May 30, 2001

UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

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

APPLICANT'S RESPONSE TO STATE OF UTAH'S REQUEST FOR ADMISSION OF LATE-FILED CONTENTION UTAH QQ

I. INTRODUCTION

Applicant Private Fuel Storage L.L.C. ("Applicant" or "PFS") hereby responds to the State of Utah's ("State") "Request for Admission of Late-Filed Contention Utah QQ (Seismic Stability)," filed May 16, 2001 ("Request"). The State asserts that proposed Contention Utah QQ ("Proposed Utah QQ") arises from newly revised design basis ground motions developed by the Applicant to incorporate soils data collected at the PFS site.[1] The proposed contention, however, does not challenge the revised seismic analyses performed by Applicant, or the design earthquake ground motions resulting from those analyses. Instead, Proposed Utah QQ alleges that the newly revised design basis ground motions have not been correctly and consistently applied to the analyses of the Canister Transfer Building ("CTB"), the spent fuel cask storage pads, and their soil cement foundations. Request at 8-11. The proposed contention further asserts that PFS has failed to prove the adequacy of the use of soil cement in the design of the CTB and the storage pads to help withstand the revised seismic loads, id. at 11-14, and also claims that PFS has not fully taken into account the properties of the underlying soils at the PFS site in determining whether the safety-related structures at the site can withstand the revised seismic loads. Id. at 14-15.

[1] PFS letter, Parkyn to U.S. NRC, License Application Amendment No. 22, dated March 30, 2001 ("LA 22").

As more fully discussed below, many of the claims raised by the State relate to the methodology used by PFS to predict the performance of the CTB and the spent fuel cask storage pads under seismic conditions. In the areas of concern raised by the State, that methodology has not changed since the analyses were first performed. The balance of the State's claims challenge the use of soil cement in the design. However, soil cement has been part of the PFS design since December 1999. Thus, the issues the State seeks to litigate in Proposed Utah QQ could and should have been raised either at the time that Contention Utah L was filed in 1997 or, at the latest, when the PFS seismic designs and analyses were modified in 1999 and 2000.[2]

Proposed Utah QQ also repeats claims that have been previously rejected in this proceeding and raises claims that are speculative or otherwise flawed. Thus, the proposed contention fails to meet the standards for admissibility in NRC proceedings.

II. BACKGROUND

A. RELEVANT FACTS

On November 23, 1997, the State filed Contention Utah L, which challenged aspects of the seismic design of the PFS facility. A month later, on December 23, 1997, the State filed a "Request for Consideration of Late-Filed Contentions EE and FF" (hereinafter "EE Request"). The State's late-filed EE Request sought to challenge the seismic analyses of the spent fuel storage casks, their pads and foundations. On April 22, 1998, the Atomic Safety and Licensing Board ("Licensing Board" or "Board") admitted Utah L into this proceeding and rejected Utah EE as impermissibly late. Private Fuel Storage, L.L.C. (Independent Spent Fuel Storage Installation), LBP-98-7, 47 NRC 142, 191, 253 (1998).

On December 16, 1999, Applicant filed License Amendment No. 8 ("LA 8"). This amendment incorporated soil cement into the design of the PFS facility, to be used beneath the

[2] Exhibit A hereto is a table listing the claims raised in Proposed Utah QQ and the date on which the design or analysis features challenged in each claim were first identified by PFS. The Exhibit demonstrates that the claims raised in Proposed Utah QQ address design or analysis features of which the State had knowledge a year or more prior to the filing of Proposed Utah QQ.

spent fuel cask storage pads. The documentation filed with LA 8 included specifications for the soil cement, including the use of American Concrete Institute ("ACI") standards to govern the procedures for placement and treatment of the soil cement.[3] 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.)[4] Apart from the use of soil cement, the cask storage pad sliding stability calculations utilized the same methodology and assumptions as those in the initial calculation package dated July 14, 1997.[5]

On June 23, 2000, Applicant submitted License 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] This amendment also incorporated into the CTB design the use of a "shear key" (a circular structure intended to minimize the potential sliding of the CTB in the event of an earthquake).[7] In terms of methodology relevant to Proposed Utah QQ, the revised calculations submitted with LA 13 continued to apply the same assumptions and methods as the previous versions of the calculations. Also, the calculations for the CTB were based on the same general methodology used for the pads.[8]

On December 30, 2000, PFS moved for summary disposition of Utah L. The State filed a response in opposition to the motion on January 31, 2001. State of Utah's Response to Applicant's Motion for Summary Disposition of Utah Contention L ("Utah L Summary Disposition Response.") In its response, the State sought to broaden the scope of Utah L by raising many of

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3 SAR at 2.6-91 (Rev. 8). See Exhibit A, item 15 for further details.

4 See Exhibit A, item 1.

5 PFS letter, Parkyn to NRC, Submittal of Calculation Package, dated July 14, 1997, Exhibit A, item 3.

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, item 12.

7 SAR Sections 2.6.1.11.2 and 2.6.1.12.2, Rev. 13. See Exhibit A, item 25.

8 For example, 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 (e.g., rigidity of the basemat or storage pad) that are being challenged by the State in Proposed Utah QQ. See Attachments 1 and 2 to Exhibit A.

the same issues which it has now included in Proposed Utah QQ, including for example attacks on the seismic stability calculations for the CTB and the cask storage pads.[9] The State's response to the motion for summary disposition of Utah L also raised issues about the adequacy, integrity and durability of soil cement, the effectiveness of the CTB shear key, and a variety of other arguments identical to those now asserted in Proposed Utah QQ.[10] While raising these issues was inappropriate in the context of Utah L because they were outside the scope of the admitted contention, by seeking to raise these arguments in January 2001 the State demonstrated that the claims are independent of the modifications incorporating the revised design basis ground motion, which were to be filed by PFS several months later.

On March 30, 2001, PFS filed LA Amendment 22 which provided revised design basis ground motions, derived from the use of additional soils data.[11] 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 changes, including use of soil cement, or revisions to storage pad analyses, soils analyses, soil-cement design calculations/analyses, and Holtec site-specific cask analyses."[12] On May 16, 2001, the State filed its request to admit Proposed Utah QQ.

B. OVERVIEW OF CLAIMS RAISED IN PROPOSED UTAH QQ

In Proposed Utah QQ, the State has asserted three general types of challenges to the PFS seismic design. As noted earlier, these general challenges are: i) that the calculations used to determine the behavior of the CTB and the storage casks and pads in the presence of seismic forces are oversimplified, incomplete or inaccurate; ii) that the soil cement whose use is proposed for both the CTB and the cask storage pads has unknown properties, and may not behave

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9 Utah L Summary Disposition Response at 20-23; State of Utah's Statement of Disputed and Relevant Material Facts ("Utah L Statement of Facts"), ¶¶90-98.

10 Utah L Summary Disposition Response at 15-17; Utah L Statement of Facts ¶¶43-51, 62-64, 72-73, 76-77.

11 PFS letter, Parkyn to NRC dated March 30, 2001 and attachments thereto.

12 Memorandum and Order (Schedule for Late-Filed Submissions Regarding License Application Amendment and Page Limit Extension) (April 26, 2001) ("April 26, 2001 Order"). The April 26, 2001 Order also authorized the State to submit by May 7, 2001 any late-filed issue statements with regard to the changes to the probabilistic seismic hazard analyses arising from LA 22. The State failed to submit any such issues.

in the way that the PFS analyses and designs assume;[13] and iii) that the properties of the soils at the site have not been properly taken into account in the design analyses and calculations.

1. Design Calculations Incorporating Ground Motions

The State challenges as oversimplified and non-conservative the methodology applied by PFS in its revised calculations that incorporate the new design ground motions. Request at 8-11. The State views as oversimplified certain assumptions made in the calculations, such as the rigidity of the CTB mat and storage pads, the behavior of structures when subjected to ground motions, and the characteristics of those ground motions. Id. at 8-9. The State argues that PFS has failed to properly take into account out-of-phase ground motions, the potential multiplicity of seismic wave patterns that will arrive at the site, and the need to utilize multiple time histories in both the CTB and storage pad/cask response calculations. Id. at 9-11. As discussed below, these issues are not being raised for the first time in Proposed Utah QQ, nor do they result from the revisions to the design ground motions. Rather, the same issues were propounded over three years ago in Proposed Contention EE. The State also raised the same issues early this year, in its attempt to defeat the Applicant's motion for summary disposition of Utah L (a matter currently pending before the Board). Thus, the inclusion of these issues in Proposed Utah QQ represents the State's third attempt to find a way to litigate them.[14]

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[13] The State, and its witness James Mitchell, seek to draw a distinction between "soil cement" and "cement treated soil." See Request at 3, n.2; Declaration of Dr. James K. Mitchell ("Mitchell Dec.") ¶ 12. However, to the extent such a distinction exists, the State has failed to raise a litigable contention relating to it. All that Dr. Mitchell alleges is that if the mixture of soil and cement to be used by PFS "is not a true soil-cement, then the durability of the cement-treated soil may be an issue, because the wet-dry, freeze-thaw exposure may be significant for this site." Mitchell Dec. ¶12, emphasis added. Such a hypothetical, speculative concern does not give rise to a litigable contention in a Commission proceeding. LBP-98-7, 47 NRC at 180.

[14] In its April 26, 2001 Order, the Board requested that in their filings the parties discuss the impact, if any, of the admission of any late-filed contention on the matters currently pending before the Board in connection with the PFS dispositive motion on contention Utah L. The answer is that there should be no impact of Proposed Utah QQ on the summary disposition of Utah L because the common issues between the two sets of contentions are certain claims, not part of Utah L as admitted by the Board, that the State raised in its Utah L Summary Disposition Response an attempt to defeat summary disposition. These additional claims, which relate only to Basis 3 or Utah L and which are subject to a pending motion to strike by Applicant, should be excluded from Utah L as improperly raised, and from Proposed Utah QQ because of untimeliness. The fact that Holtec's methodology has not changed since 1997 is evidenced by Exhibit A, Attachment 3.

2. Soil Cement

The State asserts that the proposed use of soil cement by the Applicant may not achieve its intended purpose with respect to the seismic performance of the CTB and the storage casks and their pads. Request at 11. In support of this assertion, the State provides a laundry list of reasons why this may be so. These assertions fall into two categories, those that attack the properties of the soil cement (e.g., that it will not provide as much resistance against sliding as attributed in the seismic calculations), and those that suggest that, even if the soil cement has the properties intended, the material will degrade over time, losing its beneficial effects.

The first category of challenges – that the soil cement will not have the properties intended – asserts that there are not "sufficient evaluations, testing, calculations and design to demonstrate that the cement-treated soil will perform its intended functions, both under seismic loading and long-term operational conditions." Request at 6. This line of attack is speculative, since it presumes that PFS will be unable to demonstrate that the soil cement to be used will meet its required design function. In addition, as further discussed below, the State's challenge to the use of soil cement is untimely, since the State has been on notice since at least December 1999 of PFS's intention to use the material, and has also been aware since that time of the manner in which the soil cement will be designed and applied.[15]

The second category of challenges asserts that the soil cement will not behave as intended due to environmental factors such as the effects of out-of-phase earthquake motions, the settling of the CTB and the storage pads, the weight of equipment, weather (including freeze/thaw cycles and wet/dry conditions), drying and curing, delamination or debonding along the soil cement lift interfaces, chemical effects of salts and sulfates that may be present in the soil that is mixed with the cement, and the long-term performance of the soil cement material. Request at 11-14. All of these concerns, as more fully discussed below, are intrinsic to the use of soil cement, and as such should have been raised a year and a half ago, when PFS first announced its plans to use soil ce-

[15] See Exhibit A, item 15.

ment at the site. In addition, a number of these postulated problems are mere speculation and as such do not provide the basis for an admissible contention.

3. Soil Properties

The State asserts that there are a number of characteristics of the soils underlying the PFS site that should have been taken into account in the design analyses, but were inadequately considered or ignored altogether. Request at 14-15. For example, the State claims that PFS has potentially overestimated the sliding resistance provided by the clayey-silt and silty-clay soils underlying the CTB and storage pads, because it has not considered the effects of: adhesion and potential water content changes during cement-treated soil placement, and other long-term moisture content changes; seismically generated pore pressures on the soil's shear strength during earthquake loading; and partial mobilization of undrained shear strength free-field ground motion. Id. Likewise, the State claims that PFS has not demonstrated that the applied design shear strength value is representative of actual conditions. Id. As discussed below, these claims are belated and speculative, and raise no issues that need to be considered.

III. LEGAL STANDARDS

A. STANDARDS FOR ADMISSION OF LATE-FILED CONTENTIONS

Pursuant to 10 C.F.R. § 2.714(b)(1), late-filed contentions are admissible only if a balancing of 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). Although the balancing test must take into account all of the factors, there is no requirement that the same weight be given to each factor. Houston Lighting and Power Company (South Texas Project, Units 1 and 2), LBP-82-91, 16 NRC 1364, 1367 (1982), citing South Carolina Electric and Gas Company (Virgil C. Summer

Nuclear Station, Unit 1), ALAB-642, 13 NRC 881, 895 (1981).

The Board has already ruled in this proceeding (when it dismissed Proposed Contention Utah EE, in which the State sought belatedly to introduce some of the same issues raised in Proposed Utah QQ) 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). LBP-98-7, 47 NRC at 208. In the present case, the State fails to show good cause for its extreme lateness and has not made a compelling showing on the other four factors. Therefore, Proposed Utah QQ fails to satisfy the admissibility standards of 10 C.F.R. § 2.714(a)(1).

B. SUBSTANTIVE STANDARDS FOR ADMISSION OF CONTENTIONS IN COMMISSION ADJUDICATORY PROCEEDING

In order to be admissible, a proposed contention raised by an intervenor in an NRC proceeding must include: a specific statement of the issue of law or fact to be raised or controverted; a brief explanation of the bases of the contention; and sufficient information to show a genuine dispute with the applicant on a material issue of law or fact. 10 C.F.R. § 2.714(b)(2); Georgia Institute of Technology (Georgia Tech Research Reactor), CLI-95-12, 42 NRC 111, 117-18 (1995). Many of the issues raised by the State, even if they met the requirements for consideration of late-filed contentions (which they do not), would not be admissible because they lack the requisite specificity. In addition, some of the State's claims assume that the Applicant would not comply with licensing commitments or Commission regulations, which this Board has already indicated is not an acceptable basis for a contention. Private Fuel Storage, L.L.C. (Independent Spent Fuel Storage Installation), LBP-00-35, 52 NRC 364 (2000), rev. denied, CLI-01-09 (2001). Likewise, the Commission has reiterated that "the NRC does not presume that a licensee will violate regulations whenever the opportunity arises." Private Fuel Storage, L.L.C. (Independent Spent Fuel Storage Installation), CLI-01-09, 2001 NRC LEXIS 41, at *5 (March 12, 2001) (citation omitted). For these reasons, Proposed Utah QQ also fails to satisfy the admissibility standards.

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

In filing Proposed Utah QQ, the State belatedly tries to raise issues of which it was (or should have been) aware long ago, and seeks to resurrect issues that the Board has already dismissed. The State should not be allowed to remedy its past failures to provide an admissible contention by seizing upon PFS's amendments to its License Application. Despite trying to couch its claims in terms of LA 22 changes, the allegations the State makes in Proposed Utah QQ do not arise from, nor are they dependent upon, the amendment submitted by PFS. The State has failed to satisfy the crucial good cause requirement for admissibility of belated, new contentions, and has provided nothing relating to the other factors in 10 CFR § 2.714(a) that would overcome its failure to satisfy the good cause requirement.

