage pads under the maximum components of earthquake motion. The shearing resistance has come from the undrained shear strength of the clayey silt/silty clay layer which is not much affected by upward acting earthquake loads. A silty sand/sandy silt layer underlies the clayey layer at a depth of about 10 ft. The shearing resistance of this layer is directly related to the normal stress if cementation effects are ignored. Earthquake motions resulting in upward forces reduce the normal stress and the shearing resistance. Factors of safety against sliding in such materials can be low if the maximum components of the ground motion are combined. The effect of such motions are best evaluated by examining the displacements the structure will undergo.

Newmark's method is used to estimate the displacement of the mat foundations assuming they were founded on the sand layer. For motion to occur on a slip surface in the sand layer the slip surface must pass through the overlying clay layer. The simplification therefore results in some conservatism. A friction angle of 30° was used for the sand to allow evaluation of a loose sand layer directly under the mat foundations. The deeper layers of sand are medium dense to dense with a higher friction angle.

The ground motions for the analysis requires confirmation. Accelerations used for the displacement analysis (Rev 0) are higher than the revised accelerations (Rev. 1). Maximum ground velocities were estimated by using the maximum horizontal velocities of the mat in the Canister Transfer Building and scaling it down with the ratio of the maximum accelerations.

Estimation of Horizontal Displacement using Newmark's Method

Maximum Ground Motions

The maximum ground accelerations and velocities at the Storage Pads are as follows:

a_x = 0.67 g (North-South)

a_z = 0.67 g (East-West) Assumed to be equal to the North-South component.

a_y = 0.69 g (Vertical)

Assume maximum ground velocities/acceleration relationships can be approximated by the values from the Canister Transfer Building area (Calculation #05996.02-SC-5 pg 37) in the Table below

Canister Bldg	North-South	Vertical	East-West
Acceleration	0.805g	0.720g	0.769g
Velocity	21.7 in/sec		19.8 in/sec

Velocity in N-S direction = 0.67 x 21.7/0.805 = 18.1 in/sec

Velocity in E-W direction = 0.67 x 19.8/0.769 = 17.3 in/sec

The maximum ground motions for the analysis of the storage pads are as follows:

Storage Pads	North-South	Vertical	East-West
Acceleration	0.67g	0.69g	0.67g

Storage Pads	North-South	Vertical	East-West
Velocity	18.1 in/sec		17.3 in/sec

2. Load Combinations

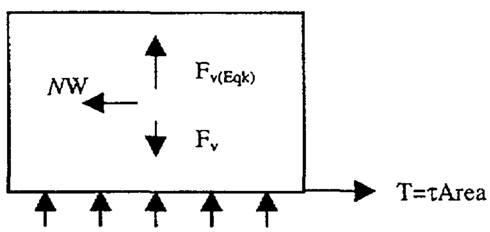
The displacement estimate is made with the maximum earthquake ground motions in the vertical, north-south (N-S), and east-west(E-W) directions using the allowable combination factors of 100% maximum motion in one direction combined with 40% of the maximum motions in the other two directions. The following ground motions result from the three possible combinations.

Load Combination 1: 100% Vertical, 40% N-S, 40% E-W (Load #1)
Load Combination 2: 40% Vertical, 100% N-S, 40% E-W (Load #2)
Load Combination 3: 40% Vertical, 40% N-S, 100% E-W (Load #3)

3. Ground Motions for Analysis

Load #	North-South		Vertical	East-West	
	Accel	Velocity	Accel	Accel	Velocity
1	0.268g	7.24 in/sec	0.690g	0.268g	6.92 in/sec
2	0.670g	18.1 in/sec	0.276g	0.268g	6.92 in/sec
3	0.268g	7.24 in/sec	0.276g	0.670g	17.3 in/sec

4. Determination of *N*



Newmark defines *N*W as the steady force applied at the center of gravity of the sliding mass in the direction which the force can have its lowest value to just overcome the stabilizing forces and keep the mass moving.

For a block sliding on a horizontal surface, *N*W = T

Where T is the shearing resistance of the block on the sliding surface.

