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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
	)	

APPLICANT'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

I. INTRODUCTION

Applicant Private Fuel Storage L.L.C. ("Applicant" or "PFS") hereby responds to the State of Utah's ("State") "Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to Further Revised Calculations from the Applicant," filed June 19, 2001 ("Request"). The State asserts that the revised calculations filed by Applicant on May 31, 2001 require the modification of the bases of proposed Contention Utah QQ ("Proposed Utah QQ") because the revised calculations "inaccurately conclude there are adequate factors of safety against sliding." Request at 2. The State further alleges, with respect to the sliding analyses of the storage cask pads, that the newly revised calculations have four deficiencies: (1) the inertial forces acting on the pads are not properly taken into account in the analyses; (2) the calculations treat the buttressing effect of cement-treated soil inconsistently; (3) PFS has failed to address impacts that affect the adhesive strengths of various foundation interfaces; and (4) PFS has not conducted an adequate longitudinal analysis of the storage pads. Id. at 3. With respect to the Canister Transfer Building ("CTB"), the State does not assert any "new" bases, but merely alleges that "PFS still persists in assuming that the passive resistance of cement-treated soil around the CTB is avail-

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able to resist sliding." Id. at 8. Thus, the Request raises no matters with respect to the CTB that need to be addressed herein.[1]

The modifications that the State seeks to make to Proposed Utah QQ, however, do not constitute admissible new contentions or bases therefor. The "new" claims propounded by the State are in many cases not new, since they are merely restatements of the allegations made in Proposed Utah QQ, and are therefore inadmissible for the reasons stated in Applicant's opposition to the admission of Proposed Utah QQ.[2] In a few other cases, the claims are "new" in the sense that they have not been raised before by the State. However, such claims do not meet the requirements for late-filed contentions, since they could have been asserted in a timely manner years ago and, in any event, could have been raised as part of Proposed Utah QQ, so their lateness is compounded twofold without any justification.[3]

Accordingly, the State's 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 documentation filed with LA 8 included specifications for the soil cement, including the use of American Concrete Institute

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1 At one point, the Request states that "PFS has failed to demonstrate that the shallow foundation system of the ... CTB will support the inertial loads for the design basis ground motions at the ISFSI site." Request at 2. However, nowhere in the Request or in the attached Declaration of Dr. Stephen F. Bartlett ("Bartlett Declaration") is there any support for this statement. Accordingly, there is no factual basis to support this bald assertion and it must be rejected as being inadmissible as a contention. See, e.g., Private Fuel Storage, L.L.C. (Independent Fuel Storage Installation), LPB-98-7, 47 NRC 142, 180-81 (1998); Georgia Institute of Technology (Georgia Tech Research Center, Atlanta, Georgia), LBP-95-6, 41 NRC 281, 306 (1995), vacated in part and remanded on other grounds, CLI-95-10, 42 NRC 1, aff'd in part, CLI-95-12, 42 NRC 111 (1995).

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

3 As discussed below and in the enclosed Declaration of Paul J. Trudeau ("Trudeau Declaration") the attempts by the State to justify its belated attempt to modify Proposed Utah QQ on the grounds of alleged changes in the sliding resistance calculation for the storage pads are unavailing, since there have been no material changes in that calculation. The revised calculations filed on May 31, 2001 introduced no new or different analyses from those filed on March 30, 2001, but only provided greater detail regarding the identification of the potential failure modes, failure planes, and shear strength of the pads. See Trudeau Declaration ¶ 6.

("ACI") standards to govern the procedures for placement and treatment of the soil cement.[4] The calculations for the sliding stability of the cask storage pads under seismic loads were also revised to incorporate the effects of the use of soil cement. License Amendment No. 9, submitted on February 2, 2000.[5]

On June 23, 2000, Applicant submitted LA Amendment No. 13 ("LA 13"), which revised additional seismic design calculations to take into account the effects of soil cement in relation to the stability and function of soil cement as a "dynamic buttress."[6] In terms of methodology relevant to the State's Request, the revised calculations submitted with LA 13 continued to apply the same assumptions and methods as the previous versions of the calculations.[7]

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.[8] 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."[9]

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

[5] See Exhibit A to Applicant's Response to Proposed Utah QQ, item 1.

[6] 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 Applicant's Response to Proposed Utah QQ, item 12.

[7] 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 CTB. The two calculations employ the same methodology in areas that were challenged by the State in Proposed Utah QQ. See Attachments 1 and 2 to Exhibit A to Applicant's Response to Proposed Utah QQ.

[8] 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").

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

On May 7, 2001, the NRC Staff advised that it could not determine a schedule for completion of its review of LA 22 until certain information was provided including, *inter alia*, the following:

> 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. [10]

On May 16, 2001, the State filed its request to admit Proposed Utah QQ. On May 31, 2001, Applicant filed its Response to Proposed Utah QQ. On the same date, Applicant 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 as to the potential failure modes, failure planes, shear strength available to resist sliding along those planes, and also provided calculations of the factors of safety against sliding along those planes. Trudeau Declaration ¶ 6. However, the underlying analyses and assumptions have remained unchanged between these and the previous revisions of the two calculations. Id.[11]

On June 19, 2001, the State filed its Request to modify the bases of Proposed Utah QQ. On June 20, 2001, the Board issued an Order setting July 3, 2001 as the date for filing responsive pleadings to the State's Request. The instant Response is filed pursuant to the Board's Order.

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[10] Attachment, "Data Needed for the Completion of the PFS LA Amendment", to letter dated May 7, 2001 from E. William Brach (NRC) to John D. Parkyn (PFS), "Soil Engineering" section, item 3.

[11] As the Commission has ruled in the past, Boards should have available for their review the documents cited by intervenors so they can assess on their own the accuracy of the characterization of those documents in proposed contentions. See Vermont Yankee Nuclear Power Corporation (Vermont Yankee Nuclear Power Station), ALAB-919, 30 NRC 29, 48 (1989); vacated in part on other ground and remanded, CLI-90-4, 31 NRC 333 (1990); see also Yankee Atomic Electric Company (Yankee Nuclear Power Station), LBP-96-2, 43 NRC 61, 90 (1996). The State has failed in its Request to provide copies of the sliding resistance calculations. Applicant is therefore providing as Exhibits to Mr. Trudeau's Declaration copies of the relevant excerpts of those calculations. Those exhibits show that the calculations are essentially equivalent in all significant respects in going from one revision to the next, and arrive at the same critical results. See, e.g., Trudeau Declaration ¶ 7.

III. LEGAL STANDARDS

Pursuant to 10 C.F.R. § 2.714(b)(1), late-filed contentions are admissible only if a balancing of 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).

For the reasons discussed below, the State has failed to show good cause for its late filing.[12] 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). Private Fuel Storage, L.L.C., supra, LBP-98-7, 47 NRC at 208. In the present instance, like in the case of Proposed Utah QQ, the State has not made a compelling showing on the other four factors to compensate for its extreme lateness. Therefore, the Request fails to satisfy the admissibility standards of 10 C.F.R. § 2.714(a)(1).

IV. APPLICATION OF LEGAL STANDARDS TO CLAIMS RAISED IN REQUEST TO MODIFY UTAH QQ SHOWS THAT THE MODIFICATION IS NOT ADMISSIBLE

A. UNTIMELINESS AND REPETITIVENESS OF STORAGE CASK PAD CLAIMS

In the Request for modification of Proposed Utah QQ, the State has asserted four "new" challenges to the sliding resistance calculation for the storage pads (Cask Storage Pad Stability Calc. Rev. 8). As discussed earlier, these challenges are: (1) the inertial forces acting on the pads are not properly included in the analysis; (2) the calculation treats the buttressing effect of ce-

[12] When, like here, the sole alleged justification for the late filing of a contention is the issuance of a licensing-related document, lateness is only justified if the information in the document was not previously publicly available. Duke Power Co. (Catawba Nuclear Station, Units 1 and 2), CLI-83-019, 17 NRC 1041, 1045 (1983). As discussed below, the information contained in the revised calculations cited by the State has been available previously for months or years, and therefore the issuance of the revised calculations provides no excuse for the State's lateness.

ment-treated soil inconsistently; (3) PFS has failed to address impacts that affect the adhesive strengths of various foundation interfaces; and (4) PFS has not conducted an adequate longitudinal analysis of the storage pads. None of these challenges amount to an admissible contention.

1. Inertial Forces Acting on Storage Cask Pads

The State challenges PFS's assumption, in one of its sliding resistance analyses for the storage cask pads (in which the casks are assumed to rest on native soil and no credit is taken for the effects of the soil cement), that the cement pad and the cement-treated soil will act as an integral block and transfer all inertial forces to the top of the native soil, because PFS has not incorporated the mass of the pad and the underlying cement-treated soil into the calculation. Request at 4. However, this analysis has remained unchanged since the filing of LA 8 in 1999, in which PFS included soil cement as part of the design of the storage cask and pad system.[13] Trudeau Declaration ¶ 10. Thus, this contention could have been raised then and is impermissibly late now.

The State, however, asserts that there was no reason to raise this issue in relation to prior revisions of the analysis, including Revision 7, because its use there was "hypothetical." Request at 5. However, there is nothing hypothetical about this methodology, which is utilized consistently in both Revisions 7 and 8 to compute the minimum factor of safety against sliding, which is conservatively obtained by taking no credit for the presence of soil cement. Trudeau Declaration ¶¶ 7, 13.

2. PFS Has Treated the Buttressing Effects of Soil Cement Consistently and Revision 8 of the Cask Storage Pad Stability Calculation Raises No New Issues Related to this Treatment

The State further asserts, as a basis for its request to modify Proposed Contention QQ, that in Revision 8 to the Cask Storage Pad stability analysis "PFS presents a confusing and inconsistent design approach in considering whether or not the passive resistance provided by ce-

[13] PFS Letter, Parkyn to NRC, License Application Amendment No. 8, dated December 16, 1999, SAR, Rev. 8 at 2.6-84 to 2.6-91.

ment-treated soil will have a 'beneficial effect' on the factor of safety against sliding." Request at 6-7. This proposed basis presupposes that there is an inconsistency in PFS presenting two sets analyses, one showing the results of a calculation that takes credit for the properties of the soil cement and another one that, for added conservatism, does not take credit for those properties in order to show that, regardless, the PFSF meets regulatory requirements. See Trudeau Declaration ¶ 10. No such inconsistency exists. Id. Instead, it is typical, and expected, of an Applicant to provide analyses based on a variety of scenarios. For that reason, the State's demand that "PFS should either include the passive resistance scenario and address all the attendant shortcomings with the use of cement-treated soil or eliminate that concept from its analyses" (Request at 7) is entirely without factual or legal support.

More importantly, the State's assertion is by its own admission inexcusably late. The calculations that the State now argues are "inconsistent" would have been equally "inconsistent" in Revision 7 of the calculation. The State admits that Revision 8 has not changed from Revision 7 in this respect when it states: "Revision 8 still persists in using the same 'beneficial effect' without any analysis whatsoever of the tensile strength of the cement-treated soil." Request at 6 (emphasis added). In explaining why the PFS approach is incorrect, the State explains that "PFS cannot claim any 'beneficial effect' from cement-treated soil unless and until it addresses several possible failure mechanisms regarding cement-treated soil's ability to withstand dynamic bending, torsional, and beam shear stresses; its long-term durability without cracking or without significant shear strength degradation; and its interaction with soil chemistry. Bartlett Dec. ¶ 10." Id. This argument, as explicitly acknowledged by Dr. Bartlett, is lifted directly from Proposed Utah QQ and is therefore neither "new" nor timely.[14] See also, Trudeau Declaration ¶ 15.

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[14] As Dr. Bartlett writes: "Left unaddressed in G(B)-04, Rev. 8, are the cement-treated soil's ability to withstand dynamic bending, torsional, and beam shear stresses; its long-term durability without cracking or without significant shear strength degradation; and its interaction with soil chemistry. *See* Utah QQ, Bartlett Dec. ¶¶ 13, 15 and 17; Mitchell Dec. ¶13." Bartlett Declaration ¶ 10.

The State admits that it was aware of its asserted "deficiency" in the calculation's failure to consider bending and torsional stresses even before the filing of Utah QQ, since the State refers to the fact that it brought this asserted deficiency to the attention of PFS during the November 2000 deposition of PFS witness Paul Trudeau.[15] Therefore, this alleged basis even if "new" would in any event be untimely because it is based on facts of which the State has been aware for at least more than six months.

In short, this "new" basis is both not new and inexcusably late and must therefore be rejected.

3. PFS's Treatment of the Adhesive Strengths of Various Foundation Interfaces has not Changed in the Revision 8 Analyses

The State asserts as a "new" basis that PFS's calculations in Cask Storage Pad Stability Calc. Rev. 8 make improper assumptions about the adhesion between the base of the pad and the underlying clayey soils. Request at 7. The State goes on to explain its argument by stating that:

> What Revision 8 still does not address is the impact of increased water content, disturbance and remolding to the strength of the partly saturated silty clay/ clayey silt soils beneath the cement-treated soil, and, thus, PFS cannot take credit in its analysis for the static undrained strength of the clayey soils being "fully engaged." Bartlett Dec. ¶11.

Request at 7. Again, this claim is expressly acknowledged to be a repetition of allegations made during written discovery and depositions concerning contention Utah L as well as in Proposed Utah QQ, and is accordingly not new.[16] Moreover, State witness Dr. Bartlett was aware of the adhesion assumption made by PFS and signified his agreement with it during his deposition in November 2000. See Trudeau Declaration ¶ 15. The State should have raised any issues it had

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[15] See Request at 7; Bartlett Declaration ¶ 10 and Exhibit A.