A. THE CLAIMS RAISED IN PROPOSED UTAH QQ ARE UNJUSTIFIABLY LATE

1. Design Calculations Incorporating New Ground Motions

a. General Methodological Concerns Applicable to the CTB, Storage Casks, and Pads

The State attacks PFS's seismic design calculations for the CTB (Calc. G(B)-13, Rev. 4) and the storage pads (*Multi Cask Response at PFS ISFSI from 2000-yr Seismic Event*, Rev. 2; G(B)-04, Rev. 7) for oversimplifying or failing to take into account certain site features. Request at 8-11. Each of these alleged oversimplifications or omissions has been part of the calculations for over a year and a half, and the State's attempts to raise them now cannot be justified.[16]

For example, the State argues that the assumption that the storage pads and the CTB mat will remain rigid during an earthquake is erroneous. Request at 6-8. This assumption was part

[16] In its "good cause" explanation of the failure to raise issues with regards to the design calculations, the State actually concedes that "[i]n its revised calculation, Holtec uses many of the same incorrect assumptions as it did in its original analyses (e.g., assume the casks will slide in a controlled manner during an earthquake) as does Stone & Webster in its dynamic analyses of the CTB and storage pads." Request at 16. The State's attempt at an explanation for the failure to raise the issues earlier is that "the issues enumerated in Utah QQ relate to the increase in design basis ground motion, Amendment 22 and related calculations." Id. at 17. However, as Exhibit A clearly shows, that is not the case.

of the calculation package submitted for the storage pads in July 1997,[17] and for the CTB in September 1999.[18] The calculations treated the storage pads and the Canister Transfer Building base mat as rigid bodies, both before and after soil cement became part of the design. Thus, this assumption is independent of the revised design basis ground motion, and could have been challenged by the State one and a half to three years ago.

Similarly, the State alleges that the seismic calculations for the CTB and the storage casks and pads are not conservative because they do not consider different patterns of seismic waves (such as inclined waves), and are based on only a single time history. Request at 6-10. However, the methodology of the calculations has remained the same with respect to the assumed pattern of seismic waves and the use of a single time history since the calculations for the stability of the storage pads and the CTB were submitted in 1997 and 1999 respectively.[19] The State likewise alleges deficiencies in the modeling of possible interactions between the pads and the foundations, including failure to take into account possible out-of-phase motions of the structures with respect to their foundations. Request at 9-10. This feature of the analyses, however, has been part of the seismic calculations since 1997.[20]

These criticisms are all directed at the methodologies used by PFS, and could have been raised when the calculations were first provided to the State. The original calculations did not include a consideration of the matters identified by the State (because they did not need to be considered), nor were these matters included in the calculations submitted when soil cement was

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[17] Holtec Report HI-2012640 submitted by letter from Parkyn to NRC, "Submittal of Calculation Package", dated July 14, 1997. See Exhibit A, item 3.

[18] SWEC Calculation No. 05996.02-G(B)-13, Rev. 1, "Stability Analyses of the Canister Transfer Building Supported on a Mat Foundation" submitted by letter from John Donnell to the U.S. NRC dated September 8, 1999. See Exhibit A, item 7.

[19] LA 13, June 23, 2000; SAR Section 8.2.1.2, Rev. 13 (Holtec Report No. HI-992277, Multi-Cask Seismic Response at the PFS ISFSI, From 2000-Year Seismic Event, Rev. 0, Aug. 20, 1999). See Exhibit A, items 4,5. The calculations were submitted in two letters. Holtec Report No. H1-992277 was submitted in a letter to the NRC from J. Donnell on August 27, 1999. The cask stability analysis calculation was submitted in a letter to the NRC from J. Donnell on September 9, 1999.

[20] See Exhibit A, item 6.

added to the design of the storage pads. It was not until now, however, that the State chose to submit a contention regarding the methodologies used in these seismic design calculations. The State should not be permitted at this late date to raise issues of which it had notice for years.

b. Storage Cask and Pad Specific Concerns

In addition to concerns that are generally applicable to the methodology of the analyses of both the CTB and the cask storage pads, the State raises claims that are specific to the methodology used in the analyses of the storage casks and pads. Most of these were previously raised, and rejected by the Board, in Proposed Contention EE; they are discussed in Section IV.B below. One claim, however, is being raised for the first time in Proposed Utah QQ: the failure of the Holtec calculation to take into account the natural frequency of the cask-pad-soil-cement system. Request at 10. Again, this allegation does not arise from design changes made to accommodate the revised seismic ground motions and is independent of those design changes, but is instead an attack on a Holtec cask tipover calculation that the State has had since June, 2000.[21] That calculation's methodology has not changed in its treatment of the natural frequency of the cask-pad-soil cement system. If the State disagreed with this assumption, the appropriate time to raise a claim would have been when it first received the calculation.

c. CTB Specific Claim[22]

Another claim in Proposed Utah QQ involves the alleged ineffectiveness of the CTB shear key in preventing sliding.[23] The shear key has have been part of the CTB design since June 2000.[24] Therefore, the adequacy of the CTB shear key should be rejected as untimely.

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[21] Id., item 14.

[22] Most of the claims raised in Proposed Utah QQ against the design of the CTB raise the same issues as claims involving the cask storage pads, and are dealt with in the discussion in subsection IV.1.a above.

[23] Bartlett Dec. ¶22.

[24] SAR Rev. 13, Sections 2.6.1.11.12 and 2.6.1.12.2; License Application Amendment No. 13, dated June 13, 2000; Calculation G(B)-4, Rev. 6, submitted June 19, 2000. See Exhibit A, item 25.

2. Soil Cement Issues

The State seeks to raise in Proposed Utah QQ a number of issues relating to the use of soil cement at PFS.[25] The issues relating to soil cement should have been raised when the State became aware of the use of soil cement as part of the PFS design on December 16, 1999.[26] There can be no good cause for the State's delay in filing contentions focusing on the intrinsic adequacy and durability of soil cement.

The State's raises the following issues regarding the use of soil cement:

1. The behavior of the soil cement under tensile and bending stresses and out-of-phase motion (Request at 12);
2. The possibility and effect of cracks forming in soil cement due to settling of the CTB or storage pads (Request at 9, 12);[27]
3. The propagation of cracks in the soil cement away from the CTB or pads as a result of differential settling (Request at 9, 10);
4. Delaminating or debonding along a soil cement lift interface (Request at 12-13);
5. Cracking or degrading of soil cement due to weight of equipment (Request at 13);
6. Long term degradation of the performance of the soil cement due to weather effects such as freeze/thaw cycles or wet/dry conditions (Request at 13-14); and
7. Cracking or degrading due to curing and drying (Request at 13).

Each of these issues could have been raised in response to the SAR Revisions (LA 8, 9 and 13) in which the use of soil cement was introduced.. For example, the State's claims regarding the long-term behavior of soil cement under the pads and the CTB are identical to those

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[25] The State does not claim, however, that the use of soil cement has given rise to new problems or issues. Rather, the soil cement concerns raised in Proposed Utah QQ address the adequacy of the soil cement treatment to solve existing design problems. In addition, the alleged soil cement problems raised by the State do not relate to the new design ground motions, and would have existed whether or not the seismic ground motions were modified. In fact, the alleged problems were raised by the State last January its opposition to summary disposition of Utah L. See, e.g. Utah L Statement of Facts ¶¶47-51, 62-64.

[26] 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 (see Exhibit A, item 15). As the State's own witnesses admit, the use of soil cement around the CTB invokes exactly the same concerns as the use of soil cement around the storage pads. See, e.g., Bartlett Dec.¶12; Mitchell Dec. ¶9.

[27] Factors the State raises as possible causes of such cracking include delamination or debonding (Response at 12-13), drying and curing (id. at 13), vehicle loads (id.), weather and other long-term wear (id. at 13-14), and soil chemistry (id. at 14); Bartlett Dec. ¶¶14-15, 18. As noted above, these issues were also raised in the State's opposition to the dismissal of Utah L.

raised in the State's opposition to the dismissal by summary disposition of Utah L, focusing on mechanisms that could theoretically cause cracks in soil cement, such curing, shrinkage, moisture, freeze/thaw and wet/dry cycles and the weight of the structures (i.e., the CTB and pads themselves) or equipment (e.g., the canister transport vehicle and the casks).

The State's attacks on the use of soil cement around the CTB closely parallel the State's contentions regarding the cask pads. For example, the State attacks Holtec's calculations for casks and pads as well as the calculations for the CTB based on an alleged failure to account for "tensile and bending stresses, and . . . cracked conditions, and separation of cement-treated soil" from the pads or the CTB. Request at 9-11 (pads); id. at 8-9 (CTB); id. at11-14 (both). Thus, the State raises no new issues regarding the behavior of soil cement in relation to the CTB. Instead, the State repeats the same claims it previously asserted in Utah EE with respect to the storage pads. Therefore, since the State lacks good cause for failing to raise these issues with respect to the storage pads when it was made aware of the use of soil cement in the storage pad designs, it also lacks good cause for raising these issues now with respect to the CTB.[28]

3. Soil Properties

Proposed Utah QQ raises a number of issues relating to the properties of the underlying soils at the PFS site as they relate to the assumptions in the seismic design calculations. The State asserts that: (i) PFS may have overestimated the sliding resistance of the soil underlying the CTB and the storage pads (Request at 14-15); (ii) PFS's seismic design calculations may be affected by adhesion and water content in the underlying soil during the placement of soil cement (id. at 15); (iii) the subsurface soils have not been properly characterized (id. at 14); and (iv) the chemical properties of the soil may undermine the intended function of the soil cement (id.) The soils under the PFS site have not changed since the PFS license application was filed in 1997,

[28] The State's "good cause" argument for its belated raising of soil cement issues is that, previously, soil cement was used as a construction cost-saving measure, whereas now it is a structural design element. Request at 16. This is patently incorrect. Starting with License Amendment 9, soil cement has been credited in the sliding stability calculations as providing a factor of safety against sliding. See Exhibit A, item 1.

and these issues could have been raised then or, to the extent they refer to interactions between the site soils and the soil cement, when the latter was introduced into the design in 1999.[29] Likewise, the calculations that the State criticizes for not taking into account these effects have not changed in this regard since they were originally submitted.[30] Likewise, the chemical properties of the soil and their possible interaction with soil cement should have been raised when the use of soil cement was proposed.

The State also questions the validity of PFS's assumptions regarding the sliding resistance provided by the soil underlying the CTB and the storage pads. Request at 14-15. These assumptions are contained in calculations that were made available to the State in December 1999.[31] As such, these issues could have been raised eighteen months ago.

B. THE STATE IMPROPERLY SEEKS TO RAISE CONTENTIONS THAT HAVE BEEN PREVIOUSLY REJECTED BY THE BOARD

As noted above, the claims that the State makes in Proposed Utah QQ could have been raised many months – or years – ago. This, however, is only half the story. The State also raises arguments that have already been rejected by the Board. The State's attacks on the seismic stability of the storage casks and the underlying pads were rejected by the Board more than three years ago when first raised by the State; [32] the State's challenges to the seismic analyses of the CTB are aimed at the same methodological aspects of the analyses as those rejected in relation to the casks and pads.[33]

The State filed its EE Request") on December 23, 1997. Proposed Contention EE focused on the stability of the storage casks and the concrete pads underlying them. In it, the State asserted that the "Holtec analysis is inadequate to support the safety of the Applicant's proposed

[29] See Exhibit A, items 29-33. The State does not even attempt to make a "good cause" argument for its failure to raise before now issues relating to the site soil properties.

[30] See Exhibit A, items 32, 33.

[31] SAR 2.5.2; Rev. 3; License Application Amendment No. 8. See Exhibit A, item 29.

[32] LBP-98-7, 47 NRC at 207-09.

[33] The State's own witnesses admit this. See Bartlett ¶12; Mitchell ¶9.

design during a seismic event at the PFS facility."[34] The State went on to allege a series of deficiencies in Holtec's seismic analysis of the casks and pads, including among others the following: (i) the assumption that the pad will remain rigid in an earthquake (item 3d); (ii) the use of one, rather than three time histories in the analysis (item 2); (iii) the failure to consider the potential interaction between adjacent pads (item 3b); (iv) the failure to consider the effects of the pads' embedment in the soil (item 3e); and (v) the failure to consider the impact of dynamic seismic loads on the structural integrity of the pads (item 5).[35] The Board rejected as untimely the State's request to introduce proposed Contention EE.[36] The State now tries to re-raise these very same issues as part of its proposed "new" contention.[37] Thus, Proposed Utah QQ alleges a variety of concerns regarding Holtec Report No. HI-2012640, *Multi Cask Response at the PFS ISFSI from 2000-yr Seismic Event*, Rev. 2,[38] that parallel those raised and rejected by the Board with respect to Utah EE.[39] Indeed, they are the same concerns raised in proposed Utah Contention EE, and are even worded in much the same manner.[40] The State does not provide any justification for its attempt to reintroduce these dismissed contentions.

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[34] LBP 98-7, 47 NRC at 206.

[35] Id. at 206-07.

[36] Id. at 207-09. While not needing to rule on the merits of Contention EE because of its untimeliness, the Board noted that at least some of the issues raised in Contention EE (which are essentially the same as those raised in the Response with respect to the Holtec analysis) would have been inadmissible on the merits, had they been timely raised. Id. at 209, n. 25.

[37] Compare, e.g., LBP-98-7, 47 NRC at 206-07 with Request at 11-12. Likewise, large sections of the declarations of State witnesses in support of Proposed Utah QQ merely repeat their declarations in support of the State's opposition to summary dismissal of Utah L, especially in relation to cask and pad interaction and concerns over cask tipover analysis.

[38] This calculation contained the same exact methodology as Holtec Report No. HI-992277, Multi-Cask Seismic Research at the PFS ISFSI, From 2000 Year Seismic Event, Rev. 0, dated August 20, 1999.

[39] Proposed Utah QQ attacks this calculation's methodology for: (i) not adequately addressing the natural frequency of the cask-pad-soil-cement mass, (ii) assuming the storage pad to be rigid, (iii) not modeling the effects of inclined waves, (iv) using only one time history, (v) assuming that the casks will slide in a controlled manner (i.e., no cold bonding, pad deformation, etc.), (vi) not taking into account actual load paths (i.e., possible pad-to-pad interaction); and (vii) not modeling differences between the static and dynamic modulus of the soil cement under the pad. Request at 9-11.

[40] For example, in Proposed Contention EE the State asserted that "[a]nother unreasonable and oversimplified assumption by Holtec is that . . . [the] concrete pad will remain rigid" (EE Request at 8). This argument is raised in Proposed Utah QQ where Dr. Ostadan asserts as a problem with the calculation that "Holtec has assumed the pads to be rigid." Ostadan Dec. ¶116.) Similarly, the State attacks Holtec's non-linear analysis, which "is sensi-

Footnote continued on next page

The State also seeks to raise with respect to the CTB some of the same arguments that were rejected in 1997 when raised in Proposed Contention EE. For example, the State claims that cracking and separation may occur around the foundations of the CTB due to seismic motions, which is the same argument raised in proposed Contention EE with respect to pad structural integrity.[41] Likewise, Proposed Utah QQ challenges the assumption that the CTB mat is rigid, which is the same claim raised against the storage cask pads.[42] More generally, the State contends that the calculations for the CTB contain "potentially unconservative assumptions" that create "unacceptable uncertainty in the estimation of the seismic loadings and their potential impacts to the foundations."[43] This is the same argument that was previously raised in Proposed Contention EE with regard to the storage casks and pads.[44]

Thus, the State is clearly trying to raise issues that have already been rejected, without any justification for doing so. For that reason, Proposed Utah QQ should not be admitted.