Shearing resistance, T = τ x Area

$$\tau = \sigma_n \tan\phi$$

where

$$\sigma_n = \text{Normal Stress}$$

$$\phi = \text{Friction angle of sand layer}$$

$$\sigma_n = (\text{Net Vertical Force})/\text{Area}$$

$$= (F_v - F_{v(Eqk)})/\text{Area}$$

$\therefore$ $$T = (F_v - F_{v(Eqk)}) \tan\phi$$

$$NW = T$$

$\therefore$ $$N = ((F_v - F_{v(Eqk)}) \tan\phi)/W$$

LOAD COMBINATION 1

Static Vertical Force, $F_v = W$ = Weight of casks and pad

Static Vertical Force, $F_v = W = 3716$ kips

Earthquake Vertical Force, $F_{v(Eqk)} = a_y \times W/g$

$$= 0.69 \times 3716$$
$$= 2564 \text{ kips}$$

$$\phi = 30^\circ$$

For load combination 1, 100% of upward earthquake force is applied to obtain net vertical force

$$N = ((3716 - 2564) \tan 30)/3716$$

$$N = 0.179$$

Resultant Acceleration in horizontal direction, $A = (0.268^2 + 0.268^2)^{0.5}$

$$= 0.379$$

Resultant Velocity in horizontal direction, $V = (7.24^2 + 6.92^2)^{0.5}$

$$= 10.02 \text{ in/sec}$$

$$N/A = 0.179/0.379$$

$$= 0.472$$

Maximum relative displacement of building relative to the ground, u_m , from Newmark (Newmark, 1965) is

$$u_m = (V^2 (1 - N/A))/(2gN)$$

$$= 10.02^2 (1 - 0.472)/(2x386.4x0.179)$$

$$= 0.4"$$

The above expression for the relative displacement is an upper bound for all the data points for N/A less than 0.15 and greater than 0.5, Figure 3. Within the range of 0.5 to 0.15 the following expression (Newmark, 1965) gives an upper bound for all data.

$$u_m = V^2/(2gN)$$

Substituting the relevant parameters for load case 1

$$u_m = 10.02^2/(2x386.4x0.179)$$

$$= 0.7"$$

Therefore maximum relative displacement ranges from 0.4" to 0.7"

LOAD COMBINATION 2

Static Vertical Force, $F_v = W = 3716$ kips

Earthquake Vertical Force, $F_{v(Eqk)} = 2564$ kips x 0.40 = 1026 kips

$$\phi = 30^\circ$$

$$N = ((3716 - 1026)\tan 30)/3716$$

$$N = 0.418$$

Resultant Acceleration in horizontal direction, $A = (0.670^2 + 0.268^2)^{0.5}$

$$= 0.722$$

Resultant Velocity in horizontal direction, $V = (18.1^2 + 6.92^2)^{0.5}$

$$= 19.4 \text{ in/sec}$$

$$N/A = 0.418/0.722$$

$= 0.579$

Maximum relative displacement of building relative to the ground, u_m , from Newmark (Newmark, 1965) is

$$u_m = (V^2 (1- N/A))/(2gN)$$

$$= 19.4^2 (1 - 0.579)/ (2x386.4x0.418)$$

$$= 0.5"$$

LOAD COMBINATION 3

Static Vertical Force, $F_v = W = 3716$ kips

Earthquake Vertical Force, $F_{v(Eqk)} = 2564$ kips x 0.40 = 1026 kips

$$\phi = 30^o$$

$$N = ((3716 - 1026) \tan30)/3716$$

$$N = 0.418$$

Resultant Acceleration in horizontal direction, $A = (0.268^2 + 0.670^2)^{0.5}$

$$= 0.722$$

Resultant Velocity in horizontal direction, $V = (7.24^2 + 17.3^2)^{0.5}$

$$= 18.75 \text{ in/sec}$$

$$N/A = 0.418/0.722$$

$$= 0.579$$

Maximum relative displacement of building relative to the ground, u_m , from Newmark (Newmark 1965) is

$$u_m = (V^2 (1- N/A))/(2gN)$$

$$= 18.75^2 (1 - 0.579)/ (2x386.4x0.418)$$

$$= 0.5"$$

Summary

LOAD COMBINATION	DISPLACEMENT
1. 100% Vertical, 40% N-S, 40% E-W	0.4 to 0.7 inches
2. 40% Vertical, 100% N-S, 40% E-W	0.5 inches
3. 40% Vertical, 40% N-S, 100% E-W	0.5 inches