[16] Dr. Bartlett identifies the source of this allegation as follows: "The Applicant has still not addressed the significant concerns expressed by State and its experts during written discovery and depositions in Utah L and, most recently, in Dr. James Mitchell's May 16, 2001 Declaration ¶ 14, supporting Utah QQ. Not in calculation G(B)-04, Rev. 8 or elsewhere, has the Applicant addressed the impact of increased water content to the adhesion of the partly saturated silty clay, clayey silt soils beneath the cement-treated soil. Changes in water content in this layer could impact the settlement, strength, and the adhesion between the soil and the soil-cement. *See* Utah QQ, Mitchell Dec. ¶ 14." Bartlett Declaration ¶ 11.

with the assumptions underlying this calculation, including the assumption that the static undrained strength of the clayey soils underlying the site can be fully engaged to prevent sliding, when the calculation's methodology was first set forth, long before the filing of Proposed Utah QQ. Trudeau Declaration ¶ 16.

4. The State's Claim Regarding PFS's Longitudinal Analysis of the Storage Pads Are Unjustifiably Late

The State's last "new" basis is that PFS has incorrectly calculated the factor of safety against sliding of individual pads in the longitudinal direction. Request at 7. This claim is based in part on the previously-discussed assertion that PFS has failed to properly account for the inertial force of the soil cement, and is inadmissible for the reasons indicated above in connection with that claim.

The second "error" asserted by the State against the computation of the factor of safety against cask sliding in the longitudinal direction is that "PFS has used an incorrect peak undrained strength to represent the soil's shear resistance for the high levels of inertial forces introduced by the new design basis ground motions." (Request at 7). The State, however, unabashedly admits that "[t]his issue is a long-standing dispute between PFS and the State in Utah L and it still persists in Revision 8." (Id. at 7-8). Thus, by the State's own admission, this is not a new matter and to assert it at this time is impermissibly late.

B. UNTIMELINESS AND REPETITIVENESS OF CLAIMS AGAINST THE SEISMIC STABILITY ANALYSIS OF THE CTB

With respect to the stability analysis of the CTB, the State asserts that "PFS *still* persists in assuming that the passive resistance of cement-treated soil around the CTB is available to resist sliding." (Response at 10, emphasis added). The State also argues that "[t]he new analyses in Revision 5 do not overcome the concerns raised in Utah QQ because the Applicant *still* inappropriately relies on the passive pressure from cement-treated soil to resist seismic loading." Id. (emphasis added). Thus, the issue that the State has with the PFS calculation is one that, by the

State's own admission, it has already raised in Proposed Utah QQ. See Trudeau Declaration ¶ 17. No modification of the proposed contention is therefore warranted.

C. 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 less than 20 days of receipt of the two revised seismic stability analyses [sic] calculations for the pads and CTB," and "the revised calculations raise new safety concerns." For the reasons discussed above, this argument is invalid. Each of the issues that the State seeks to raise were either raised in Proposed Utah QQ, or could have been raised then or much earlier. As such, the revised calculations raise no new safety concerns. 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 (emphasis added, citations omitted). The four remaining factors are: (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. Of those factors, the third and fifth are to be accorded more weight than the second and fourth. Id.

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 encompasses a broad range of issues relating to soil cement 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 in 10 C.F.R. § 2.714(a)(1). The modifications to Proposed Utah QQ are in most cases a 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.

With respect to factor two, 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 Request to modify Proposed Contention Utah QQ fails to raise a litigable contention and should be denied.

Respectfully submitted,

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Dated: July 3, 2001

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

In the Matter of	)	
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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 Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to Further Revised Calculations From the Applicant" 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 3rd day of July, 2001.

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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 Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to Further Revised Calculations From the Applicant." I have reviewed the proposed Contention Utah QQ ("Proposed Utah QQ"), as submitted by the State of Utah ("State") in this proceeding, the State of Utah's "Request to Modify the Bases of Late-Filed Contention Utah QQ in Response to Further Revised Calculations from the Applicant" ("Request"), and the supporting Declaration of Dr. Steven F. Bartlett dated June 19, 2001 ("Bartlett 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 hereto. I have twenty-eight years of experience in geotechnical engineering. Geotechnical engineering is the branch of civil engineering that con-

cerns itself with subsurface soil investigations and analyses 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"). 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. I have been lead geotechnical engineer for the PFSF since December 1997. In that capacity, I have supervised the conduct of the subsequent subsurface investigations, laboratory testing programs, and geotechnical analyses that have been conducted on behalf of PFS to support the design of the site structures.

5. 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. 8, *Stability Analyses of Cask Storage Pads* (May 31, 2001) ("Cask Storage Pad Stability Calc. Rev. 8"), and 05996.02-G(B)-13, Rev. 5, *Stability Analyses of Canister Transfer Building* (May 31, 2001) ("Canister Transfer Building Stability Calc. Rev. 5"). Both calculations were updated versions of calculations of the same respective titles, issued on March 30, 2001 ("Cask Storage Pad Stability Calc. Rev. 7" and "Canister Transfer Building Sta-

bility Calc. Rev. 4"). Copies of relevant excerpts from the four cited calculations are included as Exhibits 2 through 5 hereto.

6. The above-cited calculations were revised in May 2001 to include a clear identification of the potential failure modes, failure planes, shear strength available to resist sliding along those planes, and the factors of safety against sliding along those planes, as requested by the Staff of the U.S. Nuclear Regulatory Commission ("Staff"). However, the underlying analyses and assumptions remain unchanged between the two sets of calculations.

7. One of the sliding analyses performed by PFS (in both Rev. 7 and Rev. 8), demonstrates the conservatism built into the cask storage pad stability analyses by assuming that the casks are resting upon native clayey soil and taking no credit for the presence of soil cement underneath or adjacent to the pads. In these analyses, PFS demonstrates (pp. 16-21 of Exhibit 2 hereto (Rev. 7) and pp. 17-22 of Exhibit 3 (Rev. 8)) that the static shear strength of the native soils is sufficient to provide a factor of safety against sliding that exceeds the minimum required value of 1.1. For these analyses, Cask Storage Pad Stability Calc. Rev. 8 differs from the same analysis in Cask Storage Pad Stability Calc. Rev. 7 only in that the description of the analysis in Rev. 8 states explicitly that the soil cement is not being relied on to provide resistance against pad sliding. While the description of the analysis was clarified, no change was made to the design methodology; rather, this methodology has always been used to show the conservative nature of the design. Thus, page 20 of Rev. 7 (Exhibit 3) indicates that the minimum factor of safety against sliding of the pad would be 1.25, if the pads were constructed directly on the clayey soils. This number is unchanged in Rev. 8, as shown on p. 22 of Exhibit 3, except it is in a bolded font with "**(Mini-**

mum)" added to emphasize that this is the minimum factor of safety against sliding, obtained conservatively by taking no credit for the presence of soil cement.

8. Likewise, Rev. 8 (Exhibit 3, pp. 14-16, 23, 25, 29, 33, 37, and 38) of the calculation clarifies that the soil cement beneath the pad serves as an "engineered mechanism" to bond the pad to the underlying clayey soils, so that the full cohesive strength of the clayey soils can be relied on to resist sliding. This, again, is not a new design feature. Page 15 of Rev. 7 (Exhibit 2) states: *"The soil cement will have higher shear strength than the underlying silty clay/clayey silt layer; therefore, the resistance to sliding on that interface [between the soil cement and the underlying native soil] will be limited by the shear strength of the silty clay/clayey silt."* Page 21 of Rev. 7 (Exhibit 2) further states: *"The soil cement will be mixed and compacted into the surface of the silty clay, providing a bond at the interface that will exceed the strength of the silty clay. Therefore, this interface will have more resistance to sliding than is included in these analyses and, thus, there will be adequate resistance at this interface to preclude sliding of the pads due to the loads from the design basis ground motion."* In other words, the ability of the soil cement to bond the pad to the underlying soil has been recognized long before Rev. 8 was issued.

9. The Request also seeks to draw a distinction between the reliance upon the buttressing effect of the soil cement around the pads in Rev. 7 and the "confusing and conflicting analysis" in Rev. 8 in which PFS is allegedly "at times ignoring the buttressing effect of soil cement on the factor of safety against sliding and other times adding the buttressing effect back into its analysis." (Request at 4). This alleged distinction does not exist. Both of these calculations include analyses in which credit is taken for the presence of soil cement wherein the buttressing effect of the soil cement around

the pads is credited with increasing the resistance against sliding of the pads. Page 13 of Rev. 7 illustrates that the soil cement adjacent to the pads need only have an unconfined compressive strength of 340 psi to provide all of the resistance to sliding required to obtain a factor of safety of 1.1, ignoring all of the shear strength available at the bottom of the pad, arguably a very conservative assumption. It further indicates that soil cement with strengths higher than this are readily achievable and cites a state-of-the-art report on soil cement, published by the American Concrete Institute, as evidence. Pages 14 and 15 of Rev. 7 demonstrate that basing the resistance to sliding on only the shear resistance acting on the bottom of the pad, the soil cement beneath the pads only needs to be designed to have a shear strength of 20 psi (which equals an unconfined compressive strength of 40 psi) to preclude sliding of an entire row of pads. This is higher strength than the underlying silty clay/clayey silt; therefore, the resistance to sliding on the interface between the soil cement and the silty clay/clayey silt layer will be limited by the shear strength of the silty clay/clayey silt. As discussed above, the factor of safety against sliding of the pads constructed directly on silty clay/clayey silt has a minimum factor of safety of 1.25, ignoring the buttressing effect of the soil cement adjacent to the pads. Adding the buttressing effect and increasing the strength of the material beneath the pad from that of the native clayey soils to that of the higher strength soil cement (or cement-treated soil) clearly will increase the factor of safety against sliding. Pages 27 and 28 of Rev. 8 make this clear, presenting analyses showing the beneficial effect of the soil cement adjacent to the pads in increased factors of safety against sliding compared to the minimum value of 1.25 cited above for analyses that assumed the soil cement was not used.

10. Both the Request (at p. 5) and the Bartlett Declaration (at para. 9) attack the first of the two analyses described above as being flawed for failing to include the inertial forces due to the soil cement beneath the pad in the sliding forces. However, as just discussed, the analysis that is being attacked has not changed since soil cement was introduced into the stability analyses of the storage pads in LA Amendment 8 in December 1999. Therefore, this objection could have been raised at any time since LA Amendment 8 was filed. Moreover, these inertial forces are included in the analyses of sliding of an entire row of pads in both of these calculations (pp. 14 & 15 of Rev. 7 and pp. 29 – 33 of Rev. 8), and adequate factors of safety are demonstrated.

11. The State also asserts that in Revision 8 to the Cask Storage Pad stability analysis "PFS presents a confusing and inconsistent design approach in considering whether or not the passive resistance provided by cement-treated soil will have a 'beneficial effect' on the factor of safety against sliding." (Request at 6.) However, this criticism confuses the two types of analyses that PFS performed (as discussed above), one a calculation that takes credit for the properties of the soil cement and another one that, for added conservatism, does not take credit for those properties in order to show that, regardless, the PFS meets regulatory requirements. Thus, no inconsistency exists between the two assumptions. Instead, it is typical for, and expected of, an Applicant to provide analyses based on a variety of scenarios to demonstrate "defense in depth".

12. With respect to the analysis that includes the resistance to sliding provided by the soil cement, the Request (at p. 7) and Dr. Bartlett's Declaration (at para. 10) criticize the assumption that the soil cement beneath the pad can serve as an "engineering mechanism" because PFS has allegedly failed to address several possible failure mecha-

nisms of the soil cement. However, these potential mechanisms could have been raised at any time since the use of soil cement was first introduced in 1999, and they are in fact raised by Dr. Bartlett in his declaration in support of Proposed Utah QQ. Moreover, Dr. Bartlett has been aware of this feature of the design for some time, as evidenced by his deposition at 390:2-6 (November 17, 2000), wherein he states regarding the soil cement beneath the pads: *"We assume that the lower interface, where the soil cement interfaces with the clay, that that would probably be a fairly rough interface and that it might be appropriate then to use full cohesion."*

13. The Request also confuses the two sets of analyses described above by trying to compare the first analysis in Rev. 7 (which it labels "hypothetical") with the second analysis in Rev. 8. (Request at 5). As explained above, these are two sets of analyses, both performed in each revision of this calculation, one assuming the pads rest on native soil and the other taking into account the beneficial effects of soil cement. Comparing the first analysis in Rev. 7 to the second analysis in Rev. 8 is inappropriate.

14. The Request and the Bartlett Declaration raise other alleged deficiencies against Cask Storage Pad Stability Calc. Rev. 8. See Bartlett Declaration, para. 10-12. However, in each instance, the alleged shortcoming in the calculation is a repetition of claims already made in Proposed Utah QQ or it could have been raised in that contention because Rev. 7 (the revision in effect at the time Proposed Utah QQ was filed) is identical to Rev. 8 in that regard. For example, in para. 10 of his Declaration, Dr. Bartlett states: "Left unaddressed in G(B)-04, Rev. 8, are the cement-treated soil's ability to withstand dynamic bending, torsional, and beam shear stresses; its long-term durability without cracking or without significant shear strength degradation; and its interac-

tion with soil chemistry. *See* Utah QQ, Bartlett Dec. ¶¶ 13, 15, and 17; Mitchell Dec. ¶ 13." These claims are obviously repetitive of those raised in Utah QQ.

15. In para. 11 of his Declaration, Dr. Bartlett states: "In my opinion there are significant shortcomings in the adhesive strength the Applicant assumes at various foundation element interfaces (*e.g.*, bottom of cement-treated soil and top of native soil). The Applicant has still not addressed the significant concerns expressed by State and its experts during written discovery and depositions in Utah L and, most recently, in Dr. James Mitchell's May 16, 2001 Declaration ¶ 14, supporting Utah QQ." This assertion is also a restatement of the claims made by the State and its witnesses in Utah QQ. In addition, as indicated in ¶9 above, Dr. Bartlett has been aware for some time of the design assumption that the use of soil cement would provide increased shear strength. For example, in his deposition of November 17, 2000 at 390:2-6, Dr. Bartlett stated the following with respect to the adhesive strength between the "bottom of cement-treated soil and top of native soil" for the soil cement beneath the pads: *"We assume that the lower interface, where the soil cement interfaces with the clay, that that would probably be a fairly rough interface and that it might be appropriate then to use full cohesion."*

16. In para. 12 of the Bartlett Declaration, the use of peak undrained strength to represent the soil's shear resistance for the high levels of cyclic inertial forces introduced by the new design basis ground motion is criticized as "incorrect" because "without the results of cyclic testing performed at the appropriate strain levels for the PFS soils, a conservative reduction in peak undrained shear strength should be used in design." However, the State concedes that this is "a long-standing dispute between PFS and

the State in Utah L" (Request at 9) and is indeed part of the issues covered in Applicant's outstanding motion for summary disposition of Utah L.