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

"Lacking good cause for the one-month 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). 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

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tive to phasing of the input motion and thus multiple time histories should be used." (Id. ¶ 11d.) This is a condensed version of Contention EE's basis that "one cannot tell from the Holtec Seismic Report whether the interaction of the three independent components of the seismic time histories has been properly and conservatively evaluated" (EE Request at 7). Again, a number of the bases asserted in Proposed Utah QQ are also State's identical to those that the State raised in its opposition to the dismissal of Contention Utah L.

41 EE Request at 10-11.

42 Compare, e.g., proposed Contention EE item 3.d with Proposed Utah QQ at 10.

43 Request at 8, 9.

44 See LBP-98-7, 47 NRC at 206, item 3.

delay the proceeding. Of those factors, the third and fifth are to be accorded more weight than the second and fourth. Id.

Factor five weighs against admitting the new contention, whose introduction would clearly broaden and delay the proceeding. Proposed Utah QQ encompasses a broad range of issues relating to soil cement and other seismic design issues that are not currently being litigated in this proceeding. Thus, Proposed Utah QQ would undeniably broaden the issues and cause delays. Of particular concern, in terms of potential delays, is the State's assertion that the actual soil cement must be demonstrated and tested before a design basis can be adequately established,[45] as it is not only an incorrect statement of applicable licensing requirements, see, e.g., Private Fuel Storage, L.L.C. (Independent Fuel Storage Installation), CLI-00-13, 52 NRC 23, 33-34 (2000), but a proposition which, should it prevail, would dramatically delay this proceeding.

With respect to the third factor in 10 C.F.R. § 2.714(a)(1), the admission of the contention would do little to develop a sound record at this time, because the import of many of the issues raised in Proposed Utah QQ is an attack on commitments made by PFS to take certain actions (particularly with respect to the use of soil cement) which are to be taken after licensing is completed. Thus, admission and litigation of Proposed Utah QQ would not develop a sound record on which a licensing decision can be made at this time.

With respect to factor two, a number of concerns raised by the State relate to potential issues that may (or may not) become problems after the licensing of PFS.[46] For such issues, there is an adequate means of protecting the state's interest via a 10 CFR §2.206 petition for Staff action against PFS. And, 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

[45] Request at 6-7, 9, 11-12.

[46] For example, the State raises the possibility that "[i]f care is not taken during construction, the use of heavy equipment could cause significant remolding of the subbase soils, and such remolding could markedly affect the shear strength of the subbase at the interface with the cement treated soil." Mitchell Dec. at ¶14.

against admission of Proposed Utah QQ. The State has clearly failed to make the compelling showing required to overcome its lack of good cause.

D. PORTIONS OF PROPOSED UTAH QQ FAIL TO MEET THE REQUIREMENTS OF AN ADMISSIBLE CONTENTION

Portions of Proposed Utah QQ fail to meet the requirements for an admissible contention. NRC regulations require, inter alia, sufficient information to show that a genuine dispute exists on a material issue of law or fact. See 10 C.F.R. § 2.714(b)(2). The State's proffered bases for a number of claims raised in Proposed Utah QQ do not meet the Commission's requirements for specificity and materiality. Failure to comply with the requirements of 10 C.F.R. § 2.714 is grounds for dismissing those portions of Proposed Utah QQ. Private Fuel Storage, L.L.C. (Independent Spent Fuel Storage Installation), LBP-98-13, 47 N.R.C. 360, 365 (1998).

1. The State's Concerns Over Soil Cement Are Not Specific and Violate the Commission's Basic Licensing Principles

As discussed above, many of the concerns raised by the State in Proposed Utah QQ allege in various ways that the soil cement will not have the properties attributed to it in the seismic calculations. This allegation is insufficient to form the basis of a contention for two reasons: first, it is not definite enough to give rise to a material dispute of fact; and second, it seeks to controvert commitments made by PFS in its design calculations and the SAR with respect to the properties of the soil cement, without alleging that such commitments are unachievable.

It is well established that a contention is inadmissible if it fails to contain sufficient information to show that a genuine dispute exists with the Applicant on a material issue of law or fact. See 10 C.F.R. §2.714(b)(2)(iii); Texas Electric Utilities Co. (Comanche Peak Steam Electric Station, Unit 2), LBP-92-37, 36 NRC 370, 384 (1992). The State fails to meet this requirement in its proffered bases relating to the performance and properties of soil cement. The State merely asserts that the soil cement may be subject to a variety of factors that could cause it to not have the properties the design basis provides. For example, the State suggests that cracking in the soil cement, caused by any number of postulated conditions, might result in the soil cement no longer behaving as a single unit in the seismic calculations, (see Request at 11-14), but the State fails to assert that such postulated conditions will occur. On the other hand, the State does not challenge the proposition that the soil cement, if placed in accordance with the design specifications, will have the properties set forth in the seismic calculations. The State merely offers theoretical mechanisms by which the soil cement could, over time, fail to meet the design requirements. Such hypothetical allegations do not constitute a defined disagreement with PFS that would give rise to a litigable issue of fact. See, e.g., Vermont Yankee Nuclear Power Corp. (Vermont Yankee Nuclear Power Station), ALAB-919, 30 NRC 29 (1989).

The State contends that PFS has not shown "the strength, survivability and durability properties of the cement treated soil." (Request at 13-14.) PFS has specified the properties that the soil cement will have in order to assure that the facility meets NRC specifications. The State asserts that PFS will not meet those specifications, even though they are licensing commitments set forth in the SAR.[47] In order to operate, PFS will have to meet all the commitments set forth in the SAR. To assert that PFS will not abide by these commitments is not an admissible contention. "In the absence of evidence to the contrary, [the Board] will not presume that an applicant or licensee, and those who work for them, will not adhere to applicable regulations or standards." Private Fuel Storage, L.L.C. (Independent Spent Fuel Storage Facility), LBP-00-35, 52 NRC 364, 405 (2000); Private Fuel Storage, L.L.C. (Independent Spent Fuel Storage Facility), CLI-01-09 , 53 NRC (2001).

2. The State's Issues Regarding Soil Properties Are Speculative and Do Not Satisfy the Commission's Specificity Requirements

In order to be admissible, a contention must include a reasonably specific articulation of its rationale. See Carolina Power & Light Co. and North Carolina Eastern Municipal Power

[47] To the extent that the State and its witnesses are implying that the design developed by PFS for its soil cement cannot achieve the strengths cited in the calculations and the SAR, that assertion is contrary to well-established industry practice. See Exhibit B, which is an excerpt from ACI Report ACI-230.1R-90 (1997), which shows in Fig. 4.2 that strengths of 500 pounds per square inch – well in excess of those required by the PFS design – can be readily achieved, at low cement concentrations, for all types of soils.

Agency (Shearon Harris Nuclear Power Plant, Units 1 and 2), LBP-82-119A, 16 NRC 2069, 2070-71 (1982). The State's allegations as to soil properties fail to meet the specificity test. The State raises issues, for example, as to the potential adhesion between the soil cement and the underlying soils, and asserts that there may be changes in the water content of the soil. Request at 14-15. Yet, the State does not allege that these effects will occur, or provide a basis for expecting that they will. Instead, the State demands that PFS explain what steps will be taken in construction to avoid such changes, without even describing the mechanisms through which the changes may occur. See Bartlett Dec. ¶22.

Likewise, the State's assertion that the soil chemistry will undermine the properties of the soil cement should be rejected as inadmissible for the same reasons. The soil chemistry argument posits that "[s] alts and sulfates, if present [in the soil], could interfere with the cement hydration, and thus affect the strength and durability of the cement-treated soils." Request at 14, emphasis added. Thus, the State is claiming that because of the (hypothetical) existence of contaminants in the soil at the site with certain salts, the soil cement could be unable to exhibit the properties committed to in the SAR. For the reasons discussed above, this is an improper basis for a contention. See LBP-00-35, 364, supra.

V. CONCLUSIONS

For the foregoing reasons, PFS submits that Proposed Contention Utah QQ fails to raise a litigable contention and its admission should be denied.

Respectfully submitted,

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Dated: May 30, 2001

UNITED STATES OF AMERICA

NUCLEAR REGULATORY COMMISSION

Before the Atomic Safety and Licensing Board

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

CERTIFICATE OF SERVICE

I hereby certify that copies of Applicant's Response to State of Utah's Request for Admission of Late-Filed Contention Utah QQ were served on the persons listed below (unless otherwise noted) by electronic mail with conforming copies by U.S. mail, first class postage prepaid, this 30th day of May 2001.

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Exhibit A

EXHIBIT A
Summary of Claims Raised in Proposed Contention Utah QQ
And Dates in Which Information Relating to Each Claim Was Submitted to NRC

Item	Claim Sponsor	Claim	PFS Licensing Document	Document Issue Date
		SEISMIC CALCULATIONS		
1	Bartlett ¶12, Ostadan ¶13	PFS's calculation methodology (Calculation Nos. 05996.02-G(B)-13, Rev. 4,[1] and 05996.02-G(B)-04, Rev. 7[2]) oversimplifies model for the stability analyses of the CTB and the storage pads	PFS Letter, Parkyn to NRC, License Application Amendment No. 9, dated February 2, 2000. In SAR Rev. 9, submitted in LA Amendment No. 9, a subsection was added to SAR Section 2.6.1.12.1 entitled "Sliding Stability of the Cask Storage Pads Founded on and Within Soil Cement". This section takes credit for the beneficial effects of soil cement beneath and around the cask storage pads. The soil cement provides increased resistance to sliding and overturning of the pads due to the design basis ground motion. SAR page 2.6-60 (Rev. 9) stated: "The analysis of sliding stability of the cask storage pads embedded in soil cement is included in Calculation 05996.02-G(B)-04 (SWEC, 2000b [This is Calculation Rev. 5, dated January 26, 2000]) ... This analysis demonstrates that the soil cement can be designed to provide sufficient resistance, considering only the passive resistance of the soil cement, to provide a factor of safety against sliding that exceeds the minimum required value of 1.1 for the maximum loadings due to the PSHA 2,000-yr return period earthquake. Based on these conservative assumptions, the soil cement would need to have an unconfined compressive strength, f_c, of ~250 psi to provide sufficient thrust from passive resistance alone to obtain a factor of safety against sliding that is greater than the minimum required value of 1.1."	2/2/00
2	Ostadan ¶11.a	Holtec pad and cask stability calculations fail to take into account frequency dependency of soil spring and damping	Holtec's updated cask stability analysis was filed with LA Amendment 13, in June 2000. SAR Section 8.2.1.2 referenced Holtec Report No. HI-992277,[3] Multi-Cask Response at the PFS ISFSI, From 2000 Year Seismic Event, Revision 0, dated August 20, 1999, this report had been submitted to the NRC by PFS letter, Donnell to NRC, PFSF Site Specific Cask Stability Analysis, dated August 27, 1999. Calculation No. 05996.02-SC-4, Rev. 1, was submitted to the NRC by PFS letter from J. Donnell, dated September 9, 1999.	LA Amendment 13 submitted 6/23/00; Calc SC-4, Rev 1, submitted 9/9/99
3	Ostadan ¶11.b	Holtec's cask stability calculation incorrectly assumes that storage pads are rigid.	Holtec's cask stability analysis, including the assumption of rigid pads, was included in original PFSF License Application submittal in June 1997. SAR	

[1] Calculation No. 05996.02-G(B)-13, Rev. 3, filed in June 2000, is included as Attachment 2 to Exhibit A. The June 2000 revision is included to show that all methodological assumptions the State challenges, as referenced throughout Exhibit A, were in place by June 2000 or earlier as noted in the above table.
[2] Calculation No. 05996.02-G(B)-04, Rev. 6, filed in June 2000, is included as Attachment 1 to Exhibit A. The June 2000 revision is included to show that all methodological assumptions the State challenges, as referenced throughout Exhibit A, were in place by June 2000 or earlier as noted in the above table.
[3] A redacted excerpt of Holtec Report HI-2012640, *Multi-Cask Seismic Response at the PSF ISFSI, From 2000 Year Seismic Event,* is included as Attachment 3 to Exhibit A. The portion of Report HI-2012640 clarifies that the methodological assumptions of Holtec's cask stability analyses have remain unchanged since the initial Holtec Report HI-971631, Rev. 0, dated May 19, 1997, including in Report HI-992277.

			Section 8.2.1.2, Rev. 0, referred to Holtec Report No. HI-971631, Multi-Cask Seismic Response at the PSF ISFSI, Rev. 0, dated May 19, 1997. This report made the assumption that the storage pads were rigid, as did all following Holtec cask seismic stability analyses (Attachment 3). Holtec report HI-971631 was submitted in PFS letter, Parkyn to the NRC, Submittal of Calculation Package, dated July 14, 1997.	7/14/97
4	Ostadan ¶11.c	Holtec's cask stability calculations do not model the effects of non-vertically propagating seismic waves.	Holtec's modeling of seismic waves arriving at the pad foundations was specifically set forth in the SAR LA Amendment 13, June 2000. SAR Section 2, Rev. 13, referenced Geomatrix Consultants, Inc., 1999b, Development of design ground motions for the Private Fuel Storage Facility. This report also assessed the impact of the PFSF being "close to major faults". SAR Section 8.2.1.2, Rev 13, referenced Holtec Report No. HI-992277, Multi-Cask Response at the PFS ISFSI, From 2000 Year Seismic Event, Rev. 0, dated August 20, 1999. This report had been submitted to the NRC by PFS letter, Donnell to NRC, PFSF Site Specific Cask Stability Analysis, dated August 27, 1999.	LA Amendment 13 submitted 6/23/00
5	Ostadan ¶11.d	Holtec's cask stability calculation fails to use multiple time histories.	Holtec's cask stability analysis and the modeling of seismic time history contained in LA Amendment 13, in June 2000 (or earlier). SAR Section 8.2.1.2, Rev. 13, referenced Holtec Report No. HI-992277, Multi-Cask Response at the PFS ISFSI, From 2000 Year Seismic Event, Rev. 0, dated August 20, 1999. This report had been submitted to the NRC by PFS letter, Donnell to NRC, PFSF Site Specific Cask Stability Analysis, dated August 27, 1999. This cask stability analysis referenced Geomatrix Calculation 05996.02-(PO18)-3, Rev. 0, Development of Time Histories for 2,000-Year Return Period Design, dated August 24, 1999. This calculation had been submitted to the NRC by PFS letter from J. Donnell, dated September 9, 1999.	LA Amendment 13 submitted 6/23/00
62	Ostadan ¶11.e	Holtec's cask stability calculation assumes that casks will slide on the pad in a controlled manner, ignoring the potential for cold bonding of cask to pad, which would prevent the cask from sliding on the pad or moving smoothly in an earthquake	Holtec's cask stability analysis and its assumptions regarding the movement of casks on the storage pad have been consistent since the original PFSF License Application submittal in June 1997. SAR Section 8.2.1.2, Rev. 0, referred to Holtec Report No. HI-971631, Multi-Cask Seismic Response at the PSF ISFSI, Revision 0, dated May 19, 1997. This report made the same assumptions regarding movement of the casks on a pad during a seismic event as the Holtec report currently referenced by SAR Section 8.2.1.2 (Holtec Report HI-2012640) as did all Holtec cask seismic stability analyses. Holtec report HI-971631 was submitted in PFS letter, Parkyn to the NRC, Submittal of Calculation Package, dated July 14, 1997.	7/14/97
7	Ostadan ¶12.a	SWEC Calculation No. 05996.02-SC-5, Rev. 2, seismic Analysis of Canister Transfer Building, erroneously assumes that the mat of the CTB is rigid.	LA Amendment 6 incorporated the assumption that the CTB mat would be rigid. This LA Amendment was submitted in PFS letter, Parkyn to the NRC, License Application Amendment No. 6, dated September 8, 1999. SAR page 4.7-8c (Rev 6) stated the following "The impedance functions were developed, using the Stone & Webster computer program REFUND (Reference 41), by considering the foundation mat as a rigid structure	LA Amendment 6 submitted 9/8/99; Calcs. SC-4, Rev 1, and SC-5, Rev 1, submitted