The estimated relative displacement of the Storage Pads ranges from 0.4 inches to 0.7 inches. The higher displacement corresponds to the load combination with the maximum upward earthquake force used to reduce the normal stress and hence the shearing resistance of the sand layer. For the pads to slide a surface of sliding must be established between the horizontal sliding surface in the sand layer and the overlying clayey layer. The contribution of this surface of sliding to the dynamic resistance to sliding motion is ignored in the simplified model used to estimate the relative displacement.

The procedure used to estimate relative displacements has several measures of conservatism and the estimated displacements are most likely to represent upper bound values.

TRUDEAU SECOND DECLARATION

EXHIBIT 4

Letter from HOLTEC to PFS (August 6, 2001)

(Submitted by PFS to the NRC under cover letter dated August 7, 2001)

Private Fuel Storage, L.L.C.

7677 East Berry Ave., Englewood, CO 80111-2137
Phone 303-741-7009 Fax: 303-741-7806
John L. Donnell, P.E., Project Director

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555-0001

August 7, 2001

COMMITMENT RESOLUTION LETTER #37
DOCKET NO. 72-22 / TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

In accordance with our July 31, 2001 conference call, Private Fuel Storage (PFS) submits the following resolution to NRC/CNWRA questions and comments regarding the stability analysis for the cask storage pads.

NRC Question/Comment

PFS should provide a basis for the conclusions contained within the SAR that the storage casks do not tip over, collide, nor slide off the storage pad during the seismic event, taking into consideration the potential movement of the cask storage pads of up to 6".

PFS Response

A formal evaluation has been performed for PFS by Holtec International to assess the impact of potential movement of the cask storage pads during a seismic event on the PFS Site Specific HI-STORM Drop/Tipover Analyses, (Holtec Report No. HI-2012653, Revision 1, dated May 7, 2001). The Holtec evaluation is attached for your use.

The results of the evaluation demonstrate that the current conclusions reached in the PFSF Safety Analysis Report remain valid and are bounding for the response of the casks relative to the pad.

If you have any questions regarding this response, please contact me at 303-741-7009.

Sincerely,

JL Cooper for

John L. Donnell
Project Director
Private Fuel Storage L.L.C.

Enclosure

Copy to:
Mark Delligatti-1/1
John Parkyn-1/1
Jay Silberg-1/1
Sherwin Turk-1/1
Asadul Chowdhury-1/1
Scott Northard-1/1
Denise Chancellor-1/1
Richard E. Condit-1/1
John Paul Kennedy-1/1
Joro Walker-1/1
Duncan Steadman-1/1
Utah Document file (D. Bird)-1/1

HOLTEC
INTERNATIONAL

Holtec Center, 555 Lincoln Drive West, Marlton, NJ 08053
Telephone (856) 797-0900
Fax (856) 797-0909

August 6, 2001

Dr. Max DeLong
Executive Engineer
Xcel Energy
414 Nicollet Mall (RS-7)
Minneapolis, MN 55401

Reference: Holtec Project 70651

Dear Dr. DeLong:

In response to your request, we herewith provide the additional information related to the recent site-specific ISFSI pad sliding evaluations performed for Private Fuel Storage (PFS).

SCOPE:

Holtec International has previously performed a series of dynamic simulations of a PFSF ISFSI pad supporting from one to eight spent fuel storage casks and subject to various seismic excitations; these analyses were performed in support of the PFSF site-specific ISFSI licensing submittal. Using design input supplied by PFSF, soil-springs were included in the dynamic model to simulate the effect of the foundation between the base of the ISFSI pad and the top of competent rock driven by the design basis seismic excitation. In the previous Holtec analyses, no separation of the soil from the ISFSI pad lower surface, nor any relative motion (sliding) between the base of the ISFSI pad and the soil surface was assumed. Recent hypothetical bounding analysis (by others) has concluded that postulating loss of surface cohesion could result in as much as six inches of relative displacement of the pad with respect to the soil surface. Therefore, the effect of such relative movement on the response of the casks requires attention. In this letter report, Holtec provides the information needed to conclude that this potential sliding of the ISFSI pad relative to the underlying soil foundation has no significant effect on the conclusions based on the previous dynamic simulations that assumed no sliding.