17. Likewise, the latest stability analyses of the Canister Transfer Building ("Canister Transfer Building Stability Calc. Rev. 5"), differs from those in the previous version of the calculation only in that, in response to NRC requests for clarification, the revised calculation clearly identifies the potential failure modes, failure planes, shear strength available to resist sliding along those planes, and provides calculations of the factors of safety against sliding 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. The Request and Mr. Bartlett's Declaration admit that nothing of substance has changed in going from Rev. 4 to Rev. 5 of this calculation. Mr. Bartlett states: "This analysis assumes that full passive resistance of the soil cement (*i.e.*, full buttress effect) is available to resist sliding and that the underlying clayey soil has reached residual strength due to the large strain required to mobilize the passive resistance. In my opinion it is improper to use any passive resistance from the soil cement in any sliding *calculations for the reasons discussed in Utah QQ* and ¶ 10 of this declaration." (Bartlett, para. 10) Likewise, the Request states "PFS *still persists* in assuming that the passive resistance of cement-treated soil around the CTB is available to resist sliding. PFS's analysis *was unconservative and inaccurate in Revision 4 of Cal. G(B)-13, and it remains so* in Revision 5 too." Request at 10, emphasis added.

18. In summary, in my review of the Request and the supporting Bartlett Declaration, I did not identify any claims that have not already been made in Utah QQ, 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 July 3, 2001.

Paul J. Trudeau

Paul J. Trudeau

EXHIBIT 1
Trudeau Declaration

Paul J. Trudeau

Senior Lead Engineer

Years Experience (as of December 1998)

At Stone & Webster: 26 With other Firms: 0

Department/Division/Location

Geotechnical/Division 50/Boston

Professional History

Stone & Webster Engineering Corporation, Boston, Massachusetts - 1973 to Present
Massachusetts Institute of Technology - Cambridge, Massachusetts - 1971 to 1973
Stone & Webster Engineering Corporation, Boston, Massachusetts - 1971 to 1972
Worcester Polytechnic Institute, Worcester, Massachusetts - 1967 to 1971

Areas of Expertise

- Geotechnical Engineering And Design
- Use of Computers In Geotechnical Analyses and Designs
- Managing Geotechnical Investigations
- Geotechnical Instrumentation
- Performing Cross-Hole Shear Wave Velocity Surveys
- Regulatory Compliance, Review, and Implementation (NRC)

Awards

Desmond Fitzgerald Medal awarded by the Boston Society of Civil Engineers for "Shear Wave Velocity and Modulus of a Marine Clay," Journal of the Boston Society of Civil Engineers, January 1974.

Computer Hardware/Software Capabilities

Mr. Trudeau has considerable experience with PC and mainframe computer programs for performing geotechnical analyses. He is expert in developing spreadsheets using Microsoft Excel and Lotus for solving complex engineering calculations and also is an expert FORTRAN programmer and in programming IBM JCL. He also has considerable experience in using MicroStation for generating report-quality sketches and figures and in using InRoads for plotting contours and determining earthwork quantities.

He is adept at developing batch programs, as well as programming in dBASE, AWK, perl, and developing shell scripts in Unix. He routinely uses these techniques for automatic placement of graphics at correct locations and scales in MicroStation design files for generation of geotechnical figures, such as boring location plans, subsurface profiles, contour maps, and other figures for reports.

Department/Division Assignments

Division Computer Coordinator

Training

40 hours of instruction in Waste Site Worker Protection and 8 hours of instruction in Supervisory Training to comply with OSHA 1910.120(e)(2&3)

Experience Summary

Mr. Trudeau has over 26 years of experience in the engineering industry. Currently, as a Senior Lead Engineer in the Geotechnical Division of Stone & Webster Engineering Corporation, he is designated as the Division Computer Coordinator and as the Division Specialist in cross-hole seismic velocity surveys. As Computer Coordinator, he is responsible for the development, documentation, and maintenance of more than 80 computer programs sponsored by the Geotechnical Division and for providing consulting for Geotechnical Division computer applications. As the Division Specialist in cross-hole seismic velocity surveys, he is responsible for performing the field testing and interpreting the data for use in static and dynamic analyses.

Since joining Stone & Webster Engineering Corporation in 1973, he has served as a Lead Geotechnical Engineer on numerous fossil power plants, Independent Spent Fuel Storage Installations (ISFSI) at Private Fuel Storage Facility in Skull Valley, UT and at Maine Yankee's nuclear plant in Wiscasset, ME, the Bellefonte Nuclear Plant, the Shoreham Nuclear Power Plant, the Falcon Seaboard Gas Pipeline, the TVA Widows Creek Steam Plant, and various projects at the Hanscom Air Force Base. He has also served as a Support Engineer on several nuclear and fossil power plant projects. In these roles, he was responsible for performing geotechnical investigations, preparing geotechnical analyses, developing geotechnical design criteria for other disciplines, such as Structural, Environmental, Engineering Mechanics, and Electrical, and for preparing geotechnical sections of Preliminary and Final Safety Analyses Reports and Environmental Reports. This work was performed in accordance with quality assurance programs that satisfied the quality assurance requirements of Appendix B of 10CFR Part 50 and NQA-1.

He was also responsible for reviewing geotechnical analyses and reports prepared by others on these projects, and for preparing testimony and for testifying at public hearings. He has also completed 40 hours of instruction in Waste Site Worker Protection and 8 hours of instruction in Supervisory Training to comply with OSHA 1910.120(e)(2&3) and is certified to work on hazardous waste sites.

Mr. Trudeau's field experience includes performing cross-hole shear wave velocity tests in Maine, Connecticut, and Texas, geotechnical boring supervision at Jamesport, Shoreham, and Shoreham West on Long Island in New York and at Wards Island in New York, New York, and a compaction control investigation and intake canal revetment repair at Shoreham Unit No. 1. He has performed inspections of the haul road for transport of 300-ton steam generators at the North Anna Nuclear Power Station in Virginia, and has inspected the route proposed by Chem-Nuclear for transport of the 800-ton reactor pressure vessel from the Shoreham Nuclear Power Station to their disposal facility in Barnwell, South Carolina. In addition, he has served as Lead Scientist/Field Supervisor of environmental borings that were drilled for site assessment studies performed for New York City Department of Environmental Protection at their Jamaica, Wards Island, and 26th Ward water pollution control plants.

Mr. Trudeau's laboratory experience includes performing index property tests, consolidation tests, Hardin Oscillator tests, and static and dynamic triaxial tests. He was instrumental in selection, installation, and testing and debugging of Stone & Webster's Geotechnical laboratory data acquisition system. His educational experience encompasses many aspects of civil engineering, including soil mechanics and foundations, computer programming (FORTRAN), soil dynamics, earthquake engineering, geotextiles, and structures.

Education

Master of Science in Civil Engineering, MIT, Cambridge, Massachusetts - 1973
B.S. in Civil Engineering, Worcester Polytechnic Institute, Worcester, Massachusetts - 1971

Licenses, Registrations, and Certifications

Professional Engineer - Massachusetts - 1977

Professional Affiliations

Chi Epsilon: Member - 1969
American Society of Civil Engineers: Member 1971
Boston Society of Civil Engineers Section/ASCE: Member 1971
International Society of Soil Mechanics and Foundation Engineering: Member 1974

BSCES Director
BSCES Awards Committee - Chairman
BSCES Student Chapter Committee - Chairman
BSCES Membership Committee - Member
BSCES Task Force for Younger Members - Member
ASCE National Convention Attendance Committee - Co-Chairman
BSCES Geotechnical Engineering Practice Lecture Series Committee - Member

Publications

Trudeau, P.J., Whitman, R.V., and Christian, J.T., "Shear Wave Velocity and Modulus of a Marine Clay," Journal of the Boston Society of Civil Engineers, January 1974.

Pierce, D.S., and Trudeau, P.J., "Digital and Analog Methods for the Development of Stereoscopic Contour Maps for Geological and Geophysical Analysis," Geological Society of America Abstracts with Programs, Vol. 10, No. 7, 1978.

Experience History

STONE & WEBSTER ENGINEERING CORPORATION, BOSTON, MASSACHUSETTS - 1973 TO PRESENT

Geotechnical Division (Apr 1977 to Present)
Computer Coordinator

Independent Spent Fuel Storage Installation (Sept 1998 to Present)
Maine Yankee Atomic Power Company – Wiscasset, ME
Lead Geotechnical Engineer

VX Full Scale Plant (Mar 1998 to Present)
U.S. Army Program Manager for Chemical Demilitarization, Aberdeen, Maryland
Lead Geotechnical Engineer

Combined-Cycle Power Plant (Feb 1998 to Present)
EMI, Rumford, ME and Tiverton, RI
Lead Geotechnical Engineer

Private Fuel Storage Facility – Skull Valley, UT (Dec 1997 to Present)
Private Fuel Storage, Limited Liability Corporation
Lead Geotechnical Engineer

VX Full Scale Plant (April 1997 to Present)
U.S. Army Program Manager for Chemical Demilitarization, Newport, IN
Lead Geotechnical Engineer

Mystic, Edgar, and Medway Combined Cycle Power Plants (Mar 1998 to Dec 1998)
Sithe Energies, Inc
Geotechnical Engineer

Terminal A Area 8 (Mar 1998 to Oct 1998)
MASSPORT
Geotechnical Engineer

Tapoco Developments (Dec 1997 & July/Aug 1998)
Santeetlah Dam
Geotechnical Engineer

Tapoco Developments (Aug 1997 to Sept 1997)
Cheoah Dam

Big Brown Steam Electric Station, Fairfield, TX (July 1997 to Nov 1998)
TU Electric Company
Geotechnical Engineer

Building 99 Fuel Oil Storage Facility (June 1997 to Aug 1997)
GE River Works Plant – Lynn, MA
Geotechnical Engineer

Private Fuel Storage Facility – Skull Valley, UT (Jan 1997 to Oct 1997)
Private Fuel Storage, Limited Liability Corporation
Geotechnical Engineer

Building 66 G & L G60TX Foundation (Dec 1996 to Jan 1997)
GE River Works Plant – Lynn, MA
Geotechnical Engineer

Tapoco Developments (Nov 1996 to Feb 1997)
Calderwood Dam
Geotechnical Engineer

19th St Substation (Oct 1996 to Jan 1998)
Potomac Electric Power Co, Washington, D. C.
Geotechnical Engineer

Boston Ramps (Feb 1996 to Dec 1996)
Massachusetts Turnpike Authority
Geotechnical Engineer

Goodhue County Independent Spent Fuel Storage Installation (Dec 1995 to Sept 1996)
Northern States Power Company
Geotechnical Engineer

Central Artery/Third Harbor Tunnel Project (Feb 1994 to January 1997)
Mass. Department of Public Works
Manager of Computer Services

Bellefonte Nuclear Plant (Oct 1993 to Mar 1994)
Tennessee Valley Authority
Lead Geotechnical Engineer

Chubb & Son, Incorporated (Sept 1993 to Jan 1994)
Geotechnical Consultant

Granite State Gas Transmission Company (Nov 1993)

Petersburg Generating Station (July 1993 to Sept 1993)
Indianapolis Power and Light Company

Pease Air Force Base (Aug 1993)
United States Air Force
Geotechnical Engineer

Green Mountain Power Corporation (July 1993)
Geotechnical Engineer

E. W. Stout Generating Station (July 1993)
Indianapolis Power and Light Company
Geotechnical Engineer

Hanscom Air Force Base (Apr 1993 to July 1993)
United States Air Force
Lead Geotechnical Engineer

Portland Natural Gas Transmission System (Nov 1992 to Apr 1993)

Maine Low-Level Radioactive Waste Authority (Oct 1992 to May 1993)
Geotechnical Engineer

Afobaka Dam (Oct 1992 to Jan 1993)
Suriname Aluminum Company
Geotechnical Engineer

Widows Creek (Sept 1992 to Feb 1993)
Tennessee Valley Authority
Lead Geotechnical Engineer

General Support Services Contract, Richland Field Office (Sept 1992 to Oct 1992)
U. S. Department of Energy

Patriot Generating Station (June 1992 to Aug 1992)
Indianapolis Power and Light Company
Geotechnical Engineer

Bellefonte Nuclear Plant (Feb 1992 to July 1992)
Tennessee Valley Authority
Lead Geotechnical Engineer

Central Artery/Third Harbor Tunnel Project (Mar 1990 to Feb 1992)
Mass. Department of Public Works
Manager of Computer Services

Petersburg Generating Station (Nov 1991 to Jan 1992)
Indianapolis Power and Light Company
Geotechnical Engineer

Petersburg Generating Station (Sept 1991 to May 1992)
Indianapolis Power and Light Company
Geotechnical Engineer

New Production Reactor (Sept 1991 to Oct 1991)
US Department of Energy
Geotechnical Engineer

New Production Reactor (Feb 1991 to May 1991)
US Department of Energy
Geotechnical Engineer

Widows Creek Steam Plant - Unit 8 (Feb 1991 to June 1991)
Tennessee Valley Authority
Lead Geotechnical Engineer

North Anna Nuclear Power Station (Sept 1991)
Virginia Power Company
Geotechnical Engineer