			located at the surface of the soil profile. These assumptions are appropriate since the building foundation is a five-foot thick concrete mat located at grade. Development of the impedance functions is documented in calculation SC-4 (Reference 42). . . . The zero period accelerations (ZPA) at each point of the lumped mass model and response spectra at El. 170'-0", which is the bridge crane support location are presented in the dynamic analysis described in calculation SC-5 (Reference 44)." References 42 and 44 of SAR Rev. 6 were Calculations SC-4, Rev. 1, and SC-5, Rev. 1, respectively. These calculations were both submitted by PFS letter from J. Donnell to the NRC, dated September 9, 1999. Treatment of the foundation mat as a rigid structure for seismic analysis purposes is in accordance with Section 3.3.1.6 of ASCE-4, 1998.	9/9/99
8	Ostadan ¶12.b	Calculation No. 05996.02-SC-5, Rev. 2, and the supporting calculation (Calculation No. 05996.02-SC-4, Rev. 2, *Development of Soil Impedance Functions for Canister Transfer Building*, fail to consider the effect of soil cement around CTB on impedance functions, and the kinematic motion of the foundation.	Soil cement was first introduced around the Canister Transfer Building base mat in LA Amendment 22, dated March 30, 2001. Soil cement was introduced into the design of the storage pads in LA Amendment 8 in a letter dated December 16, 1999.	LA Amendment 22, dated 3/30/01; LA Amendment 8, dated 12/16/99.
9	Ostadan ¶12.c	Calculation Nos. 05996.02-SC-4 and 05996.02-SC-5 fail to model the effect of inclined seismic waves on CTB design	LA Amendment 6 referenced Calculation Nos. 05996.02-SC-4, Rev. 1, and 05996.02-SC-5, Rev. 1. The seismic analysis of the CTB did not consider the effect of inclined seismic waves. LA Amendment 6 was submitted in PFS letter, Parkyn to the NRC, LA Amendment 6, dated September 8, 1999. Calculation Nos. SC-4, Rev. 1, and SC-5, Rev. 1, were both submitted by PFS letter from J. Donnell to the NRC, dated September 9, 1999.	LA Amendment 6 submitted 9/8/99; Calcs. SC-4, Rev 1, and SC-5, Rev 1, submitted 9/9/99
10	Ostadan ¶14	Calculation No. 05996.02- G(B)-04, Rev. 7 on the stability analysis of the pads fails to consider the natural frequency of the cask-pad-soil cement system, thus underestimating the seismic loads.	The storage pad stability analysis was specifically referenced in LA Amendment 13, in June 2000. SAR Section 2.6.1.12.1, Rev. 13, references Calculation No. 05996.02- G(B)-04, Rev. 6, in discussions of storage pad stability analyses. This calculation was submitted to the NRC by PFS letter from J. Donnell dated June 19, 2000, "Submittal of Commitment Resolution Letter #34 Information".	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
11	Ostadan ¶14	Calculation No. 05996.02- G(B)-04, Rev. 7 fails to consider actual load path on pads from seismic loading and potential effect of pad-to-pad interaction.	Same as above.	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
12	Mitchell ¶9	Calculation No. 05996.02- G(B)-04, Rev. 7 treats CTB and the soil-cement as a rigid block, whereas there are expected to be differences in inertial loadings from one part of the building to another.	This contention focuses on the behavior of the CTB and pad foundations and surrounding soil cement. Stone & Webster Calculation No. 05996.02-G(B)-04, Rev. 6, Stability Analyses of Storage Pad, was referenced in SAR Revision 13 (Submitted to the NRC by PFS Letter, Parkyn to NRC, License Application Amendment No. 13, dated June 23, 2000). The assumptions	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted

			for sliding of a rigid foundation within the cement-treated soil made in Calculation 05996.02-G(B)-04 have not changed between Rev. 6 and Rev. 7 of this calculation.	6/19/00
13	Mitchell ¶10	Calculation No. 05996.02-G(B)-13, Revision 4 on stability analysis of the CTB incorrectly treats the building as rigid body in overturning analyses.	SAR Section 2.6.1.12.2, "Stability and Settlement Analyses—Canister Transfer Building", Rev. 13 (page 2.6-76), referenced Calculation 05996.02-G(B)-13, Rev. 3, Stability Analyses of the Canister Transfer Building Supported on a Mat Foundation, in regards to overturning stability of the CTB. This calculation was submitted to the NRC by PFS letter from J. Donnell dated June 19, 2000, "Submittal of Commitment Resolution Letter #34 Information". Calculation No. 059996.02-G(B)-13, Rev. 3 also assumed that the foundation mat of the CTB was rigid.	LA Amendment 13 submitted 6/23/00; Calc G(B)-13, Rev. 3 submitted 6/19/00
14	Ostadan ¶11.f	Holtec's drop/tipover analysis of the casks models only the static modulus of the soil cement, not the dynamic modulus, failing to take into account possible changes in bearing pressure acting on the soil/cement.	SAR Section 8.2.6.2, Rev. 21, references Holtec Report HI-2012653, which analyzes storage cask tipover and drop events onto a storage pad taking into account the soil cement beneath the pad. Soil cement was introduced into the design of the storage pads in LA Amendment 8 in a letter dated December 16, 1999.	LA Amendment 22, dated 3/30/01; LA Amendment 8, dated December 16, 1999.
		SOIL CEMENT		
15	Bartlett ¶9, Mitchell ¶13	Failure to conduct site specific soil cement tests to determine its properties, reaction with native soils, constructability, and long term performance. .	PFS Letter, Parkyn to NRC, LA Amendment 8, dated December 16, 1999, introduced the use of soil cement in the storage pad emplacement area to enhance pad stability under seismic conditions. SAR page 2.6-84 (Rev. 8) stated: "The required engineering characteristics of the soil cement can be easily engineered during detailed design to meet the necessary strength requirements." See also SAR page 2.6-26 (Rev. 8). SAR page 2.6-91 (Rev. 8) stated: "Procedures required for placement and treatment of the soil cement, lift surfaces, and foundation contact will be established in accordance with the recommendations of ACI (1998) during the mix design and testing process. Specific construction techniques and field quality control requirements will be identified in the construction specifications developed by PFS during this detailed design phase of the project."	LA Amendment 8, dated 12/16/99.
16	Bartlett ¶10	No precedent has been demonstrated for the use of soil cement to resist sliding in nuclear facilities.	PFS Letter, Parkyn to NRC, LA Amendment 8, dated December 16, 1999. SAR pages 2.6-85 and 86 (Rev. 8) stated: "Soil cement has been used extensively in the United States and around the world since the 1940's. It was first used in the United States in 1915 for constructing roads. It also has been used at nuclear power plants in the United States and in South Africa. The largest soil-cement project worldwide involved construction of soil-cement slope protection for a 7,000-acre cooling-water reservoir at the South Texas Nuclear Power Plant near Houston, TX. Soil cement also was used to replace an ~18-ft thick layer of potentially liquefiable sandy soils under the foundations of two 900-MW nuclear power plants in Koeberg, South Africa (Dupas and Pecker, 1979)."	LA Amendment 8, dated 12/16/99.
17	Bartlett ¶11	There has been a failure to demonstrate "proof of concept," for soil cement and to	Soil cement was introduced in Revision 8 of the SAR (PFS Letter, Parkyn to NRC, LA Amendment 8, dated December 16, 1999). SAR page 2.6-26d	

		perform preliminary designs capable of independent review, verification and checking.	(Rev. 8) stated: "The engineering characteristics of the soil-cement can be easily engineered during detailed design to meet the necessary strength requirements." SAR page 2.6-84 (Rev. 8) stated: "The required engineering characteristics of the soil cement can be easily engineered during detailed design to meet the necessary strength requirements."	LA Amendment 8, dated 12/16/99.
18	Bartlett ¶13	Calculations No. 05996.02-G(B)-13, Rev. 4 and No. 05996.02-G(B)-04, Rev. 7 on the dynamic stability of the CTB improperly model the building's concrete foundation and the soil cement as rigid bodies, failing to take into account that earthquake stresses may crack the soil cement buttress and give rise to preferential slip planes for passive failure wedges.	This contention focuses on the behavior of the soil cement, which buttresses the pads and CTB, increasing their sliding stability under seismic conditions. Stone & Webster Calculation No. 05996.02- G(B)-04, Rev. 6, Stability Analyses of Storage Pad, was referenced in SAR Section 2.6.1.12.1, Stability and Settlement Analyses-Cask Storage Pads, Revision 13 (Submitted to the NRC by PFS Letter, Parkyn to NRC, LA Amendment 13, dated June 23, 2000). This calculation was submitted to the NRC by PFS letter from J. Donnell dated June 19, 2000, "Submittal of Commitment Resolution Letter #34 Information".	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
19	Bartlett ¶14	Soil cement may crack through a number of mechanisms.	At least by LA Amendment 13, soil cement was incorporated into the design of the facility and included in Calculation No. 05996.02 G(B)-04 for storage pad stability.	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
20	Bartlett ¶15	PFS has failed to consider the bending, torsional and beam-shear and compressional stresses on soil cement due to the variety of complex waveforms associated with a seismic event will have on the CTB and pad foundations and underlying soil.	The assumptions regarding the seismic event have remained unchanged since at least License Amendment 13 and revision 6 of Calculation No. 05996.02-G(B)-04, which addressed storage pad stability. Calculation No. 05996.02-G(B)-13 employs the same methodology for modeling seismic events.	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
21	Bartlett ¶16	PFS has failed to consider the high stress and strain concentrations at the soil cement/CTB foundation interface during seismic events.	Calculation Nos. 05996.02-G(B)-04 and 05996.02-G(B)-13 have always treated the surface areas underlying the storage pads and the CTB as single units. This treatment remained unchanged with the incorporation of soil cement as part of License Amendment 13 and Calculation No. 05996.02-G(B)-04, Rev. 6 for storage pad stability.	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
22	Bartlett ¶17	PFS has failed to consider the potential debonding of soil cement along lift boundaries.	Same as above. In addition, SAR Section 2.6.4.11, submitted as part of LA Amendment 13, discussed techniques to improve bonding between lifts.	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
23	Bartlett ¶¶18-20	PFS has failed to take into account the potential cracking of the soil cement due to: drying and curing; frost penetration and expansion cracking and vehicle loading cracking.	Calculation Nos. 05996.02-G(B)-04 and 05996.02-G(B)-13 have always treated the entire surface area underlying the storage pads and the CTB as a single unit. This included the incorporation of soil cement as part of LA Amendment 13 and revision 6 of Calculation No. 05996.02-G(B)-04 for storage pad stability.	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00

24	Bartlett ¶21	PFS has failed to study the long term performance of soil cement and its ability to resist earthquake forces for a 40-year service period.	LA Amendment 13 incorporated the use of soil cement into the seismic calculations.	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
25	Bartlett ¶22	The proposed use of a CTB shear key to resist sliding of the building will not be effective.	The shear key around the perimeter of the CTB foundation mat was first incorporated into the CTB design and discussed in SAR Sections 2.6.1.11.2 and 2.6.1.12.2, Rev. 13, (Submitted to the NRC by PFS Letter, Parkyn to NRC, License Application Amendment No. 13, dated June 23, 2000). SAR Section 2.6.1.12.2 (page 2.6-78, Rev. 13) stated "A 1-ft deep key will be constructed around the perimeter of the mat to ensure that the full shear strength of the clayey soils is engaged to resist sliding of the structure due to loads from the design basis ground motion." Calculation No. 05996.02-G(B)-13, Rev. 3, incorporated the effects of this shear key. Revision 3 of Calculation No. 05996.02-G(B)-13 was submitted by PFS letter, J. Donnell to the NRC, dated June 19, 2000, "Submittal of Commitment Resolution Letter #34 Information". Page 4 of this Calculation, Rev. 3, stated: "Added a 1 -ft deep key around the perimeter of the Canister Transfer Building mat to permit use of the cohesive strength of the in situ silty clay/clayey silt in resisting sliding due to loads from the design basis ground motion."	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
26	Ostadan ¶13	Reliance on passive pressure from soil cement to resist seismic loads in Calculation No. 05996.02-G(B)-13, Rev. 4 is potentially wrong due to possible settlement and cracking.	Argument attacks methodology established for use with storage pad and soil cement in LA Amendment 13. SAR Section 2.6.1.12.1, Rev. 13, "Stability and Settlement Analyses – Cask Storage Pads", stated "the actual bearing pressire for this case was about 1.9 ksf, and the estimated total settlement of the pad was determined to be about 3.3 inches."	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
27	Mitchell ¶11	Calculation No. 05996.02-G(B)-04, Rev. 7 fails to calculate bending stresses on soil cement due to static loading, freeze-thaw and wet dry, shrinkage and dynamic loading and their consequences.	Calculation No. 05996.02- G(B)-04, Rev. 6, Stability Analyses of Storage Pad, was referenced in SAR Section 2.6.1.12.1, Stability and Settlement Analyses-Cask Storage Pads, Rev. 13 (Submitted to the NRC by PFS Letter, Parkyn to NRC, License Application Amendment No. 13, dated June 23, 2000). This calculation was submitted to the NRC by PFS letter from J. Donnell dated June 19, 2000, "Submittal of Commitment Resolution Letter #34 Information".	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
28		Failure to take into account the potential for differential settlement between foundations and soil cement.	The specific settlement assumptions of PFS calculations have been available since LA Amendment 13, in June 2000. SAR Section 2.6.1.12.1, Rev. 13, "Stability and Settlement Analyses—Cask Storage Pads", stated "The actual bearing pressure for this case was about 1.9 ksf, and the estimated total settlement of the pad was determined to be about 3.3 inches."	LA Amendment 13 submitted 6/23/00
		SOIL PROPERTIES		
29	Bartlett ¶23	Seismic calculations fail to accurately model the nature of subsurface soils after the	PFS Letter, Parkyn to NRC (Director, Office of NMSS), License Application Amendment No. 3, dated May 19, 1999, discussed the characteristics of	LA Amendment 3, submitted