DISCUSSION:

The loss of cohesion leading to pad movement, relative to the top layer of the soil, is well represented by assuming frictional behavior at the pad/soil interface. Therefore, at some limiting value of horizontal force, the pad begins to move, relative to the soil, and this movement may affect the response of the casks, relative to the pad. Whether the effect on the cask response is detrimental or beneficial is the subject of this letter report.

HOLTEC INTERNATIONAL

Holtec Center, 555 Lincoln Drive West, Marlton, NJ 08053

Telephone (856) 797-0900
Fax (856) 797-0909

We note that the simulation responses to date effectively assume an infinite value for the coefficient of friction between the pad and the soil as the horizontal soil resistance is modeled as a linear spring-damper that is always fully effective. The results from the various simulations predicted minimal movement of the pad and a combination of tipping and sliding of the casks relative to the pad (dependent upon the cask/pad coefficient of friction used). To address the issue at hand, we note that if we postulate the other extreme limit for the pad/soil coefficient of friction, namely zero, then the pad/cask system is fully isolated from the input seismic excitation and the casks experience no motion (either sliding or tipping) relative to the pad. The pad, however, experiences maximum relative movement relative to the soil. Based on this simple physical argument, we are led to the conclusion that any sliding of the pad relative to the soil serves to decrease the energy input to the casks and therefore decreases the motion of the casks relative to the pad. If our argument is valid, then the current FSAR statement (repeated below for completeness) remains valid and supplies bounding values for the response of the casks, relative to the pad.

> "In addition, the vendor performed a site specific analysis for HI-STORM storage casks subjected to the design basis ground motion associated with the probabilistic seismic hazard analysis with the 2,000-yr return period (0.711g horizontal, 0.695g vertical), and determined maximum displacement of the cask of less than 4 inches (Reference 61). The analyses concluded that the casks do not tip over, collide, nor slide off the storage pad for these earthquakes. Soil-structure interaction was considered in the site-specific analyses. The seismic cask stability analyses are fully described in Section 8.2.1."

Although the qualitative argument presented above is convincing in its simplicity, it must be backed by equally convincing confirmatory analyses. A series of dynamic simulations have been performed to confirm the applicability and correctness of the heuristic argument presented previously. Based on these confirmatory results, we conclude that the FSAR statements remain valid as they served to quantify the cask movements relative to the pad.

CONFIRMATORY ANALYSES:

The dynamic simulation model used in all previous submittals on this matter is capable of simulating linear or non-linear behavior across and interface; specifically, the resisting normal force and in-plane forces at the pad/soil interface may be represented by linear springs or by a compression-only normal spring and two orthogonal friction springs. The

HOLTEC INTERNATIONAL

Holtec Center, 555 Lincoln Drive West, Marlton, NJ 08053

Telephone (856) 797-0900
Fax (856) 797-0909

characteristic of each set of two friction springs (FY1, FY2) associated with a compression only normal spring (FW) is as follows:

Let $FH = (FY1^2+FY2^2)^{1/2}$

Then, if the computed value of $FH < \mu\, FW$, the springs FY1 and FY2 behave as simple linear elements at this instant in time with a stiffness and damping associated with the soil.

If the computed value of FH exceeds $\mu\, FW$, then the computed values of FY1 and FY2 are limited to the values that maintain $FH = \mu\, FW$ for the next time step.

Three dynamic analyses were performed using the Holtec QA validated simulation code DYNAMO to evaluate the effect of pad/soil relative motion. These analyses were performed using the following model parameters:

Pad/soil coefficient of friction = 0.306

Seismic input time histories – Latest 2000 Year Return Seismic Event

Cask/pad coefficient of friction = 0.8

Number of casks on ISFSI pad = 8 (2 x 4) array

The three analyses differ in only one aspect; the magnitude of the soil damping associated with the non-linear elements representing normal and in-plane resistance from the soil. For case 1, we assume that the previously computed values for soil resistance due to damping were maintained. For case 2, we assume that the soil damping forces are reduced to 10% of the values used in case 1. Finally, for case 3, we assume that the soil damping forces are reduced to 1% of the values used in case 1. The cases using reduced damping reflect the reality that the damping forces are not active while slip is occurring so that the net effect of the structural damping over the duration of the event must be reduced. The following table summarizes the results obtained for pad center in-plane movement.