EG & G Rocky Flats (Sept 1991)
US Department of Energy
Geotechnical Engineer

Hanscom Air Force Base (Jan 1991 to Feb 1991)
United States Air Force
Lead Geotechnical Engineer

Hanscom Air Force Base (Jan 1990)
United States Air Force
Lead Geotechnical Engineer

Sludge Management Project (Sept 1989 to July 1990)
New York City Department of Environmental Protection
Geotechnical Engineer / Geotechnical Field Inspector / Lead Scientist/Field Supervisor

Plattsburgh 12 In. Diameter Gas Pipeline (Feb 1989 to Apr 1990)
Falcon Seaboard Pipeline Company
Lead Geotechnical Engineer

Great Northern Paper Company (Feb 1989 to May 1989)
Geotechnical Engineer

Shoreham Nuclear Power Station - Unit No. 1 (Jan 1983 to Mar 1992)
Long Island Lighting Company
Lead Geotechnical Engineer

Office of Nuclear Waste Isolation (ONWI) of Battelle Memorial Institute (Jan 1982 to Oct 1987)
U.S. Department of Energy
Geotechnical Computer Consultant

Bradley Lake Project (Feb 1986 to Oct 1986)
Alaska Power Authority
Geotechnical Engineer

Salt Cave Hydroelectric Project (Apr 1986 to May 1986)
City of Klamath Falls, Oregon
Geotechnical Engineer

Beaver Valley Power Station - Unit 2 (Oct 1984 to Aug 1985)
Duquesne Light Company
Geotechnical Engineer

Malakoff Site (Apr 1982 to Dec 1982)
Houston Lighting & Power Company
Geotechnical Engineer

Site X (Aug 1981 to Dec 1981)
Houston Lighting & Power Company
Geotechnical Engineer

Patriot Station (May 1981 to July 1981)
Indiana Power and Light Company
Geotechnical Computer Consultant

Site X (May 1981 to July 1981)
Houston Power and Light Company
Geotechnical Computer Consultant

Site X (Mar 1981 to May 1981)
Houston Lighting & Power Company
Geotechnical Engineer

Western Fuels Association. Inc. (Dec 1980)
Geotechnical Computer Consultant

Patriot Station (Nov 1980 to Dec 1980)
Indiana Power and Light Company
Geotechnical Computer Consultant

Site X (Oct 1980)
Houston Lighting & Power Company
Geotechnical Engineer

Pumped Storage Project (Apr 1980 to July 1980)
Public Service Company of New Mexico
Geotechnical Computer Consultant

Beaver Valley Power Station - Unit No. 2 (Feb 1980 to Mar 1980)
Duquesne Light Company
Geotechnical Computer Consultant

Millstone Unit No. 3 (Feb 1980)
Northeast Utilities Service Company
Geotechnical Engineer

Martin Cooling Dike (Jan 1980)
Florida Power and Light Company
Geotechnical Engineer

Beaver Valley Power Station - Unit No. 1 (Mar 1970 to May 1979)
Duquesne Light Company
Geotechnical Computer Consultant

Haven Nuclear Power Station (Dec 1978 to Jan 1979)
Wisconsin Electric Power Company
Geotechnical Engineer

Office of Nuclear Waste Isolation (ONWI) of Battelle Memorial Institute (Sept 1978 to Nov 1979)
U.S. Department of Energy
Geotechnical Computer Consultant

Stuyvesant & New Haven Sites (Apr 1978 to Sept 1978)
New York State Electric and Gas Corp.
Geotechnical Computer Consultant

Sundesert 500 kV Transmission and Substation Project (Aug 1977 to Dec 1977)
San Diego Gas and Electric Company
Geotechnical Computer Consultant

Jamesport Nuclear Power Station (July 1976 to Apr 1977)
Long Island Lighting Company
Geotechnical Engineer

Shoreham Unit No. 1 (Feb 1976 to June 1976)
Long Island Lighting Company
Geotechnical Engineer

Jamesport Nuclear Power Station (Feb 1975 to Jan 1976)
Long Island Lighting Company
Geotechnical Engineer

Shoreham Unit No. 1 (Sept 1974 to Jan 1975)
Long Island Lighting Company
Geotechnical Engineer

Shoreham Unit No. 1 (June 1974 to Aug 1974)
Long Island Lighting Company
Geotechnical Engineer

Jamesport Nuclear Power Station (Mar 1974 to June 1974)
Long Island Lighting Company
Geotechnical Engineer

Shoreham Unit No. 1 (Oct 1973 to Apr 1974)
Long Island Lighting Company
Geotechnical Engineer

Jamesport and Shoreham West (Sept 1973)
Long Island Lighting Company
Geotechnical Engineer

Northfield Mountain Pumped Storage Project (Aug 1973 to Oct 1973)
Northeast Utilities Service Company
Geotechnical Engineer

Jamesport Nuclear Power Station (Aug 1973)
Long Island Lighting Company
Geotechnical Engineer

Geotechnical Division Computer Coordinator (Mar 1973 to Nov 1973)

North Anna Power Station (Feb 1973)
Virginia Electric and Power Company
Geotechnical Engineer

Massachusetts Institute of Technology - Cambridge, Massachusetts - 1971 to 1973
Graduate Research Assistant

EXHIBIT 2
Trudeau Declaration

bound value of the coefficient of friction between the casks and the storage pad (μ = 0.8, as shown in SAR Section 8.2.1.2) x the normal force acting between the casks and the pad. This force is maximum when the vertical inertial force due to the earthquake acts downward. However, when the vertical force from the earthquake acts downward, it acts in the same direction as the weight, tending to stabilize the structure. Therefore, the minimum factor of safety against overturning will occur when the dynamic vertical force acts in the upward direction, tending to unload the pad.

When the vertical inertial force due to the earthquake acts upward, the friction force = 0.8 x (2,852K - 0.695 x 2,852K) = 696 K. This is less than the maximum dynamic cask horizontal driving force of 2,212 K (Table D-1(c) in CEC, 2001). Therefore, the worst-case horizontal force that can occur when the vertical earthquake force acts upward is limited by the upper-bound value of the coefficient of friction between the bottom of the casks and the top of the storage pad, and it equals 696K.

$$\Sigma M_{Driving} = 1.5\text{ ft} \times \underset{a_h}{0.711} \times \underset{Wp}{904.5\text{ K}} + 3\text{ ft} \times \underset{EQhc}{696\text{ K}} = 3{,}053\text{ ft-K.}$$

$$\Rightarrow FS_{OT} = \frac{17{,}186\text{ ft} - \text{K}}{3{,}053\text{ ft} - \text{K}} = 5.63$$

This is greater than the criterion of 1.1; therefore, the cask storage pads have an adequate factor of safety against overturning due to dynamic loadings from the design basis ground motion.

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) = $\Sigma F_v = W_c + W_p + EQ_{vc} + EQ_{vp}$

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

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

B = 30 feet

L = 67 feet

SLIDING STABILITY OF THE PADS CONSTRUCTED ON AND WITHIN SOIL CEMENT

Objective:

Determine the minimum required strength of the soil cement along the sides of the pads and below the pads to provide a factor of safety against sliding of the cask storage pads of 1.1.

Method/Assumptions:

1. In determining the required strength of soil cement along the sides of the pad, assume that the resistance to sliding is provided only by the passive resistance of the soil-cement layer above the bottom of the pads, ignoring the contribution of the frictional portion of the strength.
2. Ignore the passive resistance of the overlying compacted aggregate, since it is only 1' thick (Figure 3).
3. Assume the active thrust of the compacted aggregate is less than the passive thrust and, thus, the active thrust can be ignored.
4. Use Eq 23.8a of Lambe & Whitman (1969) to calculate passive thrust, P_p, as follows:

$$P_p = \tfrac{1}{2}\gamma_w H^2 + \tfrac{1}{2}\gamma_b H^2 N_\phi + q_s H N_\phi + 2\bar{c} H \sqrt{N_\phi}$$

where:

H	=	height of soil cement above bottom of pad
N_ϕ	=	K_p, coefficient of passive pressure, = 1 assuming $\phi = 0$.
q_s	=	uniform surcharge, = $(\gamma \times H)_{\text{compacted aggregate}}$, > 0.125 kcf x 0.71 ft = 0.09 ksf
$\bar{c}$	=	effective cohesion

5. An adhesion factor (α) of 0.5 is conservatively assumed to determine resistance to sliding at the interface between the soil cement and the underlying silty clay.

Analysis:

Figure 3 presents an elevation view of the minimum thickness of soil cement in the vicinity of the cask storage pads. Figure 4 illustrates the passive pressures acting on the pads.

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

$$1.1\times\left[3'\times30'\times67'\times0.15\frac{K}{ft^3}+8\text{ casks}\times356.5\frac{K}{cask}\right]\times0.711$$

$\therefore$ T = 2,938 K

Assuming this resisting force is provided only by the passive resistance provided by the 2-ft thick layer of soil cement adjacent to the pads, as shown in Figures 3 & 4, the minimum required strength of the soil cement is calculated as follows. Note, ignore buoyancy, since the depth to the water table is ~124.5 ft below grade, as measured in Observation Well CTB-5 OW.

$$P_P=\frac{1}{2}\gamma H^2 N_\phi + q_s H N_\phi + 2\bar{c}H\sqrt{N_\phi}$$ EQ 23.8a of Lambe & Whitman (1969

where $q_s=(\gamma\cdot H)_{\substack{\text{compacted}\\ \text{aggregate}}}=0.125\frac{K}{ft^3}\times\frac{8.5\text{ in.}}{12\text{ in./ft}}=0.09\text{ ksf/LF}$, which is negligible.

Conservatively assuming $\phi = 0°$ for soil cement, $N_\phi = K_P = 1.0$.

Assuming sliding resistance is provided only by the passive resistance of the soil cement, the minimum resistance will exist for sliding in the N-S direction, because the width in the east-west direction (B=30') is less than the length in the north-south direction (L=67').

Find the minimum cohesion required to provide FS = 1.1.

$$P_P \text{ must be} \geq 2{,}938K = \frac{1}{2}\cdot\overset{\gamma}{0.100\frac{K}{ft^3}}\times\overset{H^2}{(2\text{ ft})^2}\times\overset{K_P}{1.0}+2\bar{c}\cdot\overset{H}{2\text{ ft}}\cdot\overset{\sqrt{N_\phi}}{\sqrt{1.0}}$$

$$\frac{2{,}938K}{30\text{ ft}}=0.2\frac{K}{ft}+4\bar{c}=97.93\frac{K}{LF} \Rightarrow 4\bar{c}=97.93\frac{K}{LF}$$

$$\therefore\ \bar{c}\ \geq\ 24.48\text{ ksf}\times\left(\frac{ft}{12\text{ in.}}\right)^2\times\frac{1{,}000\#}{K}=170\text{ psi}$$

The unconfined compressive strength equals twice the cohesion, or 340 psi. 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 >340 psi, the percentage of cement required would be ~340/40 =

8.5%. This is even less cement than would typically be used in constructing soil cement for use as road base, and it would be even lower if shear resistance acting on the base of the pad was included or if K_p was calculated for $\phi > 0°$. Note, Tables 5 & 6 of Nussbaum & Colley (1971) indicate ϕ exceeds 40° for all A-4 soils (CL & ML) treated with cement. Therefore, soil cement will greatly improve the sliding stability of the cask storage 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 Figure's 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.

The following analysis calculates the minimum strength required to preclude sliding of an entire row of pads along the base of the soil cement.

WEIGHTS

Casks: Wc = 8 x 356.5 K/cask = 2,852 K

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

EARTHQUAKE ACCELERATIONS – PSHA 2,000-YR RETURN PERIOD

a_H = horizontal earthquake acceleration = 0.711g

a_V = vertical earthquake acceleration = 0.695g

Consider a row of 10 pads with 2' of soil cement in between the pads and at least 1' of soil cement under the pads:

Weight of Casks	= 10 x 2,852 K	= 28,520 K
Weight of Pads	= 10 x 904.5 K	= 9,045 K
Weight of Soil Cement	= 9 x 3 ft x 30 ft x 5 ft x .10 kips/ft^3	= 405 K
	Total	= 37,970 K

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

1.1 x (Weight of Casks + Weight of Pads + Weight of Soil Cement) x 0.711

= 1.1 x (28,520 K + 9,045 K + 405 K) x 0.711

Therefore, T = 29,696 K

Base Area, A, of a row of 10 pads is given by

$A = 10 \times 30 \text{ ft} \times 67 \text{ ft} + 9 \times 30 \text{ ft} \times 5 \text{ ft} = 21{,}450 \text{ ft}^2$

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

T = cohesion ($\bar{c}$) x adhesion factor (α) x area (A)

$T = \bar{c} \times 0.5 \times 21{,}450 \text{ ft}^2$

$\bar{c} \times 0.5 \times 21{,}450 \text{ ft}^2 = 29{,}696 \text{ K}$

$\bar{c} = 2.77 \text{ ksf} = 20 \text{ psi}$

The unconfined compressive strength equals twice the cohesion, or 40 psi.

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.

The soil cement will have higher shear strength than the underlying silty clay/clayey silt layer; therefore, the resistance to sliding on that interface will be limited by the shear strength of the silty clay/clayey silt. Direct shear tests on samples of the soils from the pad emplacement area indicate the shear strength available to resist sliding from loads due to the design basis ground motion is 2.1 ksf as shown in Figure 7 of Calc 05996.02-G(B)-5-2 (copy included in Attachment C).

The following pages illustrate that there is an adequate factor or safety against sliding of the pads, postulating that they are constructed directly on the silty clay/clayey silt and neglecting the passive resistance provided by the soil cement that will be surrounding the pads. The factor of safety against sliding along the soil cement/silty clay interface will be much greater than this, because the shearing resistance will be available over the areas between the pads, as well as under the pads, and additional passive resistance will be provided by the continuous soil cement layer existing below the pads.