		addition of soil cement, such that a soft layer of soil is enclosed between two much stiffer layers, and the stress and strain concentrations imparted on the soft soil layer.	the soil foundation. SAR Section 2.5.2, Rev. 3, December 16, 1999, incorporated updated soil data. Section 2.5.2, Rev. 3 states, "based on boring data obtained at the site, the uppermost soil layer consists of interbedded silt, silty clay, and clayey silt with a thickness of approximately 30 ft. This layer is underlain by very dense fine sand and silt that extends to a depth of approximately 45 ft." LA Amendment 8 also incorporated the use of soil cement as the stiff, topmost layer.	May 19, 1999 12/16/99
30	Mitchell ¶12	The amounts of cement that PFS plans to use may not be sufficient to produce a true soil cement.	Calculation No. 05996.02- G(B)-04, Revision 6, Stability Analyses of Storage Pad, was referenced in SAR Section 2.6.1.12.1, Stability and Settlement Analyses-Cask Storage Pads, Revision 13 (Submitted to the NRC by PFS Letter, Parkyn to NRC, LA Amendment 13, dated June 23, 2000). This calculation was submitted to the NRC by PFS letter from J. Donnell dated June 19, 2000, "Submittal of Commitment Resolution Letter #34 Information".	LA Amendment 13 submitted 6/23/00; Calc G(B)-04, Rev. 6 submitted 6/19/00
31	Mitchell ¶14	PFS has failed to account for potential effect of change in water content below pads over time from the use of soil cement and the effect of the use of heavy placement equipment on remolding of the subbase soils, affecting the shear strength of the subbase at the interface with the cement treated soil.	Soil cement's use was introduced in Revision 8 of the PFS SAR (PFS Letter, Parkyn to NRC, License Application Amendment No. 8, dated December 16, 1999). That License Amendment indicated that soil cement would be used under and around the storage pads.	12/16/99
32	Bartlett ¶24	PFS has failed to take into account possible partial reduction in undrained shear strength – due to pore pressure generation during earthquake cycling.	The original SAR filed as part of the initial PFS License Application and the supporting calculations used undrained shear strength. This assumption was further clarified by LA Amendment 6 (submitted in PFS letter, Parkyn to the NRC, "License Application Amendment No. 6", dated September 8, 1999). This amendment referenced Calculation No. 05996.02-G(B)-04, Rev. 4, for storage pad stability analyses, and Calculation No. 05996.02-G(B)-13, Rev. 1, for CTB stability analyses. Both these calculation revisions were submitted in PFS letter, John Donnell to the NRC, "Calculation Package Submittal, dated September 9, 1999.	Initial License Application; LA Amendment 6 submitted 9/8/99; Calc G(B)-04, Rev. 4, and Calc G(B)-13, Rev. 1, both submitted 9/9/99
33	Bartlett ¶25	PFS fails to consider the effect of potential moisture content changes in the foundation soils with time and their potential effect on undrained shear strength used in design of the foundation systems.	The original SAR and supporting calculations did not consider change sin moisture content. For example, 2.6.1.11.1, 2.6.1.11.2, et seq., which would be the place where such effects would be discussed, do not discuss the effect of water content changes in discussing soil strength. This assumption was further clarified by LA Amendment 6 (submitted in PFS letter, Parkyn to the NRC, "License Application Amendment No. 6", dated September 8, 1999). This amendment referenced Calculation No. 05996.02-G(B)-04, Rev. 4, for storage pad stability analyses, and Calculation No. 05996.02-G(B)-13, Rev. 1, for CTB stability analyses. Both these calculation revisions were submitted in PFS letter, John Donnell to the NRC, "Calculation Package Submittal, dated September 9, 1999.	Initial License Application; LA Amendment 6 submitted 9/8/99; Calc G(B)-04, Rev. 4, and Calc G(B)-13, Rev. 1, both submitted 9/9/99

Attachments
to
Exhibit A

Exhibit A

Attachment 1

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CALCULATION TITLE PAGE

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CLIENT & PROJECT	PAGE 1 OF
PRIVATE FUEL STORAGE, LLC - PRIVATE FUEL STORAGE FACILITY	~~87~~ + 1A, 1B, 5A +15 pp OF ATTACHMENTS

CALCULATION TITLE (Indicative of the Objective):	QA CATEGORY (✓)
STABILITY ANALYSES OF STORAGE PADS	☑ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ____ OTHER

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

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

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CLIENT & PROJECT	PAGE
PRIVATE FUEL STORAGE, LLC - PRIVATE FUEL STORAGE FACILITY	1A OF 87

CALCULATION TITLE (Indicative of the Objective):	QA CATEGORY (✓)
STABILITY ANALYSES OF STORAGE PADS	☑ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ______ OTHER

CALCULATION IDENTIFICATION NUMBER

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

*** APPROVALS - SIGNATURE & DATE**

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CONTINUED FROM PAGE 1						
PAUL J. TRUDEAU 6/27/97 Paul J. Trudeau	LP SINGH L.P. Singh 7-1-97	LP SINGH L.P. Singh 7-1-97	3	2		✓
D L ALOYSIUS D L Aloysius S. Y. BOAKYE 9/3/99 S.Y. Boakye	S. Y. BOAKYE 9/3/99 S.Y. Boakye D L ALOYSIUS D L Aloysius	T. Y. Chang 9/3/99 Thomas Y. Chang T Y. Chang 9/3/99 Thomas Y. Chang	4	3	✓	
SEE PAGE 3 FOR IDENTIFICATION OF PREPARED/REVIEWED BY						
PJ TRUDEAU 1-26-2000 Paul J. Trudeau	SY BOAKYE 1-26-2000 TYC for S Y Boakye L. LIU 1-26-2000 Lee Liu	Thomas Y. Chang 1-26-2000 Thomas Y. Chang	5	4		✓

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

CALCULATION IDENTIFICATION NUMBER				
J. O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.02	G(B)	04		

* APPROVALS – SIGNATURE & DATE						
PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES * CALC. NO. OR REV. NO.	CONFIRMATION *REQUIRED (✓) YES	NO
PAUL J. TRUDEAU 6-16-00 Paul J. Trudeau	THOMAS Y. CHANG 6-16-00 Thomas Y. Chang	THOMAS Y. CHANG 6-16-00 Thomas Y. Chang	6	5		✓

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CALCULATION IDENTIFICATION NUMBER				PAGE 2
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05996.02	G(B)	04 - 6		

TABLE OF CONTENTS

RECORD OF REVISIONS

REVISION 0

Original Issue

REVISION 1

Revision 1 was prepared to incorporate the following:

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

REVISION 2

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

REVISION 3

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

REVISION 4

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

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

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

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

Modified/updated conclusions.

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

Remaining pages prepared by DLAloysius and reviewed by SYBoakye.

REVISION 5

Major re-write of the calculation.

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

REVISION 6

1. Added "References" section.
2. Revised shear strength used in the sliding stability analyses of the soil cement/silty clay interface to be the strength measured in the direct shear tests performed on samples obtained from depths of ~5.8 ft in the pad emplacement area. The shear strength equaled that measured for stresses corresponding to the vertical stresses at the bottom of the fully loaded cask storage pads.
3. Removed static and dynamic bearing capacity analyses based on total-stress strengths and added dynamic bearing capacity analyses based on c_u = 2.2 ksf..
4. Revised method of calculating the inclination factor in the bearing capacity analyses to that presented by Vesic in Chapter 3 of Winterkorn and Fang (1975). Vesic's method expands upon the theory developed by Hansen for plane strain analyses of footings with inclined loads. Vesic's method permits a more rigorous analysis of inclined loads acting in two directions on rectangular footings, which more closely represents the conditions applicable for the cask storage pads.

OBJECTIVE OF CALCULATION

Evaluate the static & seismic stability of the cask storage pad foundations at the proposed site, including overturning, sliding, and bearing capacity for static loads & for dynamic loads due to the design basis ground motion (PSHA 2,000-yr return period earthquake).

ASSUMPTIONS/DATA

The arrangement of the cask storage pads is shown on SWEC Drawing 0599601-EY-2-B. The spacing of the pads is such that each N-S row of pads may be treated as one long strip footing with B/L ~ 0 & B=30 ft for the bearing capacity analyses.

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

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

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

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

Based on probabilistic seismic hazard analysis, the peak acceleration levels of 0.528g for horizontal ground motion and 0.533g for the vertical ground motion were determined as the design bases of the PFSF for a 2,000-yr return period earthquake (Geomatrix Consultants, Inc, 1999b).

Geotechnical Properties

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

γ_{moist} = 80 pcf for the soils underlying the pad emplacement area.

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

In practice, the average shear strength along the anticipated slip surface of the failure mode should be used in the bearing capacity analysis. This slip surface is normally confined to within a depth below the footing equal to the minimum width of the footing. In this case, the effective width of the footing is decreased because of the large eccentricity of the load on the pads due to the seismic loading. As indicated in Table 2.6-7, the minimum effective width occurs for Load Case IIIB, where B' = 16.3 ft. Figure 7 illustrates that the anticipated slip surface of the bearing capacity failure would be limited to the soils within the upper two-thirds of the upper layer. Therefore, in the bearing capacity analyses presented herein, the undrained strength measured in the UU triaxial tests was not increased to reflect the increase in strength observed for the deeper-lying soils in the cone penetration testing.

Table 6 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C) summarizes the results of the triaxial tests that were performed within depths of ~10 ft. The undrained shear strengths measured in these tests are plotted vs confining pressure in Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C). This figure is annotated to indicate the vertical stresses existing prior to construction and following completion of construction.

The undrained shear strengths measured in the triaxial tests are used for the dynamic bearing capacity analyses because the soils are partially saturated and they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. As indicated in Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C), the undrained strength of the soils within ~10 ft of grade is assumed to be 2.2 ksf. This value is the lowest strength measured in the UU tests, which were performed at confining stresses of 1.3 ksf. This confining stress corresponds to the in situ vertical stress existing near the middle of the upper layer, prior to construction of these structures. It is much less than the final stresses that will exist under the cask storage pads and the Canister Transfer Building following completion of construction. Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C) illustrates that the undrained strength of these soils increase as the loadings of the structures are applied; therefore, 2.2

ksf is a very conservative value for use in the dynamic bearing capacity analyses of these structures.

Direct shear tests were performed on undisturbed specimens of the silty clay/clayey silt obtained at a depth of 5.7 ft to 6 ft in Boring C-2. These tests were performed at normal stresses that were essentially equal to the normal stresses expected:

1. under the fully loaded pads before the earthquake,
2. with all of the vertical forces due to the earthquake acting upward, and
3. with all of the vertical forces due to the earthquake acting downward.

The results of these tests are presented in Attachment 7 of the Appendix 2A of the SAR and they are plotted in Figure 7 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C). Because of the fine grained nature of these soils, they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. Therefore, sliding stability analyses included below of the cask storage pads constructed directly on the silty clay are performed using the shear strength measured in these direct shear tests for a normal stress equal to the vertical stress under the fully loaded cask storage pads prior to imposition of the dynamic loading due to the earthquake. As shown in Figure 7 of Calc 05996.02-G(B)-05-2 (copy included in Attachment C), this shear strength is 2.1 ksf and the friction angle is set equal to 0°.

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

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

Case IA Static using undrained strength: ϕ = 0° & c = 2.2 ksf.

Case IB Static using effective-stress strength: ϕ = 30° & c = 0.

The pads will be constructed on and within soil cement, as illustrated in SAR Figure 4.2-7 and described in SAR Sections 2.6.1.7 and 2.6.4.11. The unit weight of the soil cement is assumed to be 100 pcf in the bearing capacity analyses included herein. The strength of the soil cement is conservatively ignored in these bearing capacity analyses.

METHOD OF ANALYSIS

DESCRIPTION OF LOAD CASES

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

The following load combinations are analyzed:

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

For Case II, 100% of the dynamic lateral forces in both X and Z directions are combined. For Cases III and IV, the effects of the three components of the design basis ground motion are combined in accordance with procedures described in ASCE (1986) to account for the fact that the maximum response of the three orthogonal components of the earthquake do not occur at the same time. For these cases, 100% of the dynamic loading in one direction is assumed to act at the same time that 40% of the dynamic loading acts in the other two directions. For these cases, the suffix "A" is used to designate 40% in the X direction (N-S, as shown in Figure 1), 100% in the Y direction (vertical), and 40% in the Z direction (E-W). Similarly, the suffix "B" is used to designate 40% in the X direction, 40% in the Y, and 100% in the Z, and the suffix "C" is used to designate 100% in the X direction and 40% in the other two directions. Thus,

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

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

OVERTURNING STABILITY OF THE CASK STORAGE PADS

The factor of safety against overturning is defined as:

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

The resisting moment is calculated as the weight of the pad and casks x the distance from one edge of the pad to the center of the pad in the direction of the minimum width. The weight of the pad is calculated as 3 ft x 64 ft x 30 ft x 0.15 kips/ft^3 = 864 K, and the weight of 8 casks is 8 x 356.5 K/cask = 2,852 K. The moment arm for the resisting moment equals ½ of 30 ft, or 15 ft. Therefore,

$$\Sigma M_{Resisting} = [\underset{Wp}{864\ K} + \underset{Wc}{2{,}852K}] \times \underset{B/2}{15\ ft} = 55{,}740\ ft\text{-}K$$

The driving moment includes the moments due to the horizontal inertial force of the pad x ½ the height of the pad, the vertical inertial force of the pad plus casks x ½ the minimum width of the pad, and the horizontal force from the casks acting at the top of the pad x the height of the pad. The casks are simply resting on the top of the pads; therefore, this force cannot exceed the friction force acting between the steel bottom of the cask and the top of the concrete storage pad. This friction force was calculated based on the upper-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 - 0.533 x 2,852K) = 1,066 K. This is less than the maximum dynamic cask horizontal driving force of 1,855 K (Table D-1(c) in CEC, 1999). 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 1,066K.

$$\Sigma M_{Driving} = 1.5\text{ ft} \times \overset{a_h}{0.528} \times \overset{Wp}{864\text{ K}} + \overset{a_v}{0.533} \times [\overset{Wp}{864\text{ K}} + \overset{Wc}{2{,}852\text{ K}}] \times \overset{B/2}{15\text{ ft}} + 3\text{ ft} \times \underset{EQhc}{1{,}066\text{ K}} = 33{,}592\text{ ft-K.}$$

$$\Rightarrow FS_{OT} = \frac{55{,}740\text{ ft} - \text{K}}{33{,}592\text{ ft} - \text{K}} = 1.66$$

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 = 64 feet

SLIDING STABILITY OF THE PADS CONSTRUCTED ON AND WITHIN SOIL CEMENT

Objective:

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

Method/Assumptions:

1. 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.
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

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 =

$$1.1\times\left[3'\times30'\times64'\times0.15\frac{K}{ft^3}+8\text{ casks}\times356.5\frac{K}{cask}\right]\times0.528$$

$$\therefore\quad T = 2{,}158\text{ K}$$

Assuming this resisting force is provided <u>only</u> 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=64').