HOLTEC INTERNATIONAL

Holtec Center, 555 Lincoln Drive West, Marlton, NJ 08053
Telephone (856) 797-0900
Fax (856) 797-0909

CASE	% OF SOIL DAMPING VALUE PREVIOUSLY USED IN LINEAR ANALYSES	MAX. PAD MOVEMENT (inch) N-S	MAX. PAD MOVEMENT (inch) E-W
1	100	0.537	0.537
2	10	3.989	2.692
3	1	8.808	5.178

As expected, the amount of pad sliding, as a rigid body is a strong function of the level of soil damping assumed to continuously act over the entire duration of the seismic event. Note that cases 2 and 3 bound from either side, the 6" result obtained from a static equivalent analysis using the 100%-40%-40% combination rule.

The results for cask movement relative to the pad from each of the simulations confirmed the initial assertion that as more pad/soil sliding occurred, the cask/pad relative movements decreased and the propensity for cask overturning was nonexistent. For example, for case 2, the maximum cask excursions, relative to the pad, did not exceed 0.02" at the top or bottom of the cask; i.e., even though the cask/pad coefficient of friction was 0.8, the "redirection" of the input energy to moving the pad sufficed to eliminate all overturning cask motion.

Based on the confirming dynamic simulations, we conclude that the initial simulations of the soil/pad interface with linear springs results in the largest values for cask motion relative to the pad; any sliding of the pad relative to the underlying soil due to reduced cohesion has the beneficial effect of reducing or elimination cask movements relative to the pad.

Sincerely,

Brian Gutherman, P.E.
Project Manager

Document ID: 70651014

Emcc: J. Cooper, SWEC

TRUDEAU SECOND DECLARATION

EXHIBIT 5

Letter from Stone & Webster Assistant Project Manager to PFS Board Chairman (January 11, 2001)

Stone & Webster A Shaw Group Company
Founded 1889

Mr. John Parkyn
Board Chairman
Private Fuel Storage L. L. C.
P. O. Box C4010
La Crosse, WI 54602-4010

January 11, 2001

SWEC J.O. No. 05996.02
Letter No. S-M-147
File No. M2.2

CHANGE ORDER #111: SOIL CEMENT LABORATORY TESTING PROGRAM – VENDOR COSTS
PRIVATE FUEL STORAGE FACILITY

Reference: Stone & Webster letter, Cooper to Parkyn, Change Order #96: Step 4 Base Scope Additions– Revision 1, dated November 15, 2000

The above reference letter included Stone & Webster's (S&W) effort required to develop the soil cement mix design. This effort included, among other things, preparation of an ESSOW to obtain soil samples from the site, preparation of a laboratory testing ESSOW, evaluation of bids and placement of a purchase order. It did not include the vendor costs associated with collecting the soil samples and performing the laboratory analysis.

Soil Cement Laboratory Testing
We have prepared an ESSOW and solicited proposals from 7 vendors for the soil cement laboratory testing. Proposals were received from 4 vendors: [redacted] All the proposals were evaluated and were found to be technically acceptable. [redacted] We therefore recommend awarding this work [redacted] This work is considered QA Category I and the vendor will perform all work per the S&W QA program.

[redacted]

Stone & Webster, Inc.
7677 East Berry Avenue
Englewood, Colorado 80111-2137
Phone: 303.741.7700
Fax: 303.741.7670 or 303.741.7671

[redacted]

If you have any questions or comments, or would like to discuss this proposal further, please call me at 303-741-7139.

Sincerely,

J L Cooper

Jerry L. Cooper
Asst. Project Manager

COPY
STONE & WEBSTER ENGINEERING CORPORATION

JLDonnell
JLCooper
PJTrudeau
CKruger
Cfile R2.1.1
Jb Bk M2.2
S03363.doc