SLIDING STABILITY OF THE PADS CONSTRUCTED DIRECTLY ON SILTY CLAY/CLAYEY SILT

Material around the pad will be soil cement. In this analysis, the passive resistance provided by the soil cement is ignored to demonstrate that there is an acceptable factor of safety against sliding of the pads if they were founded directly on the silty clay/clayey silt. The soil cement is assumed to have the same properties that were used in Rev 4 of this calculation to model the crushed stone (compacted aggregate) that was originally proposed adjacent to the pads. These include:

γ = 125 pcf — Because of the low density of the eolian silts that will be used to construct the soil cement, it is likely that γ will be less than this value. It is conservative to use this higher value, because it 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.

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 is not used, however, in this analysis for calculating sliding resistance. It 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.

H = 3 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).

For frictional materials, the resistance to sliding is lower when the forces due to the earthquake act upward; therefore, analyze the sliding stability for Load Case III, which has the dynamic forces due to the earthquake acting upward. To increase the conservatism of this analysis, assume 100% of the dynamic forces due to the earthquake act in both the N-S 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, estimate the driving forces due to dynamic active earth pressures acting on the length of the pad, tending to cause sliding to occur in the E-W direction.

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.

P_a = [0.5 x 125 pcf x (3 ft)^2 x 0.22] x 67 ft (length)/storage pad = 8,291 lbs.

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:

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

where :

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

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:

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

γ = unit weight of soil,

H = wall height, and

K_{AE} is calculated as shown above.

$\beta = \alpha = 0$

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

$\phi = 40^\circ$

Approximating $\sin(\phi - \theta) \approx 0$ and $\cos(\phi - \theta) \approx 1$

$$K_{AE} = \frac{1 - \alpha_V}{\cos\theta \cdot \cos(\delta + \theta)}$$

$$\delta = \frac{\phi}{2} = 20^\circ$$

$$\therefore \quad K_{AE} = \frac{1 - 0.695}{\cos 66.8^\circ \cdot \cos(20^\circ + 66.8^\circ)} = 13.87$$

Therefore, the combined static and dynamic active lateral earth pressure force is:

$$F_{AE_{E\text{-}W}} = P_{AE} = \frac{1}{2} \times \overset{\gamma}{125 \text{ pcf}} \times \overset{H^2}{(3 \text{ ft})^2} \times \overset{K_{AE}}{13.87} \times \overset{L}{67 \text{ ft}} / \text{storage pad} = 522.7 \text{ K in E - W direction.}$$

$$F_{AE_{N\text{-}S}} = 522.7 \text{ K} \times \frac{30 \text{ ft}}{67 \text{ ft}} = 234.1 \text{ K in the N - S direction.}$$

WEIGHTS

Casks: Wc = 8 x 356.5 K/cask = 2,852 K

Pad: Wp = 3 ft x 67 ft x 30 ft x 0.15 kips/ft^3 = 904.5 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)

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

$EQhc_y$ = 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. For frictional materials, sliding is critical when the foundation is unloaded due to uplift forces from the earthquake. Therefore, $EQh_{c\ max}$ 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: 100% N-S, -100% Vertical, 0% E-W Earthquake Forces Act Upward

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

FOUNDATION PAD EARTHQUAKE LOADINGS

EQvp = -0.695 x 904.5 K = -629 K

EQhp = 0.711 x 904.5 K = 643 K

CASE III: 100% N-S, -100% VERTICAL, 0% E-W

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

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

$$T = \underset{N}{1{,}146\ K} \times \tan \underset{\phi}{0°} + \underset{c}{2.1\ ksf} \times \underset{B}{30\ ft} \times \underset{L}{67\ ft} = 4{,}221\ K$$

The driving force, V, is defined as:

$$V = F_{AE} + EQhp + EQhc$$

The factor of safety against sliding is calculated as follows:

$$FS = \underset{T}{4{,}221\ K} \div (\underset{F_{AE}}{522.7\ K} + \underset{EQhp}{643\ K} + \underset{EQhc}{696\ K}) = 2.27$$

For this analysis, the value of EQhc was limited to the upper-bound value of the coefficient of friction, μ = 0.8, x the cask normal load, because if Qxd exceeds this value, the cask would slide. The factor of safety exceeds the minimum allowable value of 1.1; therefore the pads 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: 100% N-S, 100% VERTICAL, 0% 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

$$N = \underset{Wc}{2{,}852\ K} + \underset{Wp}{904.5\ K} + \underset{EQvc}{1{,}982\ K} + \underset{EQvp}{629\ K} = 6{,}368\ K$$

$$T = \underset{N}{6{,}368\ K} \times \tan \underset{\phi}{0°} + \underset{c}{2.1\ ksf} \times \underset{B}{30\ ft} \times \underset{L}{67\ ft} = 4{,}221\ K$$

The driving force, V, is defined as:

$$V = F_{AE} + EQhp + EQhc$$

The factor of safety against sliding is calculated as follows:

$$FS = \underset{T}{4{,}221\ K} \div (\underset{F_{AE}}{522.7\ K} + \underset{EQhp}{643\ K} + \underset{EQhc}{2{,}212\ K}) = 1.25$$

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. The factor of safety exceeds the minimum allowable value of 1.1; therefore the pads 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.

These analyses illustrate that if the cask storage pads were constructed directly on the silty clay/clayey silt layer, they would have an adequate factor of safety against sliding due to loads from the design basis ground motion. Because the soil cement is continuous between the pads, its interface with the silty clay will be much larger than that provided by the footprint of the pads and used in the analyses presented in this section. The soil cement will be mixed and compacted into the surface of the silty clay, providing a bond at the interface that will exceed the strength of the silty clay. Therefore, this interface will have more resistance to sliding than is included in these analyses and, thus, there will be adequate resistance at this interface to preclude sliding of the pads due to the loads from the design basis ground motion.

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				PAGE 88
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 7		

FIGURE 3

DETAIL OF SOIL CEMENT UNDER & ADJACENT TO CASK STORAGE PADS

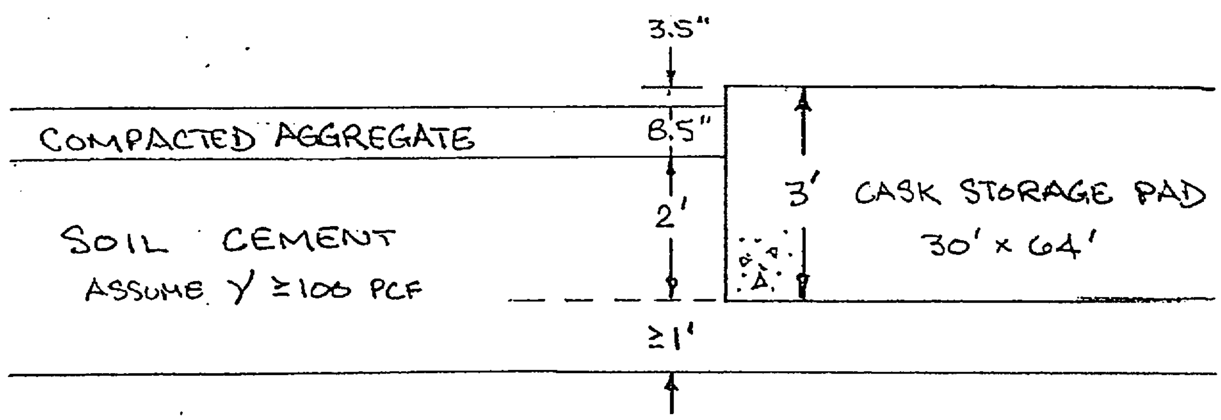


FIGURE 4

PASSIVE PRESSURE ACTING ON CASK STORAGE PADS

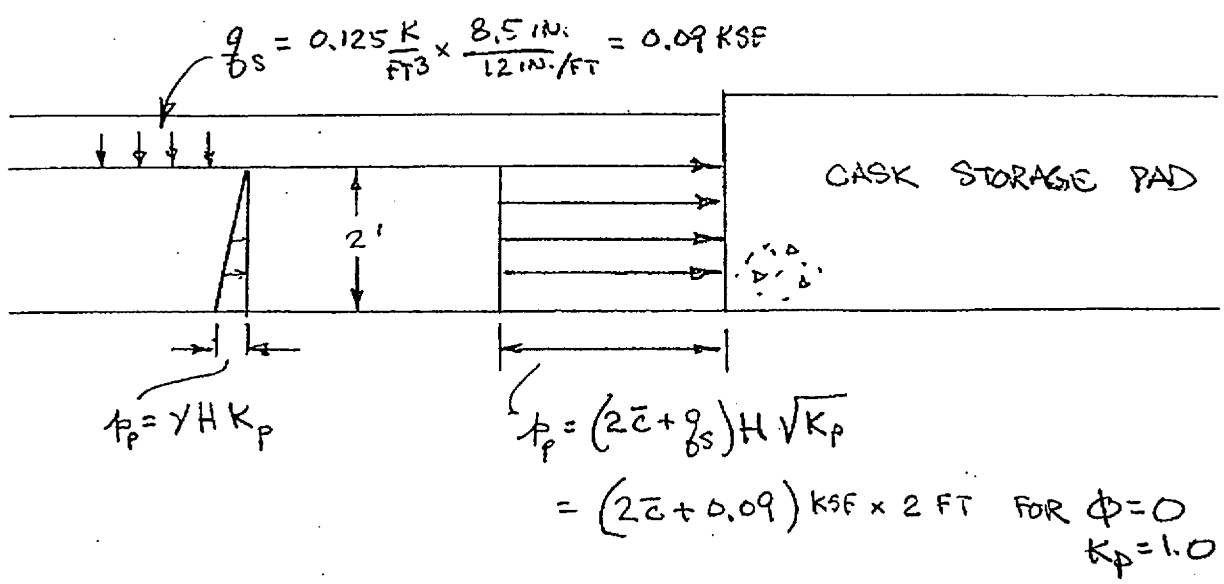
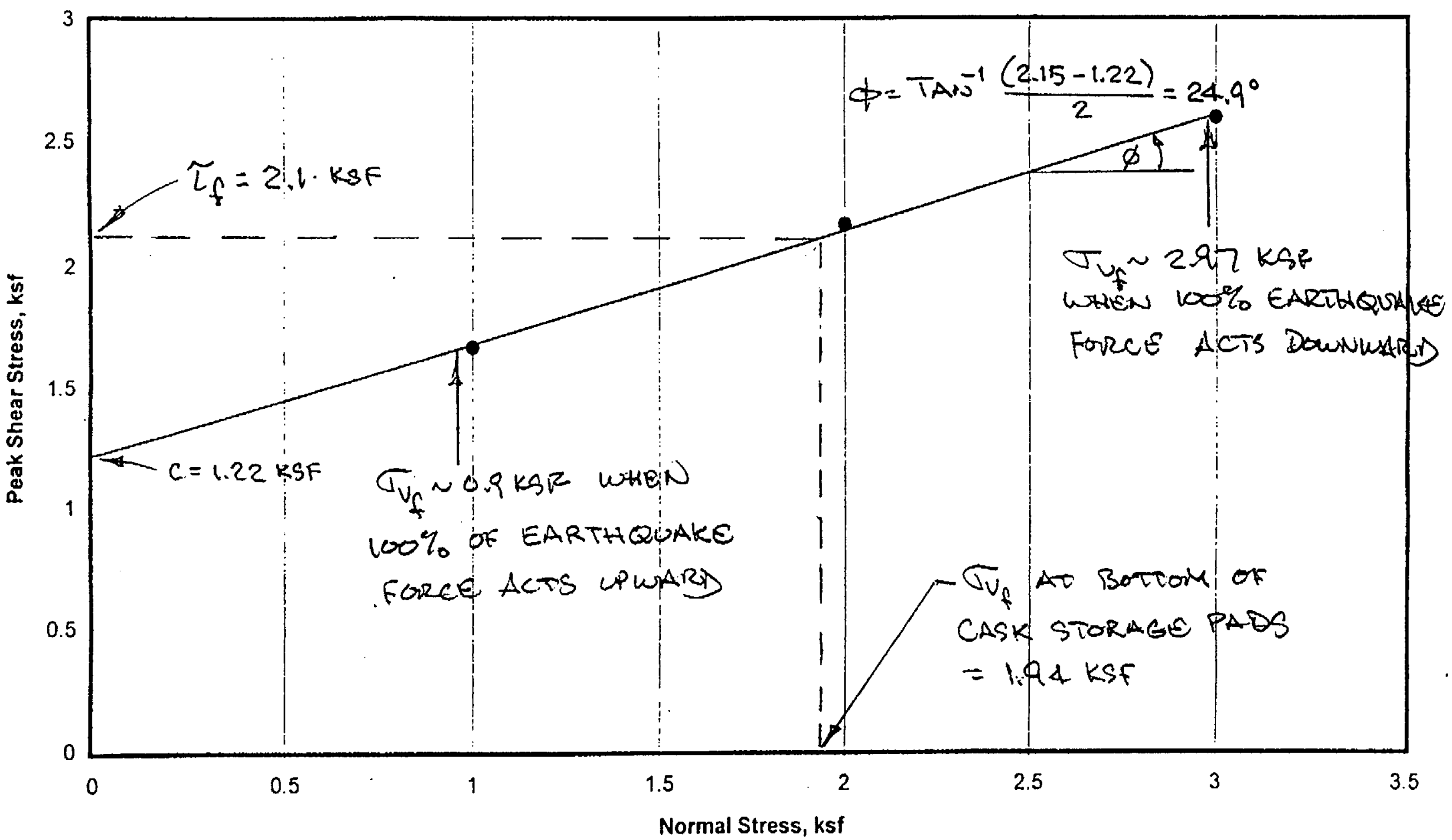


FIGURE 7

DIRECT SHEAR TEST

Boring C-2, Sample U-1C

PAD EMPLACEMENT AREA



CALC 05996.02-G(B)-05-2

p 32

ATTACHMENT C p C2
CALC 05996.02-G(B)-04-7

REF SAR APP 2A ATT 7

EXHIBIT 3
Trudeau Declaration

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 1.85 ksf (12.84 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 1.25 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 are 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 50 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 = 1.25 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 1.25.