Find the minimum cohesion required to provide FS = 1.1.

$$P_P \text{ must be} \geq 2{,}158K = \frac{1}{2}\cdot\underbrace{0.100\frac{K}{ft^3}}_{\gamma}\times\underbrace{(2\text{ ft})^2}_{H^2}\times\underbrace{1.0}_{K_P} + 2\bar{c}\cdot\underbrace{2\text{ ft}}_{H}\cdot\underbrace{\sqrt{1.0}}_{\sqrt{N_\phi}}$$

$$\frac{2{,}158\text{ K}}{30\text{ ft}} = 0.2\frac{K}{ft} + 4\bar{c} = 71.93\frac{K}{LF} \Rightarrow 4\bar{c} = 71.73\frac{K}{LF}$$

$$\therefore\quad \bar{c} \geq 17.93\frac{ksf}{LF}\times\left(\frac{ft}{12\text{ in.}}\right)^2\times\frac{1{,}000\,\#}{K} = 125\text{ psi}$$

The unconfined compressive strength equals twice the cohesion, or 250 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 >250 psi, the percentage of cement required would be ~250/40 =

6.25%. 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 3 to 5 ft below most of the pads. Thus, the area available to resist sliding will greatly exceed that of the pads alone. 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 in the pad emplacement area indicate the shear strength available to resist sliding from loads due to the design basis ground motion 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, but it is constructed such that the top is 3.5" above grade to accommodate potential settlement. The depth of the pad is used in this analysis only for calculating the maximum dynamic lateral earth pressure; therefore, it is conservative to ignore the 3.5" that the pad sticks out of the ground.

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 (64 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. The maximum dynamic cask driving force, however, acts in the N-S direction. To be conservative, assume that it acts in the E-W direction in this analysis of sliding stability. However, the maximum horizontal force that can be applied to the top of the pad by the casks is limited to the maximum value of the coefficient of friction between the cask and the top of the pad, which equals 0.8, multiplied by the cask normal force.

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.

$P_a = [0.5 \times 125 \text{ pcf} \times (3 \text{ ft})^2 \times 0.22] \times 64$ ft (length)/storage pad = 7,920 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.

$\beta = \alpha = 0$

$$\theta = \tan^{-1} \times \left(\frac{0.528}{1 - 0.533} \right) = 48.5^\circ$$

$\phi = 40^\circ$

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

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

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

$$\therefore \quad K_{AE} = \frac{1 - 0.533}{\cos 48.5^\circ \cdot \cos(20^\circ + 48.5^\circ)} = 1.92$$

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

$$F_{AE_{E\text{-}W}} = P_{AE} = \frac{1}{2} \times \overbrace{125 \text{ pcf}}^{\gamma} \times \overbrace{(3 \text{ ft})^2}^{H^2} \times \overbrace{1.92}^{K_{AE}} \times \overbrace{64 \text{ ft}}^{L} / \text{storage pad} = 69.1 \text{ K in E-W direction.}$$

$$F_{AE_{N\text{-}S}} = 69.1 \text{ K} \times \frac{30 \text{ ft}}{64 \text{ ft}} = 32.4 \text{ K in the N-S direction.}$$

WEIGHTS

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

Pad: Wp = 3 ft x 64 ft x 30 ft x 0.15 kips/ft^3 = 864 K

EARTHQUAKE ACCELERATIONS – PSHA 2,000-YR RETURN PERIOD

a_H = horizontal earthquake acceleration = 0.528g

a_V = vertical earthquake acceleration = 0.533g

CASK EARTHQUAKE LOADINGS

EQvc = -0.533 x 2,852 K = -1,520 K (minus sign signifies uplift force)

$EQhc_x$ = 1,855 K (acting short direction of pad, E-W) $Q_{xd\ max}$ in Table D-1(c) in Att B

$EQhc_y$ = 1,791 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, 1999), 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 1,066 K for Case III, based on the upper-bound value of μ = 0.8, as shown in the following table:

	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	-1,520	1,332	266	1,066	**1,066**
Case IV - EQ_v Down	2,852	1,520	4,372	874	3,498	**1,855** E-W 1,791 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 = -0.533 x 864 K = -461 K

EQhp = 0.528 x 864 K = 456 K

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

Minimum sliding resistance exists when EQvc and EQvp act in an upward direction (Case III), tending to unload the pad. For this case,

$$N = \underset{Wc}{2{,}852\ K} + \underset{Wp}{864\ K} + \underset{EQvc}{(-1{,}520\ K)} + \underset{EQvp}{(-461\ K)} = 1{,}735\ K$$

$$T = \underset{N}{1{,}735\ K} \times \tan \underset{\phi}{0^\circ} + \underset{c}{2.1\ ksf} \times \underset{B}{30\ ft} \times \underset{L}{64\ ft} = 4{,}032\ K$$

The driving force, V, is defined as:

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

The factor of safety against sliding is calculated as follows:

$$FS = \underset{T}{4{,}032\ K} \div (\underset{F_{AE}}{69.1\ K} + \underset{EQhp}{456\ K} + \underset{EQhc}{1{,}066\ K}) = 2.53$$

For this analysis, the value of EQhc was limited to the upper-bound value of the coefficient of friction, $\mu = 0.8$, 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, $N \text{ (normal force)} = \Sigma Fv = Wc + Wp + EQvc + EQvp$

$$N = \underset{Wc}{2{,}852\ K} + \underset{Wp}{864\ K} + \underset{EQvc}{1{,}520\ K} + \underset{EQvp}{461\ K} = 5{,}697\ K$$

$$T = \underset{N}{5{,}697\ K} \times \tan \underset{\phi}{0^\circ} + \underset{c}{2.1\ ksf} \times \underset{B}{30\ ft} \times \underset{L}{64\ ft}] = 4{,}032\ K$$

The driving force, V, is defined as:

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

The factor of safety against sliding is calculated as follows:

$$FS = \underset{T}{4{,}032\ K} \div (\underset{F_{AE}}{69.1\ K} + \underset{EQhp}{456\ K} + \underset{EQhc}{1{,}855\ K}) = 1.69$$

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 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 upper layer 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.

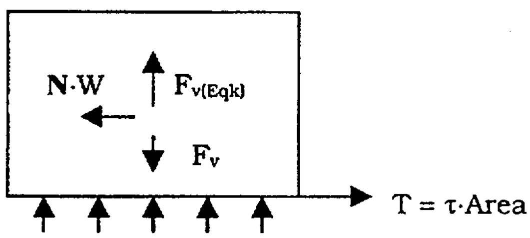
EVALUATION OF SLIDING ON DEEP SLIP SURFACE BENEATH PADS

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., $a_H = 0.528g$ and $a_V = 0.533g$. 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 = 0.528 x 48 = 25.3 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	0.211g	10.1	0.533g	0.211g	10.1
IIIB	0.211g	10.1	0.213g	0.528g	25.3
IIIC	0.528g	25.3	0.213g	0.211g	10.1

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 + 864 K = 3,716 kips

Earthquake Vertical Force, $F_{v\,Eqk} = a_v \times W/g$ = 0.533g x 3,716 K/g = 1,981 K

$$\phi = 30°$$

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

$$\mathbf{N} = [(\underset{F_v}{3{,}716} - \underset{F_{v\,Eqk}}{1{,}981}) \tan \underset{\phi}{30°}] / \underset{W}{3{,}716} = 0.270$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\underset{40\%\ N\text{-}S}{0.211^2} + \underset{40\%\ E\text{-}W}{0.211^2})} = 0.299g$

Resultant velocity in horizontal direction, $V = \sqrt{(\underset{40\%\ N\text{-}S}{10.1^2} + \underset{40\%\ E\text{-}W}{10.1^2})}$ = 14.3 in./sec

$$\Rightarrow \quad \mathbf{N / A} = 0.270 / 0.299 = 0.903$$

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

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

where g is in units of inches/sec².

$$\Rightarrow \quad u_m = \left(\frac{(14.3\,\text{in./sec})^2 \cdot (1 - 0.903)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.270} \right) = 0.1''$$

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 ~0.1 inches.

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

Static Vertical Force, F_v = W = 3,716 K

Earthquake Vertical Force, $F_{v(Eqk)}$ = 1,981 K x 0.40 = 792 K

$$\phi = 30°$$

$$\mathbf{N} = [(\underset{F_v}{3{,}716} - \underset{F_{v\,Eqk}}{792}) \tan \underset{\phi}{30°}] / \underset{W}{3{,}716} = 0.454$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\underset{40\%\ N\text{-}S}{0.211^2} + \underset{100\%\ E\text{-}W}{0.528^2})}\, g = 0.569g$

Resultant velocity in horizontal direction, $V = \sqrt{(\underset{40\%\ N\text{-}S}{10.1^2} + \underset{100\%\ E\text{-}W}{25.3^2})} = 27.2$ in./sec

$$\Rightarrow \quad \mathbf{N} / A = 0.454 / 0.569 = 0.798$$

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

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

$$\Rightarrow \quad u_m = \left(\frac{(27.2\,\text{in./sec})^2 \cdot (1 - 0.798)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.454} \right) = 0.43''$$

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 ~0.4 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

LOAD COMBINATION				DISPLACEMENT
Case IIIA	40% N-S	-100% Vert	40% E-W	0.1 inches
Case IIIB	40% N-S	-40% Vert	100% E-W	0.4 inches
Case IIIC	100% N-S	-40% Vert	40% E-W	0.4 inches

Assuming the cask storage pads are founded directly on a layer of cohesionless soils with $\phi = 30°$, the estimated relative displacement of the pads due to the design basis ground motion based on Newmark's method of estimating displacements of embankments and dams due to earthquakes ranges from ~0.1 inches to 0.4 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.

ALLOWABLE BEARING CAPACITY OF THE CASK STORAGE PADS

The bearing capacity for shallow foundations is determined using the general bearing capacity equation and associated factors, as referenced in Winterkorn and Fang (1975). The general bearing capacity equation is a modification of Terzaghi's bearing capacity equation, which was developed for strip footings and indicates that $q_{ult} = c \cdot N_c + q \cdot N_q + ½\gamma \cdot B \cdot N_\gamma$. The ultimate bearing capacity of soil consists of three components: 1) cohesion, 2) surcharge, and 3) friction, which are represented by the bearing capacity factors N_c, N_q, and N_γ. Terzaghi's bearing capacity equation has been enhanced by various investigators to incorporate shape, depth, and load inclination factors for different foundation geometries and loads as follows:

$$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + q\,N_q\,s_q\,d_q\,i_q + ½\,\gamma B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$$

where

q_{ult} = ultimate bearing capacity

c = cohesion or undrained strength

q = effective surcharge at bottom of foundation, = γD_f

γ= unit weight of soil

B = foundation width

s_c, s_q, s_γ = shape factors, which are a function of foundation width to length

d_c, d_q, d_γ = depth factors, which account for embedment effects

i_c, i_q, i_γ = load inclination factors

N_c, N_q, N_γ = bearing capacity factors, which are a function of ϕ.

γ in the third term is the unit weight of soil below the foundation, whereas the unit weight of the soil above the bottom of the footing is used in determining q in the second term.

BEARING CAPACITY FACTORS

Bearing capacity factors are computed based on relationships proposed by Vesic (1973), which are presented in Chapter 3 of Winterkorn and Fang (1975). The shape, depth and load inclination factors are calculated as follows:

$$N_q = e^{\pi \tan\phi} \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$N_c = (N_q - 1)\ \cot\phi\text{, but} = 5.14 \text{ for } \phi = 0.$$

$$N_\gamma = 2\ (N_q + 1)\ \tan\phi$$

SHAPE FACTORS (FOR $L>B$)

$$s_c = 1 + \frac{B}{L} \cdot \frac{N_q}{N_c}$$

$$s_q = 1 + \frac{B}{L} \tan\phi$$

$$s_\gamma = 1 - 0.4\frac{B}{L}$$

DEPTH FACTORS (FOR $\frac{D_f}{B} \leq 1$)

$$d_c = d_q - \frac{(1 - d_q)}{N_q \cdot \tan\phi} \text{ for } \phi > 0 \text{ and } d_c = 1 + 0.4\left(\frac{D_f}{B}\right) \text{ for } \phi = 0.$$

$$d_q = 1 + 2\tan\phi \cdot (1 - \sin\phi)^2 \cdot \left(\frac{D_f}{B}\right)$$

$$d_\gamma = 1$$

INCLINATION FACTORS

$$i_q = \left(1 - \frac{F_H}{F_V + B'L'c\cot\phi}\right)^m$$

$$i_c = i_q - \frac{(1 - i_q)}{N_c \cdot \tan\phi} \text{ for } \phi > 0 \text{ and } i_c = 1 - \left(\frac{m F_H}{B'L'c N_c}\right) \text{ for } \phi = 0$$

$$i_\gamma = \left(1 - \frac{F_H}{F_V + B'L'c\cot\phi}\right)^{m+1}$$

where F_H and F_V are the total horizontal and vertical forces acting on the footing.

STATIC BEARING CAPACITY OF THE CASK STORAGE PADS

The following pages present the details of the bearing capacity analyses for the static load cases. These cases are identified as follows:

Case IA Static using undrained strength parameters (ϕ = 0° & c = 2.2 ksf).

Case IB Static using effective-stress strength parameters (ϕ = 30° & c = 0).

Allowable Bearing Capacity of Cask Storage Pads

Static Analysis: **Case** **IA - Static** **0 % in X, 0 % in Y, 0 % in Z**

Soil Properties: $c =$ **2,200** Cohesion (psf)

Undrained Strength $\phi =$ **0.0** Friction Angle (degrees)

$\gamma =$ **80** Unit weight of soil (pcf)

$\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

Foundation Properties: $B' =$ **30.0** Footing Width - ft (E-W) $L' =$ **64.0** Length - ft (N-S

$D_f =$ **2.7** Depth of Footing (ft)

$\beta =$ **0.0** Angle of load inclination from vertical (degrees)

$FS =$ **3.0** Factor of Safety required for $q_{allowable}$.

$F_V =$ **3,716** k $EQ_V =$ **0** k

$EQ_{H\,E\text{-}W} =$ **0** k $EQ_{H\,N\text{-}S} =$ **0** k

$$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.81	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.04	Eq 3.27

No inclined loads; therefore, $i_c = i_q = i_\gamma = 1.0$.

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**13,056**	**psf =**	**0**	**+**	**6,497**	**+**	**21,842**

q_{all} = 4,350 psf = q_{ult} / FS

q_{actual} = 1,936 psf = $(F_v + EQ_v) / (B' \times L')$

FS_{actual} = 6.75 = q_{ult} / q_{actual} > 3 Hence OK

Allowable Bearing Capacity of Cask Storage Pads

Static Analysis: **Case** **IB - Static** **0 % in X, 0 % in Y, 0 % in Z**

Soil Properties:	$c =$	**0**	Cohesion (psf)
Effective-Stress Strengths	$\phi =$	**30.0**	Friction Angle (degrees)
	$\gamma =$	**80**	Unit weight of soil (pcf)
	$\gamma_{surch} =$	**100**	Unit weight of surcharge (pcf)
Foundation Properties:	$B' =$	**30.0**	Footing Width - ft (E-W) $L' =$ **64.0** Length - ft (N-S
	$D_f =$	**2.7**	Depth of Footing (ft)
	$\beta =$	**0.0**	Angle of load inclination from vertical (degrees)
	$FS =$	**3.0**	Factor of Safety required for $q_{allowable}$.
	$F_V =$	**3,716**	k $EQ_V =$ **0** k
	$EQ_{H\ E\text{-}W} =$	**0**	k $EQ_{H\ N\text{-}S} =$ **0** k

$$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but $= 5.14$ for $\phi = 0$	=	30.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	18.40	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	22.40	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.29	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.27	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.81	"
For $D_f/B \leq 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.03	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	1.03	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	N/A	Eq 3.27

No inclined loads; therefore, $i_c = i_q = i_\gamma = 1.0$.

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**28,340**	**psf =**	**0**	**+**	**6,497**	**+**	**21,842**
q_{all} =	**9,440**	**psf = q_{ult} / FS**					
q_{actual} =	**1,936**	**psf = $(F_v + EQ_v)/(B' \times L')$**					
FS_{actual} =	**14.64**	**= q_{ult}/q_{actual}**			**> 3**		**Hence OK**

Table 2.6-6 presents a summary of the results of the bearing capacity analyses for the static load cases. As indicated in this table, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 4 ksf. However, loading the storage pads to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume $\phi = 0°$ and c = 2.2 ksf, as measured in the UU tests that are reported in Attachment 2 of Appendix 2A of the SAR, to model the end of construction. Using the estimated effective-stress strength of $\phi = 30°$ and c = 0 results in higher allowable bearing pressures. As shown in Table 2.6-6, the gross allowable bearing capacities of the cask storage pads for static loads for this soil strength is greater than 9 ksf.

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS

Dynamic bearing capacity analyses are performed using two different sets of dynamic forces. In the first set of analyses, which are presented on Pages 32 to 45, the dynamic loads are determined as the inertial forces applicable for the peak ground accelerations from the design basis ground motion. The second set of analyses use the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 1999), for the pad supporting 2 casks, 4 casks, and 8 casks.

BASED ON INERTIAL FORCES

This section presents the analysis of the allowable bearing capacity of the pad for supporting the dynamic loads defined as the inertial forces applicable for the peak ground accelerations from the design basis ground motion. The total vertical force includes the static weight of the pad and eight fully loaded casks ± the vertical inertial forces due to the earthquake. The vertical inertial force is calculated as a_V x [weight of the pad + cask dead loads], multiplied by the appropriate factor (±40% or ±100%) for the load case. In these analyses, the minus sign for the percent loading in the vertical direction signifies uplift forces, which tend to unload the pad. Similarly, the horizontal inertial forces are calculated as a_H x [weight of the pad + cask dead loads], multiplied by the appropriate factor (40% or 100%) for the load case. The horizontal inertial force from the casks was confirmed to be less than the maximum force that can be transmitted from the cask to the pad through friction for each of these load cases. This friction force was calculated based on the upper-bound value of the coefficient of friction between the casks and the storage pad considered in the HI-STORM cask stability analysis (μ = 0.8, as shown in SAR Section 8.2.1.2, Accident Analysis) x the normal force acting between the casks and the pad.

The lower-bound friction case (discussed in SAR Section 4.2.3.5.1B), wherein μ between the steel bottom of the cask and the top of the concrete storage pad = 0.2, results in lower horizontal forces being applied at the top of the pad. This decreases the inclination of the load applied to the pad, which results in increased bearing capacity. Therefore, the dynamic bearing capacity analyses are not performed for μ = 0.2.

Table 2.6-7 presents the results of the bearing capacity analyses for the following cases, which include static loads plus inertial forces due to the earthquake. Because the *in situ* fine-grained soils are not expected to fully drain during the rapid cycling of load during the earthquake, these cases are analyzed using the undrained strength that was measured in unconsolidated-undrained triaxial tests (ϕ = 0° and c = 2.2 ksf).

Case II	100%	N-S direction,	0%	Vertical direction,	100%	E-W direction.
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.
Case IVA	40%	N-S direction,	100%	Vertical direction,	40%	E-W direction.
Case IVB	40%	N-S direction,	40%	Vertical direction,	100%	E-W direction.
Case IVC	100%	N-S direction,	40%	Vertical direction,	40%	E-W direction

Case II: 100% N-S, 0% Vertical, 100% E-W

Determine forces and moments due to earthquake.

$F_v = \overset{Wc}{2{,}852\ K} + \overset{Wp}{864\ K} = 3{,}716\ K$ and $EQ_v = 0$ for this case.

$EQ_{H\ Pad} = \overset{a_H}{0.528} \times \overset{HT_{pad}}{3'} \times \overset{B}{30'} \times \overset{L}{64'} \times \overset{\gamma_{conc}}{0.15\ kcf} = 456\ K$

$EQhc = \text{Minimum of } [\underbrace{\overset{a_H}{0.528} \times \overset{Wc}{2{,}852\ K}}_{1{,}506\ K}\ \&\ \underbrace{\overset{\mu}{0.8} \times \overset{Nc}{2{,}852\ K}}_{2{,}282K}] \Rightarrow EQhc = 1{,}506\ K$

Note, Nc = Wc in this case, since $a_V = 0$.

$EQ_{H\ N\text{-}S} = \overset{EQhp}{456\ K} + \overset{EQhc}{1{,}506\ K} = 1{,}962\ K$

The horizontal components are the same for this case; therefore, $EQ_{H\ E\text{-}W} = EQ_{H\ N\text{-}S}$

Combine these horizontal components to calculate F_H:

$$\Rightarrow \quad F_H = \sqrt{EQ^2_{H\ E\text{-}W} + EQ^2_{H\ N\text{-}S}} = \sqrt{1{,}962^2 + 1{,}962^2} = 2{,}775\ K$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb.

$$\Delta b = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 1{,}506\ K}{2{,}852\ K + 0} = 5.19\ ft$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{a_H}{0.528} \times \overset{Wp}{864\ K} + 3' \times \overset{EQhc}{1{,}506\ K} + \overset{\Delta b}{5.19'} \times (\overset{Wc}{2{,}852K} + \overset{EQvc}{0})$$

$$= 684\ ft\text{-}K + 4{,}518\ ft\text{-}K + 14{,}804\ ft\text{-}K = 20{,}006\ ft\text{-}K$$

The horizontal forces are the same N-S and E-W for this case; therefore,

$$\Sigma M_{@E\text{-}W} = \Sigma M_{@N\text{-}S} = 20{,}006\ ft\text{-}K$$

Determine $q_{allowable}$ for FS = 1.1.

CALCULATION IDENTIFICATION NUMBER				PAGE 33
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	04 - 6		

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON INERTIAL FORCES

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case II — **100 % in X, 0 % in Y, 100 % in Z**

Soil Properties:

$c =$	**2,200**	Cohesion (psf)	
$\phi =$	**0.0**	Friction Angle (degrees)	
$\gamma =$	**80**	Unit weight of soil (pcf)	
$\gamma_{surch} =$	**100**	Unit weight of surcharge (pcf)	

Foundation Properties:

$B' =$	**19.2**	Footing Width - ft (E-W)	L' = **53.2** Length - ft (N-S)
$D_f =$	**2.7**	Depth of Footing (ft)	
$\beta =$	**27.8**	Angle of load inclination from vertical (degrees)	
$FS =$	**1.1**	Factor of Safety required for $q_{allowable}$.	
$F_V =$	**3,716**	k	$EQ_V =$ **0** k
$EQ_{H\,E\text{-}W} =$	**1,962**	k & $EQ_{H\,N\text{-}S} =$ **1,962** k →	**2,775** k for F_H

$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\,(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.07	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.86	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2\,D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.06	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.79 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.50	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.64	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**8,459**	**psf =**	**8,188**	+	**271**	+	**0**
q_{all} =	**7,690**	**psf = q_{ult} / FS**					
q_{actual} =	**3,630**	**psf = $(F_v + EQ_v)/(B' \times L')$**					
FS_{actual} =	**2.33**	**= q_{ult}/q_{actual}**			**> 1.1**	**Hence OK**	

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

Determine forces and moments due to earthquake.

$$EQ_V = -100\% \times \overset{a_V}{0.533} \times (\overset{Wp}{864\text{ K}} + \overset{Wc}{2{,}852\text{ K}}) = -1{,}981\text{ K}$$

$$EQhp = \overset{a_H}{0.528} \times \overset{Wc}{864\text{ K}} = 456\text{ K}$$

Normal force at base of the cask = Cask DL = 2,852 K

— Cask EQvc = -1. x 0.533 x 2,852 K = - 1,520 K = a_V x Wc

⇒ Nc = 1,332 K

⇒ $F_{EQ\,\mu=0.8} = 0.8 \times 1{,}332\text{ K} = 1{,}066\text{ K}$

$$EQhc = \text{Minimum of } [\underbrace{\overset{a_H}{0.528} \times \overset{Wc}{2{,}852\text{ K}}}_{1{,}506\text{ K}} \ \& \ \underbrace{\overset{\mu}{0.8} \times \overset{Nc}{1{,}332\text{K}}}_{1{,}066\text{K}}]$$

Note: Use only 40% of the horizontal earthquake forces in this case.

40% of 1,506 K = 602 K, which is < $F_{EQ\,\mu=0.8}$; therefore, EQhc =1,506 K

$$\Rightarrow \quad EQ_{H\,N\text{-}S} = \overset{40\%\text{ of}}{0.4} \times [\overset{EQhp}{456\text{ K}} + \overset{EQhc}{1{,}506\text{ K}}] = 785\text{ K}$$

Since horizontal components are the same for this case, $EQ_{H\,E\text{-}W} = EQ_{H\,N\text{-}S}$

$$\Rightarrow \quad F_H = \sqrt{EQ^2_{H\,E\text{-}W} + EQ^2_{H\,N\text{-}S}} = \sqrt{785^2 + 785^2} = 1{,}110\text{ K}$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E-W} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\text{ K}}{2{,}852\text{ K} - 1. \times 0.533 \times 2{,}852\text{ K}} = 4.45\text{ ft}$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{40\%\ a_H}{0.4 \times 0.528} \times \overset{Wp}{864\text{ K}} + \overset{40\%\ EQhc}{0.4 \times 3' \times 1{,}506\text{ K}} + \overset{\Delta b}{4.45'} \times (\overset{Wc}{2{,}852\text{K}} - \overset{EQvc}{1{,}520\text{ K}})$$

$$= 274\text{ ft-K} + 1{,}807\text{ ft-K} + 5{,}927\text{ ft-K} = 8{,}008\text{ ft-K}$$

The horizontal forces are the same N-S and E-W for this case; therefore,

$$\Sigma M_{@E\text{-}W} = \Sigma M_{@N\text{-}S} = 8{,}008\text{ ft-K}$$

Determine $q_{allowable}$ for FS = 1.1.

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DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON INERTIAL FORCES

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IIIA $\boxed{\textbf{40 \% in X, -100 \% in Y, 40 \% in Z}}$

Soil Properties: $c =$ **2,200** Cohesion (psf)

$\phi =$ **0.0** Friction Angle (degrees)

$\gamma =$ **80** Unit weight of soil (pcf)

$\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

Foundation Properties: $B' =$ **20.8** Footing Width - ft (E-W) $L' =$ **54.8** Length - ft (N-S)

$D_f =$ **2.7** Depth of Footing (ft)

$\beta =$ **24.3** Angle of load inclination from vertical (degrees)

$FS =$ **1.1** Factor of Safety required for $q_{allowable}$.

$F_V =$ **3,716** k $EQ_V =$ **-1,981** k

$EQ_{H\,E\text{-}W} =$ **785** k & $EQ_{H\,N\text{-}S} =$ **785** k → **1,110** k for F_H

$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$ **General Bearing Capacity Equation, based on Winterkorn & Fang (1975)**

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.07	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.85	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.79	rad
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.50	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.87	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**11,394**	**psf =**	**11,123**	+	**271**	+	**0**
q_{all} =	**10,350**	**psf = q_{ult} / FS**					
q_{actual} =	**1,525**	**psf = $(F_v + EQ_v)/(B' \times L')$**					
FS_{actual} =	**7.47**	**= q_{ult}/q_{actual}**			> 1.1	**Hence OK**	

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DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON INERTIAL FORCES

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

Determine forces and moments due to earthquake.

$$EQ_V = -40\% \times \overset{a_V}{0.533} \times (\overset{Wp}{864\text{ K}} + \overset{Wc}{2{,}852\text{ K}}) = -792\text{ K}$$

Normal force at base of the cask = Cask DL = 2,852 K

— 40% of Cask EQvc = -0.4 x 0.533 x 2,852 K = - 608 K = 40% of a_V x Wc

$\Rightarrow$ Nc = 2,244 K

$\Rightarrow$ $F_{EQ\,\mu=0.8} = 0.8 \times 2{,}224\text{ K} = 1{,}795\text{ K}$

$$EQhc = \text{Min of } [\underbrace{\overset{a_H}{0.528} \times \overset{Wc}{2{,}852\text{ K}}}_{1{,}506\text{ K}}\ \&\ \underbrace{\overset{\mu}{0.8} \times \overset{Nc}{2{,}244\text{ K}}}_{1{,}795\text{K}}] \Rightarrow EQhc = 1{,}506\text{ K, since it is} < F_{EQ\,\mu=0.8}$$

Using 40% of N-S: 40% of [EQhp EQhc]

$$\Rightarrow EQ_{H\,N\text{-}S} = 0.4 \times [456\text{ K} + 1{,}506\text{ K}] = 785\text{ K}$$

Using 100% of E-W: 100% of [EQhp EQhc]

$$\Rightarrow EQ_{H\,E\text{-}W} = 1.0 \times [456\text{ K} + 1{,}506\text{ K}] = 1{,}962\text{ K}$$

$$\Rightarrow F_H = \sqrt{EQ^2{}_{H\,E\text{-}W} + EQ^2{}_{H\,N\text{-}S}} = \sqrt{1{,}962^2 + 785^2} = 2{,}113\text{ K}$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E-W} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 1.0 \times 1{,}506\text{ K}}{2{,}852\text{ K} - 0.4 \times 0.533 \times 2{,}852\text{ K}} = 6.60\text{ ft}$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{100\%\ a_H}{0.528} \times \overset{Wp}{864\text{ K}} + 3' \times \overset{100\%\ EQhc}{1{,}506\text{ K}} + \overset{\Delta b}{6.60'} \times (\overset{Wc}{2{,}852\text{K}} - \overset{40\%\ EQvc}{0.4 \times 1{,}520\text{ K}})$$

$$= 684\text{ ft-K} + 4{,}518\text{ ft-K} + 14{,}810\text{ ft-K} = 20{,}012\text{ ft-K}$$

$$\Delta b_{N-S} = \frac{9.83' \times 40\%\, EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\text{ K}}{2{,}852\text{ K} - 0.4 \times 0.533 \times 2{,}852\text{ K}} = 2.64\text{ ft}$$

$$\Sigma M_{@E\text{-}W} = 1.5' \times \overset{40\%\ a_H}{0.4 \times 0.528} \times \overset{Wp}{864\text{ K}} + 3' \times \overset{40\%\ EQhc}{0.4 \times 1{,}506\text{ K}} + \overset{\Delta b}{2.64'} \times (\overset{Wc}{2{,}852\text{K}} - \overset{40\%\ EQvc}{0.4 \times 1{,}520\text{ K}})$$

$$= 274\text{ ft-K} + 1{,}807\text{ ft-K} + 5{,}924\text{ ft-K} = 8{,}005\text{ ft-K}$$

Determine $q_{allowable}$ for FS = 1.1.

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON INERTIAL FORCES

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IIIB $\boxed{\textbf{40 \% in X,} \quad \textbf{-40 \% in Y,} \quad \textbf{100 \% in Z}}$

Soil Properties:

$c =$ **2,200** Cohesion (psf)

$\phi =$ **0.0** Friction Angle (degrees)

$\gamma =$ **80** Unit weight of soil (pcf)

$\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **16.3** Footing Width - ft (E-W) $L' =$ **58.5** Length - ft (N-S)

$D_f =$ **2.7** Depth of Footing (ft)

$\beta =$ **33.9** Angle of load inclination from vertical (degrees)

$FS =$ **1.1** Factor of Safety required for $q_{allowable}$.