SLIDING STABILITY AT INTERFACE BETWEEN THE SOIL CEMENT BENEATH THE PADS AND IN SITU CLAYEY SOILS

Material under and around the pad will be soil cement. In this analysis, however, the presence of the soil cement is ignored, both below the pad and adjacent to the sides of the pads, to demonstrate that there is an acceptable factor of safety against sliding of the pads if they were founded directly on the silty clay/clayey silt. 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.

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 = 125 pcf — Because of the low density of the eolian silts that will be used to construct the soil cement, it is likely that γ will be less than this value. It is conservative to use this higher value, because it 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.

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 = 3 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).

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.

$P_a = [0.5 \times 125$ pcf $\times (3\text{ ft})^2 \times 0.22] \times 67$ ft (length)/storage pad = 8,291 lbs.

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:

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

where :

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

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:

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

γ = unit weight of soil,

H = wall height, and

K_{AE} is calculated as shown above.

$\beta = \alpha = 0$

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

$\phi = 40^\circ$

Approximating $\sin(\phi - \theta) \approx 0$ and $\cos(\phi - \theta) \approx 1$

$$K_{AE} = \frac{1 - \alpha_V}{\cos\theta \cdot \cos(\delta + \theta)}$$

$$\delta = \frac{\phi}{2} = 20^\circ$$

$$\therefore \quad K_{AE} = \frac{1 - 0.695}{\cos 66.8^\circ \cdot \cos(20^\circ + 66.8^\circ)} = 13.87$$

Therefore, the combined static and dynamic active lateral earth pressure force is:

$$F_{AE\,E\text{-}W} = P_{AE} = \frac{1}{2} \times \overset{\gamma}{125\ \text{pcf}} \times \overset{H^2}{(3\ \text{ft})^2} \times \overset{K_{AE}}{13.87} \times \overset{L}{67\ \text{ft}} / \text{storage pad} = 522.7\ \text{K in E - W direction.}$$

$$F_{AE\,N\text{-}S} = 522.7\ \text{K} \times \frac{30\ \text{ft}}{67\ \text{ft}} = 234.1\ \text{K in the N - S direction.}$$

WEIGHTS

Casks: Wc = 8 x 356.5 K/cask = 2,852 K

Pad: Wp = 3 ft x 67 ft x 30 ft x 0.15 kips/ft^3 = 904.5 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

EQvp = -0.695 x 904.5 K = -629 K

EQhp = 0.711 x 904.5 K = 643 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:

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

$$T = \overset{N}{1{,}146\ K} \times \tan \overset{\phi}{0^\circ} + \overset{c}{2.1\ ksf} \times \overset{B}{30\ ft} \times \overset{L}{67\ ft} = 4{,}221\ K$$

The driving force, V, is defined as:

$$V = F_{AE} + EQhp + EQhc$$

The factor of safety against sliding is calculated as follows:

$$FS = \overset{T}{4{,}221\ K} \div \underbrace{(\overset{F_{AE}}{522.7\ K} + \overset{EQhp}{643\ K} + \overset{EQhc}{696\ K})}_{(1{,}861.7\ K)} = \mathbf{2.27}$$

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, μ = 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 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) = $\Sigma \cdot Fv$ = Wc + Wp + EQvc + EQvp

$$N = \overset{Wc}{2{,}852\ K} + \overset{Wp}{904.5\ K} + \overset{EQvc}{1{,}982\ K} + \overset{EQvp}{629\ K} = 6{,}368\ K$$

$$T = \overset{N}{6{,}368\ K} \times \tan \overset{\phi}{0^\circ} + \overset{c}{2.1\ ksf} \times \overset{B}{30\ ft} \times \overset{L}{67\ ft} = 4{,}221\ K$$

The driving force, V, is defined as:

$$V = F_{AE} + EQhp + EQhc$$

The factor of safety against sliding is calculated as follows:

$$\mathbf{FS}_{\text{Soil Cement to Clayey Soil}} = \overset{T}{4{,}221\text{ K}} \div \underbrace{(\overset{F_{AE\ E\text{-}W}}{522.7\text{ K}} + \overset{EQhp}{643\text{ K}} + \overset{EQhc_{E\text{-}W}}{2{,}212\text{ K}})}_{(3{,}377.7\text{ K})} = \underline{\mathbf{1.25\ \ (Minimum)}}$$

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 are stable with respect to sliding for this load case if the full undrained strength of the underlying soils is engaged to resist sliding.

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:

$$FS = \overset{T}{T} \div \underbrace{(\overset{F_{AE\ E\text{-}W}}{522.7\text{ K}} + \overset{EQhp}{643\text{ K}} + \overset{EQhc_{E\text{-}W}}{2{,}212\text{ K}})}_{(3{,}377.7\text{ K})} \geq 1.1$$

$$\rightarrow\quad T \geq 1.1 \times 3{,}377.7\text{ K} = 3{,}715.5\text{ K}$$

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

$$\tau = \frac{3{,}715.5\text{ K}}{30\text{ ft} \times 67\text{ ft}} = 1.85\,\frac{\text{K}}{\text{ft}^2} \times \left(\frac{\text{ft}}{12\text{ in.}}\right)^2 \times \frac{1{,}000\text{ lbs}}{\text{K}} = \mathbf{12.84\ psi}$$

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 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, 88% (= 1.85 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 and the surface of the underlying clayey soils must be 0.88. 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 of the pads. Therefore, an "engineered mechanism" is required to ensure that the full strength of the clayey soils is available to resist sliding of these pads.

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 12.84 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 (12.84 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 (12.84 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. 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 1.25 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 Figure's 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.

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 resist sliding before a sliding failure has occurred. In this case, the shear strength of the clayey soils under the pad must be reduced to the residual strength, because of the strains required to reach the full passive state. Note, the soil cement is much stiffer than normal soils; therefore, these strain levels 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. Note, the horizontal displacement of ~0.025" past the peak strength corresponds to a horizontal strain of ~1%, since the diameter of these specimens was 2.5". 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:

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

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

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

$$\textbf{FS}_{\text{Pad to Clayey Soil N-S w/Passive}} = \underbrace{6{,}526\ \text{K}}_{T_{N\text{-}S}} \div \underbrace{(\underbrace{234\ \text{K}}_{F_{AE\ N\text{-}S}} + \underbrace{643\ \text{K}}_{EQhp} + \underbrace{2{,}102\ \text{K}}_{Eqhc_{N\text{-}S}})}_{(2{,}979\ \text{K})} = \mathbf{2.19}$$

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:

$$\textbf{FS}_{\text{Pad to Clayey Soil N-S w/o Passive}} = \underbrace{4{,}221\ \text{K}}_{T_{N\text{-}S}} \div \underbrace{(\underbrace{234\ \text{K}}_{F_{AE\ N\text{-}S}} + \underbrace{643\ \text{K}}_{EQhp} + \underbrace{2{,}102\ \text{K}}_{Eqhc_{N\text{-}S}})}_{(2{,}979\ \text{K})} = \mathbf{1.42}$$

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:

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

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

$$\textbf{FS}_{\text{Pad to Clayey Soil E-W}} = \underbrace{9{,}630\ \text{K}}_{T_{E\text{-}W}} \div \underbrace{(\underbrace{522.7\ \text{K}}_{F_{AE\ E\text{-}W}} + \underbrace{643\ \text{K}}_{EQhp} + \underbrace{2{,}212\ \text{K}}_{EQhc_{E\text{-}W}})}_{(3{,}377.7\ \text{K})} = \mathbf{2.85}$$

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 0.10 kips/ft^3	=	450 K
		+10 x 30 ft x 67 ft x 1 ft x 0.10 kips/ft^3	=	2,010 K
		Total Weight	=	11,505 K

Inertial forces due to Pads + Soil Cement = 0.711 x 11,505 K = 8,180 K

Dynamic active earth pressure acting in the N-S direction = 234 K

Total driving force in N-S direction = 21,020 K + 8,180 K + 234 K = 29,434 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-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-S} = 42,210 K + 1,890 K = 44,100 K

Total driving force in N-S direction = 21,020 K + 8,180 + 234 K = 29,434 K, as calculated above.

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

$$FS_{\text{Pad to Clayey Soil N-S}} = \overset{T_{N\text{-}S}}{44{,}100\ K} \div \overset{\text{Driving Force}_{N\text{-}S}}{29{,}434} = \underline{\mathbf{1.50}}$$

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:

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

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

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

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

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.

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

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

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

$$FS_{\text{Pad to Clayey Soil N-S}} = \underset{T_{N\text{-}S}}{44{,}411\ K} \div \underset{\text{Driving Force}_{N\text{-}S}}{29{,}434\ K} = \underline{\mathbf{1.51}}$$

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 + (11,505 K x 0.711) + 234 K]

Therefore, $T_{FS=1.1}$ = 32,378 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:

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

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

A = 10 x 30 ft x 67 ft + 9 x 30 ft x 5 ft

A = 20,100 ft^2 + 1,350 ft^2 = 21,450 ft^2

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

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

$T_{N\text{-}S} = \tau_{Pad} \times 20{,}100\ ft^2 + \tau_{Soil\ Cement} \times 1{,}350\ ft^2 = 32{,}378\ K - 3{,}596\ K = 28{,}782\ K$

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

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

$\tau_{Pad} \times 21{,}000\ ft^2 = 28{,}782\ K$

$\tau_{Pad} = 28{,}782\ K \div 21{,}000\ ft^2 = 1.37$ 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,

$T_{N\text{-}S} = \tau_{Pad} \times 20{,}100\ ft^2 + \tau_{Soil\ Cement} \times 1{,}350\ ft^2 = 32{,}378\ K$

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

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

$\tau_{Pad} \times 21{,}000\ ft^2 = 32{,}378\ K$

$\tau_{Pad} = 32{,}378\ K \div 21{,}000\ ft^2 = 1.54$ 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 ~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 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:

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

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.

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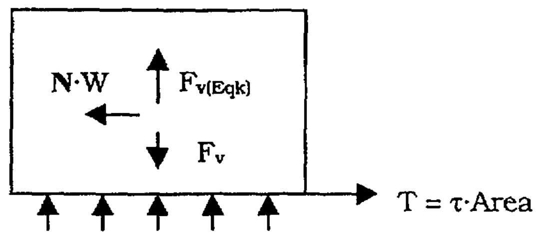
EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

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, **N**·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.

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 x 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

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

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

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

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

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

$$u_m = [V^2] / (2gN)$$

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

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

$$\Rightarrow \quad N / A = 0.417 / 0.766 = 0.544$$

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

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

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

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 WORST-CASE HYPOTHETICAL ASSUMPTION THAT CASK STORAGE PADS ARE FOUNDED DIRECTLY ON COHESIONLESS SOILS WITH ϕ = 0 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

Assuming the cask storage pads are founded directly on a layer of cohesionless soils with ϕ = 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.

FIGURE 3

DETAIL OF SOIL CEMENT UNDER & ADJACENT TO CASK STORAGE PADS

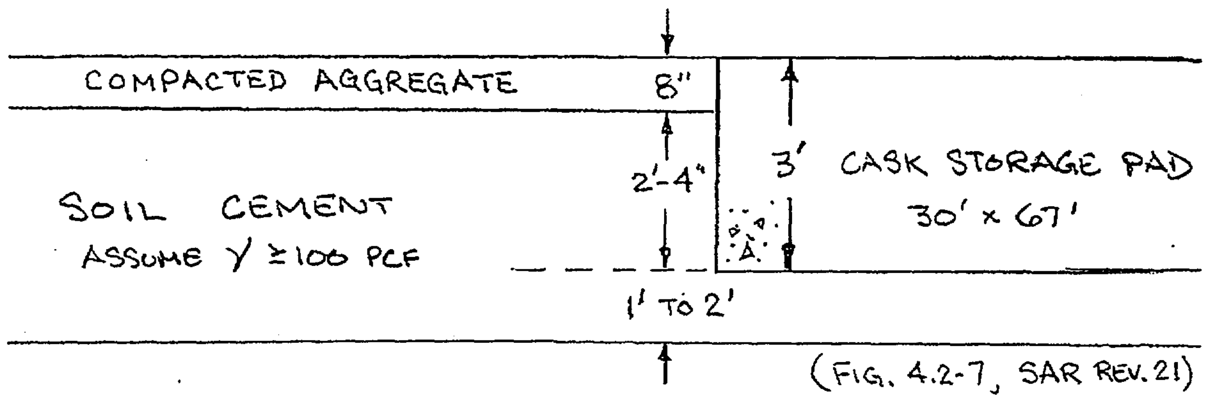
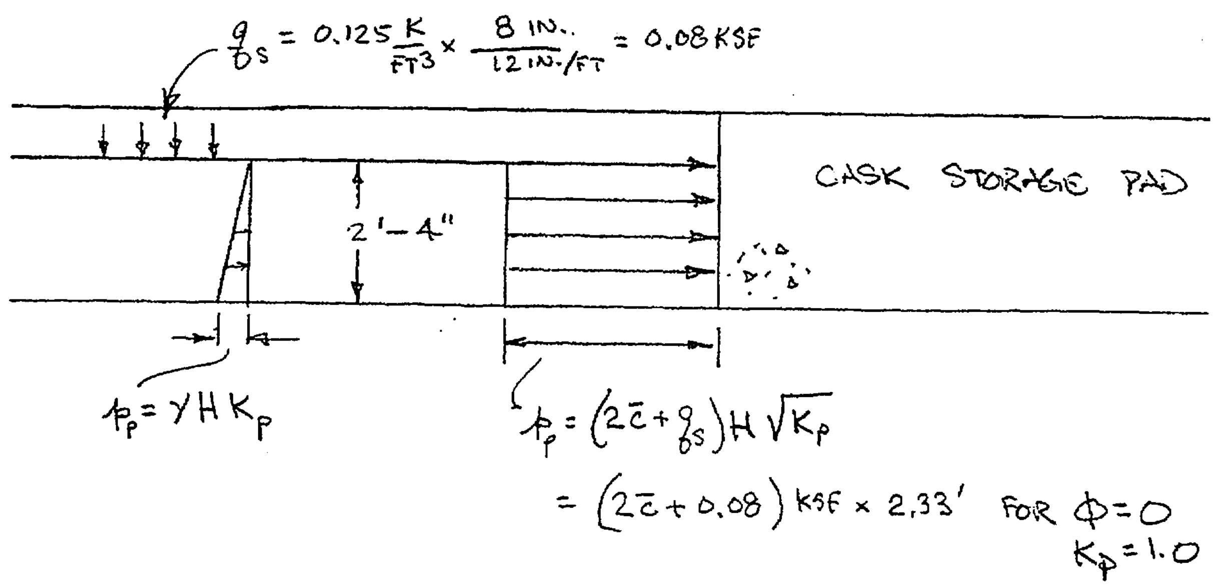


FIGURE 4

PASSIVE PRESSURE ACTING ON CASK STORAGE PADS



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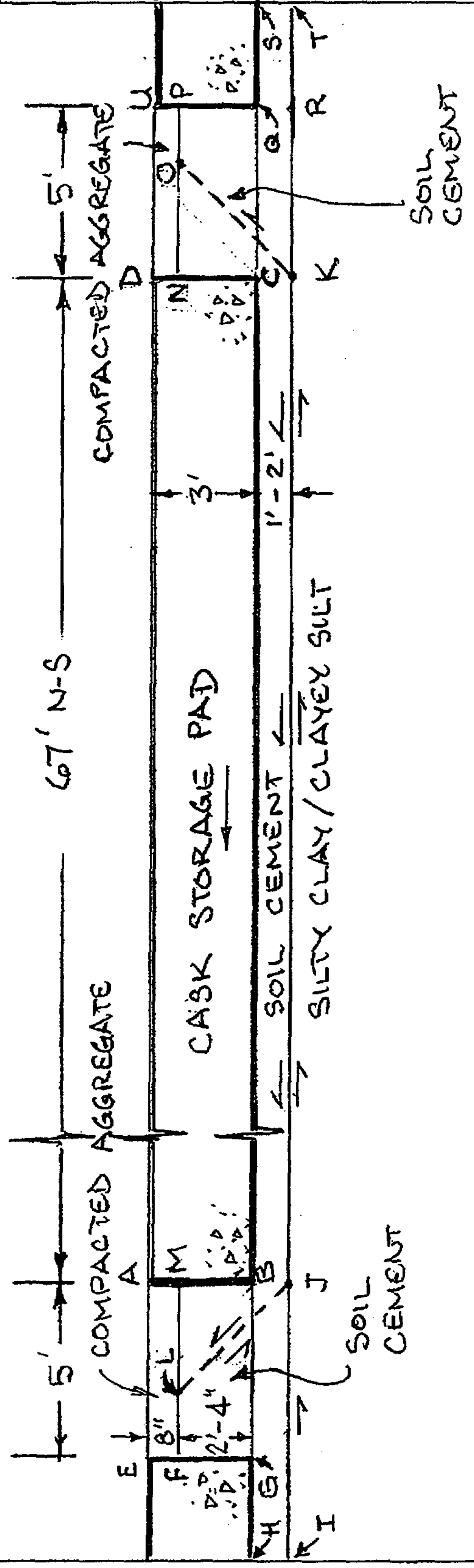

FIGURE 8

ELEVATION VIEW OF COLUMN OF CASK STORAGE PADS - LOOKING EAST

EXHIBIT 4
Trudeau Declaration

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) = $\sum F_v = F_{v\,Static} + F_{v\,Eqk}$

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

c = 1.8 ksf, as discussed above under "Geotechnical Properties."

B = 240 feet

L = 279.5 feet

The driving force, V, is calculated as follows:

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

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

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 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. As discussed above under Geotechnical Properties, $\phi = 0°$ and a shear strength of 1.8 ksf were used for the clayey soils underlying the Canister Transfer Building in determining resisting forces for the earthquake loading combinations.

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 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, $P_p = 2c$

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

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

The CTB mat is 240' wide in the E-W direction and 279.5' long in the N-S direction; therefore, the total passive force available to resist sliding is at least 240' x 90 K/LF = 26,500 K acting in the N-S direction.

Lambe & Whitman (1969, p 165) indicates that little horizontal compression, ~0.5%, is required to reach half of full passive resistance for dense sands. The eolian silts will be compacted to a dense state; therefore, assume that half of the total passive resistance is available to resist sliding of the building. Note, 0.5% of the 5 ft height of the mat + 1.5-ft deep key = 0.005 x 6.5 ft x 12 in./ft = 0.39 in. 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 results of the sliding stability analysis of the Canister Transfer Building are presented in Table 2.6-13. These results 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 (Schimming et al, 1966, Casagrande and Shannon, 1948, and Das, 1993); therefore, they represent a conservative lower-bound value of the sliding stability of the Canister Transfer Building founded on in situ silty clay/clayey silt with soil-cement backfill around the mat.

The sliding stability for Cases IIIA, IIIB, and IIIC was calculated by summing the driving forces using the SRSS rule, but without similarly summing the passive resisting forces. The sliding stability for Cases IVA, IVB, and IVC were calculated by summing both the driving forces and the passive resisting forces using the SRSS rule, which is believed to more realistically represent the actual condition. As expected, the sliding stability calculated for Case IV generally gives a higher factor of safety against sliding than those for Case III.

These results indicate that the factors of safety are acceptable for all load combinations. The lowest factor of safety was 1.13, which applies for Case IIIC, where 100% of the dynamic earthquake forces act in the N-S direction and 40% act in the other two directions. These results are all > 1.1, the minimum value required for sliding.

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.

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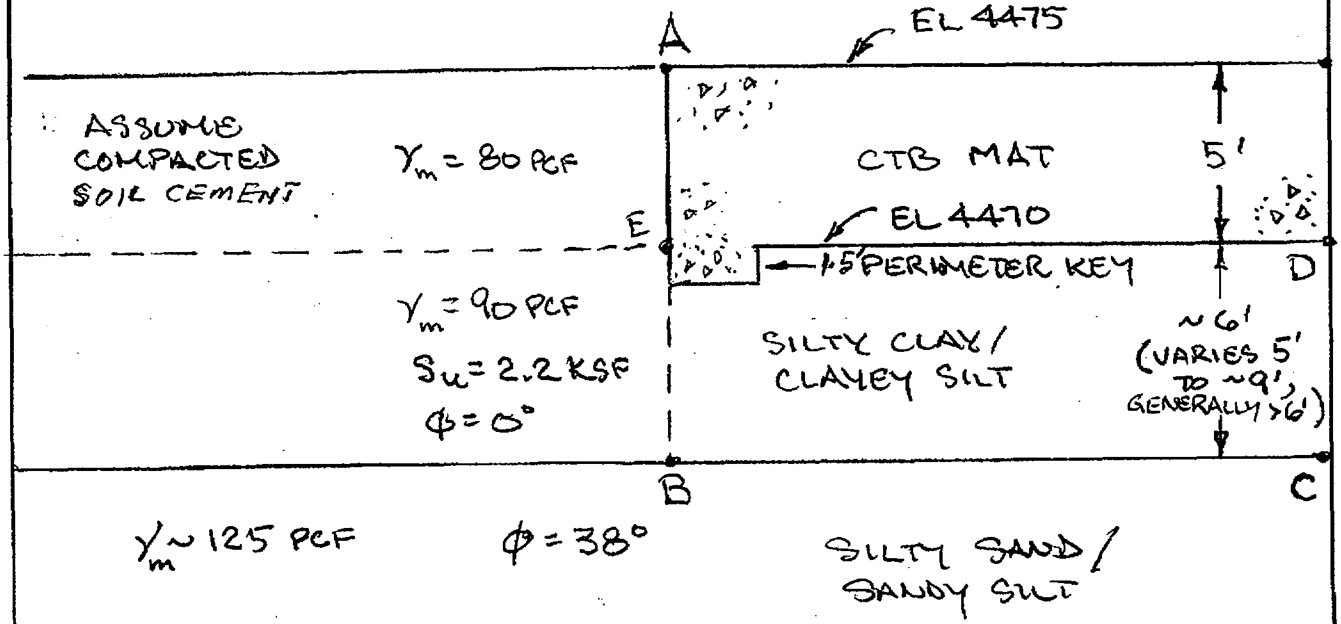
SLIDING ON DEEP PLANE AT TOP OF SILTY SAND/ SANDY SILT LAYER



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. $\Rightarrow \phi = 38°$.

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

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

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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

(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 15.

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

AREA BCDE $= 6\,FT \times 240\,FT_{E\text{-}W} = 1440 \frac{FT^2}{END}$

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

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

(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.

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

(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 NEGLIBLE.

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

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SLIDING ON DEEP PLANS

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

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

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

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

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

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

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

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

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

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

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05996.02	G(B)	13-4		

SLIDING ON DEEP PLANE

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

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

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

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

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 Resistance from Soil Cement

Joint	MASS X k-sec²/ft	MASS Y k-sec²/ft	MASS Z k-sec²/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	0.763	0.920	8,368	8,761	6,551	7,699
1	1,908.0	1,908.0	1,908.0	1.047	0.763	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	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 = 1.80 ksf			N (k)	T (k)	V (k)	FS
Earthquake Vertical Forces Acting Up	IIIA	$F_{V(stat)}$	40% $F_{H(N-S)}$	100% $F_{V(EQ)}$	40% $F_{H(E-W)}$				
		97,749	44,443	-79,779	39,999	17,970	133,173	59,792	2.23
	IIIB	$F_{V(stat)}$	40% $F_{H(N-S)}$	40% $F_{V(EQ)}$	100% $F_{H(E-W)}$				
		97,749	44,443	-31,912	99,997	65,837	133,173	109,429	1.22
	IIIC	$F_{V(stat)}$	100% $F_{H(N-S)}$	40% $F_{V(EQ)}$	40% $F_{H(E-W)}$				
		97,749	111,108	-31,912	39,999	65,837	133,173	118,088	1.19
Earthquake Vertical Forces Acting Down	IVA	$F_{V(stat)}$	40% $F_{H(N-S)}$	100% $F_{V(EQ)}$	40% $F_{H(E-W)}$				
		97,749	44,443	79,779	39,999	177,529	129,677	59,792	2.17
	IVB	$F_{V(stat)}$	40% $F_{H(N-S)}$	40% $F_{V(EQ)}$	100% $F_{H(E-W)}$				
		97,749	44,443	31,912	99,997	129,661	140,198	109,429	1.28
	IVC	$F_{V(stat)}$	100% $F_{H(N-S)}$	40% $F_{V(EQ)}$	40% $F_{H(E-W)}$				
		97,749	111,108	31,912	39,999	129,661	138,400	118,088	1.17
Soil Cement ΔF_H for q_u (psi) = 250			21,600	N/A	25,155	for FS = 2.0			

EXHIBIT 5
Trudeau Declaration

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) = $\sum F_v = F_{v\,Static} + F_{v\,Eqk}$

$\phi = 0°$ (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:

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

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 total 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 the peak strength of the clayey soils under the building.

Lambe & Whitman (1969, p 165) 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; therefore, assume that half of the total passive resistance is available to resist sliding of the building. Note, 0.5% of the 5 ft height of the mat + 1.5-ft deep key = 0.005 x 6.5 ft x 12 in./ft = 0.39 in. 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 along with the resistance provided by the peak shear strength of the clayey soils enclosed within the perimeter key at the base of the mat.

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 strains 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. Note, the horizontal displacement of ~0.025" past the peak strength corresponds to a horizontal strain of ~1%, since the diameter of these specimens was 2.5". 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 ~12.5%, since the final effective stresses under the building are ~1.5 ksf. Therefore, based of these results, conservatively assume that the peak strength of the clayey soils beneath the soil cement layer underlying the pads is reduced by 20% to reach residual strength, to account for horizontal straining required to reach a strain applicable to the full passive resistance of the soil cement adjacent to the pad.

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.

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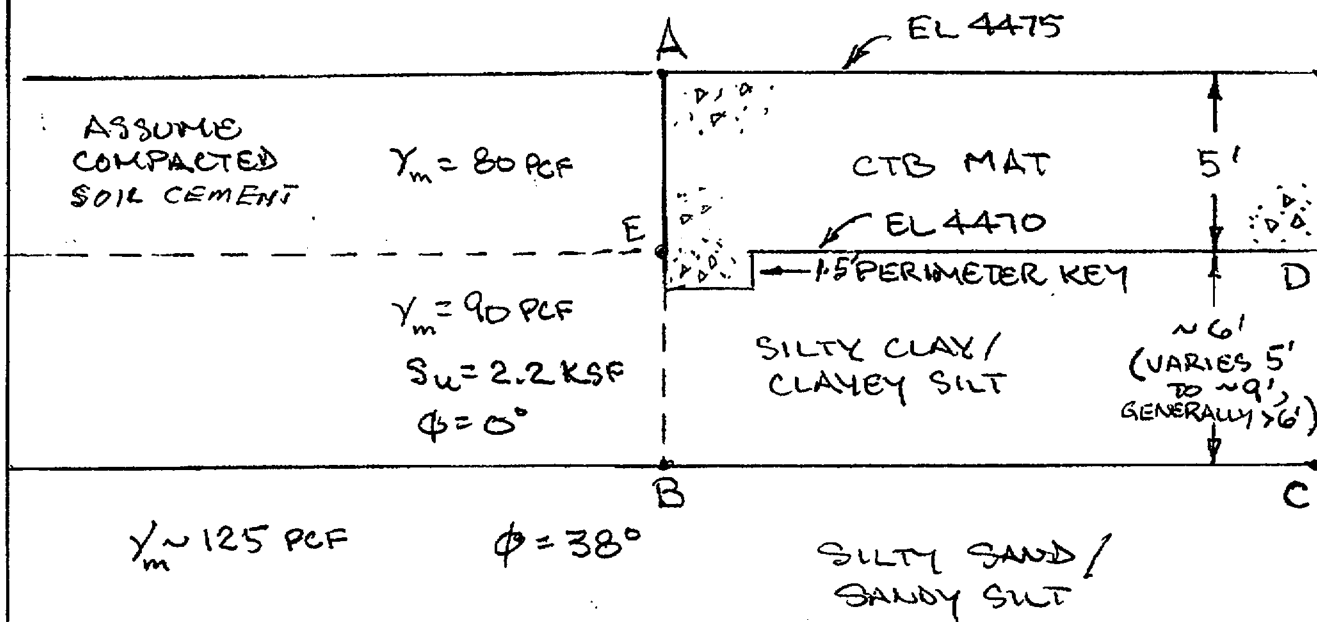
SLIDING ON DEEP PLANE AT TOP OF SILTY SAND/ SANDY SILT LAYER



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 $z = 13.8'$

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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.

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

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

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

∴ TOTAL PASSIVE RESISTANCE AVAILABLE ALONG AB

= 180 + 25.52 = 205.52 K/LF

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(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 = $6\,FT \times 240\,FT_{E\text{-}W} = 1440\ \frac{FT^2}{END}$

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

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

(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.

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

(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 NEGLIBLE.

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

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35996.02	G(B)	13-5		

SLIDING ON DEEP PLANE

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

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

$$N \tan\phi = 54{,}193K \tan 38^\circ = 42{,}340\,K$$

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

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

$\therefore$ OK

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

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

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

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

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

$\therefore$ O.K

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SLIDING ON DEEP PLANE

	N-S	VERT	E-W
<u>CASE IIIC</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

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

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

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

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	MASS Y	MASS Z	N-S a_x	Vert a_y	E-W a_z	Static F_v	Earthquake $Shear_{N-S}$	Earthquake F_v	Earthquake $Shear_{E-W}$
	$k\text{-}sec^2/ft$	$k\text{-}sec^2/ft$	$k\text{-}sec^2/ft$	g	g	g	k	k	k	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	$F_{v(Static)}$	40% $F_{H(NS)}$	100% $F_{v(Eqk)}$	40% $F_{H(EW)}$					
		97,749	44,443	-79,779	39,999		17,970	135,999	59,792	**2.27**
	IIIB	$F_{v(Static)}$	40% $F_{H(NS)}$	40% $F_{v(Eqk)}$	100% $F_{H(EW)}$					
		97,749	44,443	-31,912	99,997		65,837	135,999	109,429	**1.24**
	IIIC	$F_{v(Static)}$	100% $F_{H(NS)}$	40% $F_{v(Eqk)}$	40% $F_{H(EW)}$					
		97,749	111,108	-31,912	39,999		65,837	135,999	118,088	**1.15**
Earthquake Vertical Forces Acting Down	IVA	$F_{v(Static)}$	40% $F_{H(NS)}$	100% $F_{v(Eqk)}$	40% $F_{H(EW)}$					
		97,749	44,443	79,779	39,999		177,529	135,999	59,792	**2.27**
	IVB	$F_{v(Static)}$	40% $F_{H(NS)}$	40% $F_{v(Eqk)}$	100% $F_{H(EW)}$					
		97,749	44,443	31,912	99,997		129,661	135,999	109,429	**1.24**
	IVC	$F_{v(Static)}$	100% $F_{H(NS)}$	40% $F_{v(Eqk)}$	40% $F_{H(EW)}$					
		97,749	111,108	31,912	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			

Table 2.6-14

Sliding Stability of Canister Transfer Building Using Shear Strength Along Bottom of Plane Formed by 1.5-ft Deep Perimeter Key and Resistance from Soil Cement Using Residual Strength = 80% of Peak Strength of

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 $Shear_{N\text{-}S}$ k	Earthquake F_v k	Earthquake $Shear_{E\text{-}W}$ k
0	260.1	260.1	260.1	1.047	0.783	0.920	5,365	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	0.521	0.994	13,524	15,028	11,106	13,446
3	304.3	304.3	170.3	1.775	0.913	1.185	9,789	17,402	8,959	6,493
4	144.7	117.1	144.7	1.215	0.928	1.408	3,767	5,656	3,495	6,354
5	1.0	27.6	1.0	0.000	1.840	0.000	588	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 = 1.36				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% $F_{v,EQ}$ -79,779	40% $F_{H,EW}$ 39,999	17,970	148,586	59,792	2.49
	IIIB	$F_{v,Static}$ 97,749	40% $F_{H,NS}$ 44,443	40% $F_{v,EQ}$ -31,912	100% $F_{H,EW}$ 99,997	65,837	148,586	109,429	1.36
	IIIC	$F_{v,Static}$ 97,749	100% $F_{H,NS}$ 111,108	40% $F_{v,EQ}$ -31,912	40% $F_{H,EW}$ 39,999	65,837	148,586	118,058	1.26
Earthquake Vertical Forces Acting Down	IVA	$F_{v,Static}$ 97,749	40% $F_{H,NS}$ 44,443	100% $F_{v,EQ}$ 79,779	40% $F_{H,EW}$ 39,999	177,529	148,586	59,792	2.49
	IVB	$F_{v,Static}$ 97,749	40% $F_{H,NS}$ 44,443	40% $F_{v,EQ}$ 31,912	100% $F_{H,EW}$ 99,997	129,661	148,586	109,429	1.36
	IVC	$F_{v,Static}$ 97,749	100% $F_{H,NS}$ 111,108	40% $F_{v,EQ}$ 31,912	40% $F_{H,EW}$ 39,999	129,661	148,586	118,085	1.26
Soil Cement ΔF_H for q_u (psi) = 250			43,200	N/A	50,310	for FS_{SC} = 1.0			

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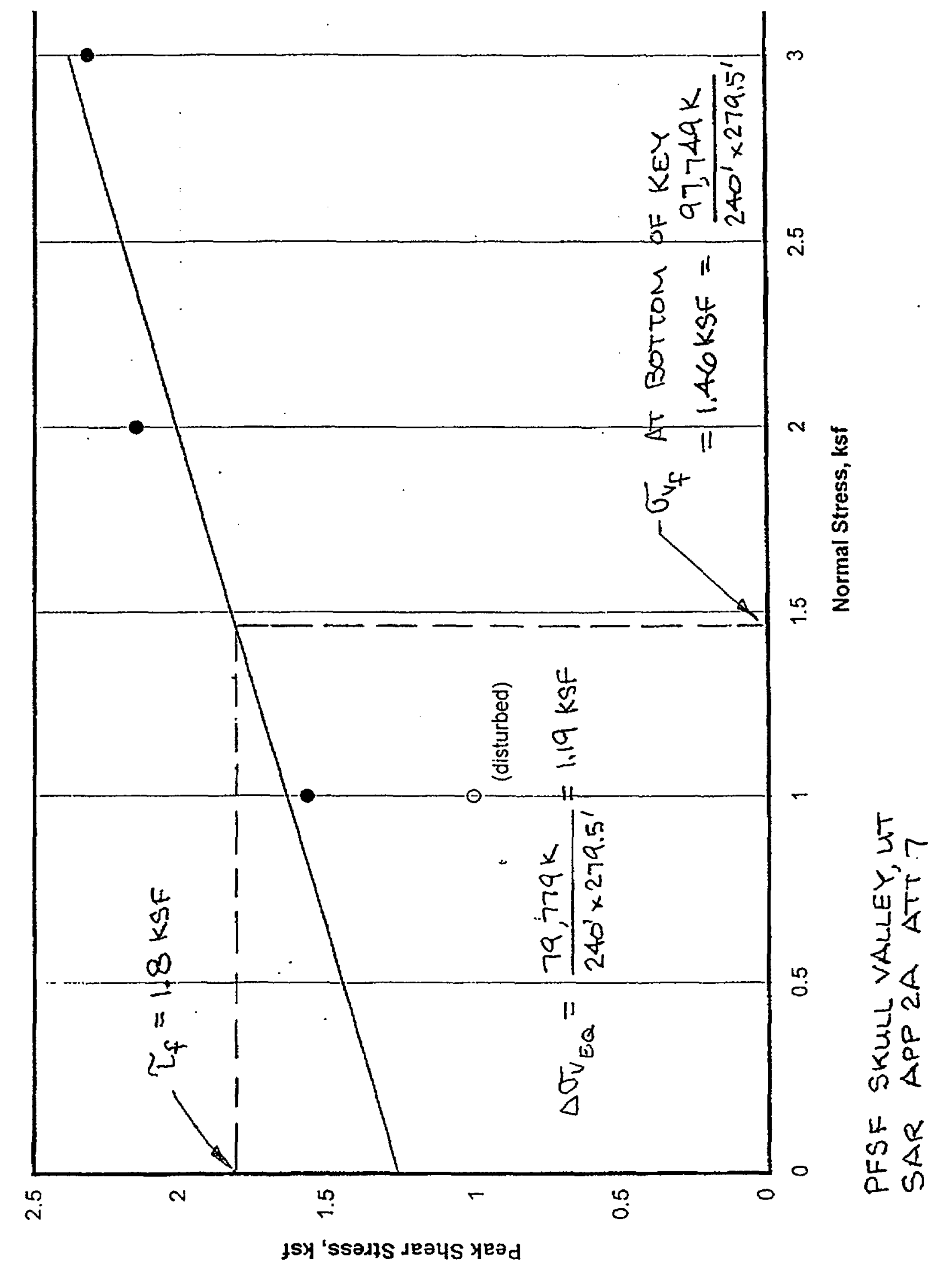
FIGURE 7
DIRECT SHEAR TEST
Boring CTB-6, Sample U-3B&C
CANISTER TRANSFER BUILDING AREA



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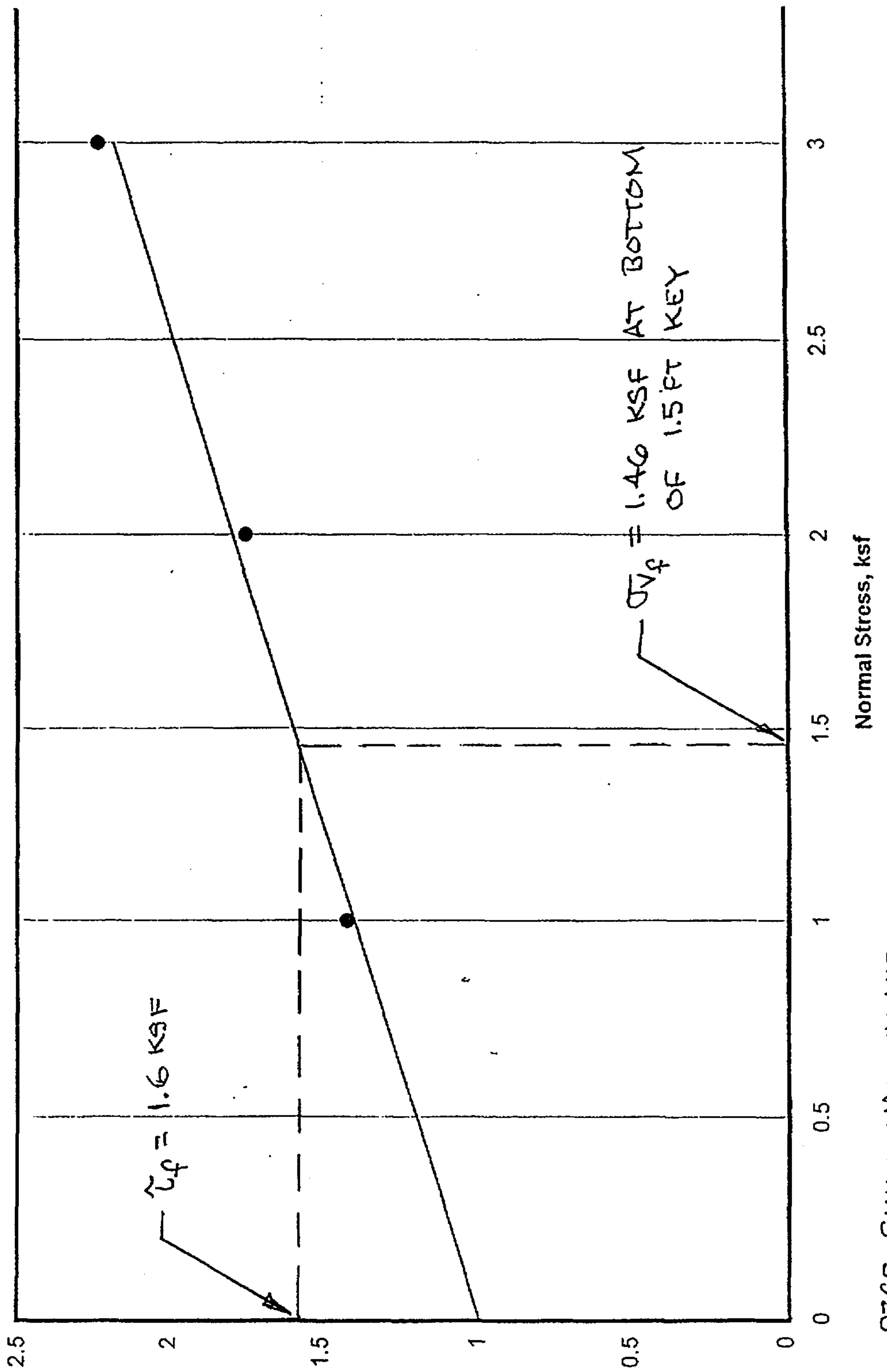

FIGURE 8
DIRECT SHEAR TEST
Boring CTB-S, Sample U-1AA
CANISTER TRANSFER BUILDING AREA

PFSF SKULL VALLEY, UT
SAR APP 2A ATT 7