$F_V =$ **3,716** k $EQ_V =$ -792 k

$EQ_{H\ E\text{-}W} =$ **1,962** k & $EQ_{H\ N\text{-}S} =$ 785 k → **2,113** k for F_H

$$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\ \cot(\phi)$, but $= 5.14$ for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\ \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\,(N_q + 1)\ \tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.05	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.89	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\ (1 - \sin\phi)^2\,D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.07	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	1.19 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.63	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.68	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**8,926**	**psf =**	**8,655**	+	**271**	+	**0**
q_{all} =	**8,110**	**psf = q_{ult} / FS**					
q_{actual} =	**3,062**	**psf = $(F_v + EQ_v)/(B' \times L')$**					
FS_{actual} =	**2.92**	**$= q_{ult}/q_{actual}$**			**> 1.1**	**Hence OK**	

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

Determine forces and moments due to earthquake.

$$EQ_V = -40\% \times \overset{a_V}{0.533} \times [\overset{Wp}{864\text{ K}} + \overset{Wc}{2{,}852\text{ K}}] = -792\text{ K}$$

Normal force at base of the cask = Cask DL = 2,852 K

— 40% of Cask EQvc = -0.4 x 0.533 x 2,852 K = - 608 K = 40% of a_V x Wc

$\Rightarrow$ Nc = 2,244 K

$\Rightarrow$ $F_{EQ\,\mu=0.8}$ = 0.8 x 2,224 K = 1,795 K

$$EQhc = \text{Min of } [\underbrace{\overset{a_H}{0.528} \times \overset{Wc}{2{,}852\text{ K}}}_{1{,}506\text{ K}} \;\&\; \underbrace{\overset{\mu}{0.8} \times \overset{Nc}{2{,}244\text{ K}}}_{1{,}795\text{K}}] \Rightarrow EQhc = 1{,}506\text{ K, since it is} < F_{EQ\,\mu=0.8}$$

Using 100% of N-S:

$$\Rightarrow EQ_{H\,N\text{-}S} = \overset{100\%\text{ of}}{1.0} \times [\overset{EQhp}{456\text{ K}} + \overset{EQhc}{1{,}506\text{ K}}] = 1{,}962\text{ K}$$

Using 40% of E-W:

$$\Rightarrow EQ_{H\,E\text{-}W} = \overset{40\%\text{ of}}{0.4} \times [\overset{EQhp}{456\text{ K}} + \overset{EQhc}{1{,}506\text{ K}}] = 785\text{ K}$$

$$\Rightarrow F_H = \sqrt{EQ^2{}_{H\,E\text{-}W} + EQ^2{}_{H\,N\text{-}S}} = \sqrt{785^2 + 1{,}962^2} = 2{,}113\text{ K}$$

Determine moments acting on pad due to casks

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E-W} = \frac{9.83' \times 40\%\, EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\text{ K}}{2{,}852\text{ K} - 0.4 \times 0.533 \times 2{,}852\text{ K}} = 2.64\text{ ft}$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{40\%\,a_H}{0.4 \times 0.528} \times \overset{Wp}{864\text{ K}} + 3' \times \overset{40\%\,EQhc}{0.4 \times 1{,}506\text{ K}} + \overset{\Delta b}{2.64'} \times (\overset{Wc}{2{,}852\text{K}} - \overset{40\%\,EQvc}{0.4 \times 1{,}520\text{ K}})$$

$$= 274\text{ ft-K} + 1{,}807\text{ ft-K} + 5{,}924\text{ ft-K} = 8{,}005\text{ ft-K}$$

$$\Delta b_{N-S} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 1.0 \times 1{,}506\text{ K}}{2{,}852\text{ K} - 0.4 \times 0.533 \times 2{,}852\text{ K}} = 6.60\text{ ft}$$

$$\Sigma M_{@E\text{-}W} = 1.5' \times \overset{100\%\,a_H}{0.528} \times \overset{Wp}{864\text{ K}} + 3' \times \overset{100\%\,EQhc}{1{,}506\text{ K}} + \overset{\Delta b}{6.60'} \times (\overset{Wc}{2{,}852\text{K}} - \overset{40\%\,EQvc}{0.4 \times 1{,}520\text{ K}})$$

$$= 684\text{ ft-K} + 4{,}518\text{ ft-K} + 14{,}810\text{ ft-K} = 20{,}012\text{ ft-K}$$

Determine $q_{allowable}$ for FS = 1.1.

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DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON INERTIAL FORCES

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IIIC — **100 % in X, -40 % in Y, 40 % in Z**

Soil Properties:
- $c =$ **2,200** Cohesion (psf)
- $\phi =$ **0.0** Friction Angle (degrees)
- $\gamma =$ **80** Unit weight of soil (pcf)
- $\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

Foundation Properties:
- $B' =$ **24.5** Footing Width - ft (E-W) $L' =$ **50.3** Length - ft (N-S)
- $D_f =$ **2.7** Depth of Footing (ft)
- $\beta =$ **15.0** Angle of load inclination from vertical (degrees)
- $FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **3,716** k $EQ_V =$ **-792** k
- $EQ_{H\ E\text{-}W} =$ **785** k & $EQ_{H\ N\text{-}S} =$ **1,962** k → **2,113** k for F_H

$$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but $= 5.14$ for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi\tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.81	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.04	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.38	rad
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.37	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m F_H / B'L'c N_c)$	=	0.79	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	10,518	psf =	10,247	+	271	+	0
q_{all} =	9,560	psf = q_{ult} / FS					
q_{actual} =	2,369	psf = $(F_v + EQ_v)/(B' \times L')$					
FS_{actual} =	4.44	= q_{ult}/q_{actual}			> 1.1	Hence OK	

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Case IVA: 40% N-S, 100% Vertical, 40% E-W

Determine forces and moments due to earthquake.

$$EQ_V = 100\% \times \underbrace{0.533}_{a_V} \times (\underbrace{864\text{ K}}_{Wp} + \underbrace{2{,}852\text{ K}}_{Wc}) = 1{,}981\text{ K}$$

$$EQhp = \underbrace{0.528}_{a_H} \times \underbrace{864\text{ K}}_{Wc} = 456\text{ K}$$

Normal force at base of the cask = Cask DL = 2,852 K

+ Cask EQvc = 1. x 0.533 x 2,852 K = + 1,520 K = a_V x Wc

⇒ Nc = 3,498 K

$$\Rightarrow \quad F_{EQ\,\mu=0.8} = 0.8 \times 4{,}372\text{ K} = 3{,}498\text{ K}$$

$$EQhc = \text{Min of } [\underbrace{\underbrace{0.528}_{a_H} \times \underbrace{2{,}852\text{ K}}_{Wc}}_{1{,}506\text{ K}} \;\&\; \underbrace{\underbrace{0.8}_{\mu} \times \underbrace{4{,}372\text{ K}}_{Nc}}_{3{,}498\text{K}}] \Rightarrow EQhc = 1{,}506\text{ K, since it is} < F_{EQ\,\mu=0.8}$$

$$\Rightarrow \quad EQ_{H\,N\text{-}S} = \underbrace{0.4}_{40\%\text{ of}} \times [\underbrace{456\text{ K}}_{EQhp} + \underbrace{1{,}506\text{ K}}_{EQhc}] = 785\text{ K}$$

Since horizontal components are the same for this case, $EQ_{H\,E\text{-}W} = EQ_{H\,N\text{-}S}$

$$\Rightarrow \quad F_H = \sqrt{EQ^2{}_{H\,E\text{-}W} + EQ^2{}_{H\,N\text{-}S}} = \sqrt{785^2 + 785^2} = 1{,}110\text{ K}$$

Determine moments acting on pad due to casks.

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E\text{-}W} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\text{ K}}{2{,}852\text{ K} + 1. \times 0.533 \times 2{,}852\text{ K}} = 1.35\text{ ft}$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \underbrace{0.4 \times 0.528}_{40\%\ a_H} \times \underbrace{864\text{ K}}_{Wp} + \underbrace{0.4 \times 3' \times 1{,}506\text{ K}}_{40\%\ EQhc} + \underbrace{1.35'}_{\Delta b} \times (\underbrace{2{,}852\text{K}}_{Wc} + \underbrace{1{,}520\text{ K}}_{EQvc})$$

$$= 274\text{ ft-K} + 1{,}807\text{ ft-K} + 5{,}921\text{ ft-K} = 8{,}002\text{ ft-K}$$

The horizontal forces are the same N-S and E-W for this case; therefore,

$$\Sigma M_{@E\text{-}W} = \Sigma M_{@N\text{-}S} = 8{,}002\text{ ft-K}$$

Determine $q_{allowable}$ for FS = 1.1.

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DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON INERTIAL FORCES

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IVA — **40 % in X, 100 % in Y, 40 % in Z**

Soil Properties:

$c =$ **2,200** Cohesion (psf)

$\phi =$ **0.0** Friction Angle (degrees)

$\gamma =$ **80** Unit weight of soil (pcf)

$\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **27.2** Footing Width - ft (E-W) $L' = 61.2$ Length - ft (N-S)

$D_f =$ **2.7** Depth of Footing (ft)

$\beta =$ **7.8** Angle of load inclination from vertical (degrees)

$FS =$ **1.1** Factor of Safety required for $q_{allowable}$.

$F_V =$ **3,716** k $EQ_V =$ **1,981** k

$EQ_{H\,E\text{-}W} =$ **785** k & $EQ_{H\,N\text{-}S} =$ **785** k → **1,110** k for F_H

$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but $= 5.14$ for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.82	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.04	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.79 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.50	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.91	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross $q_{ult} =$	11,915	psf =	11,645	+	271	+	0

$q_{all} =$ 10,830 psf $= q_{ult}/FS$

$q_{actual} =$ 3,424 psf $= (F_v + EQ_v)/(B' \times L')$

$FS_{actual} =$ 3.48 $= q_{ult}/q_{actual}$ > 1.1 Hence OK

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON INERTIAL FORCES

Case IVB: 40% N-S, 40% Vertical, 100% E-W

Determine forces and moments due to earthquake.

$$EQ_V = 0.4 \times \overset{a_V}{0.533} \times (\overset{Wp}{864\text{ K}} + \overset{Wc}{2{,}852\text{ K}}) = 792\text{ K}$$

Normal force at base of the cask = Cask DL = 2,852 K

+ 40% of Cask EQvc = +0.4 x 0.533 x 2,852 K = + 608 K = 40% of a_V x Wc

$\Rightarrow$ Nc = 3,460 K

$\Rightarrow$ $F_{EQ\ \mu=0.8} = 0.8 \times 3{,}460\text{ K} = 2{,}768\text{ K}$

$$EQhc = \text{Min of } [\underbrace{\overset{a_H}{0.528} \times \overset{Wc}{2{,}852\text{ K}}}_{1{,}506\text{ K}} \ \& \ \underbrace{\overset{\mu}{0.8} \times \overset{Nc}{3{,}460\text{ K}}}_{2{,}768\text{K}}] \Rightarrow EQhc = 1{,}506\text{ K, since it is} < F_{EQ\ \mu=0.8}$$

Using 40% of N-S:

$$\Rightarrow \quad EQ_{H\ N\text{-}S} = \overset{40\%\text{ of}}{0.4} \times [\overset{EQhp}{456\text{ K}} + \overset{EQhc}{1{,}506\text{ K}}] = 785\text{ K}$$

Using 100% of E-W:

$$\Rightarrow \quad EQ_{H\ E\text{-}W} = \overset{100\%\text{ of}}{1.0} \times [\overset{EQhp}{456\text{ K}} + \overset{EQhc}{1{,}506\text{ K}}] = 1{,}962\text{ K}$$

$$\Rightarrow \quad F_H = \sqrt{EQ^2_{H\ E\text{-}W} + EQ^2_{H\ N\text{-}S}} = \sqrt{1{,}962^2 + 785^2} = 2{,}113\text{ K}$$

Determine moments acting on pad due to casks

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E\text{-}W} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 1.0 \times 1{,}506\text{ K}}{2{,}852\text{ K} + 0.4 \times 0.533 \times 2{,}852\text{ K}} = 4.28\text{ ft}$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \overset{100\%\ a_H}{0.528} \times \overset{Wp}{864\text{ K}} + 3' \times \overset{100\%\ EQhc}{1{,}506\text{ K}} + \overset{\Delta b}{4.28'} \times (\overset{Wc}{2{,}852\text{K}} + \overset{40\%\ EQvc}{0.4 \times 1{,}520\text{ K}})$$

$$= 684\text{ ft-K} + 4{,}518\text{ ft-K} + 14{,}810\text{ ft-K} = 20{,}012\text{ ft-K}$$

$$\Delta b_{N\text{-}S} = \frac{9.83' \times 40\%\ EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\text{ K}}{2{,}852\text{ K} + 0.4 \times 0.533 \times 2{,}852\text{ K}} = 1.71\text{ ft}$$

$$\Sigma M_{@E\text{-}W} = 1.5' \times \overset{40\%\ a_H}{0.4 \times 0.528} \times \overset{Wp}{864\text{ K}} + 3' \times \overset{40\%\ EQhc}{0.4 \times 1{,}506\text{ K}} + \overset{\Delta b}{1.71'} \times (\overset{Wc}{2{,}852\text{K}} + \overset{40\%\ EQvc}{0.4 \times 1{,}520\text{ K}})$$

$$= 274\text{ ft-K} + 1{,}807\text{ ft-K} + 5{,}917\text{ ft-K} = 7{,}998\text{ ft-K}$$

Determine $q_{allowable}$ for FS = 1.1.

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DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON INERTIAL FORCES

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IVB **40 % in X, 40 % in Y, 100 % in Z**

Soil Properties:

$c =$	**2,200**	Cohesion (psf)	
$\phi =$	**0.0**	Friction Angle (degrees)	
$\gamma =$	**80**	Unit weight of soil (pcf)	
$\gamma_{surch} =$	**100**	Unit weight of surcharge (pcf)	

Foundation Properties:

$B' =$	**21.1**	Footing Width - ft (E-W)	$L' =$ **60.5** Length - ft (N-S)
$D_f =$	**2.7**	Depth of Footing (ft)	
$\beta =$	**23.5**	Angle of load inclination from vertical (degrees)	
$FS =$	**1.1**	Factor of Safety required for $q_{allowable}$.	
$F_V =$	**3,716** k	$EQ_V =$ **792** k	
$EQ_{H\ E\text{-}W} =$	**1,962** k	& $EQ_{H\ N\text{-}S} =$ 785 k →	**2,113** k for F_H

$q_{ult} = c\, N_c\, s_c\, d_c\, i_c + \gamma_{surch}\, D_f\, N_q\, s_q\, d_q\, i_q + 1/2\,\gamma\, B\, N_\gamma\, s_\gamma\, d_\gamma\, i_\gamma$ — **General Bearing Capacity Equation, based on Winterkorn & Fang (1975)**

$N_c = (N_q - 1)\ \cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\ \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\ (N_q + 1)\ \tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.07	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\ (B/L)$	=	0.86	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\ (1 - \sin\phi)^2\ D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1 - d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\ (D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	1.19 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.63	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.76	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross $q_{ult} =$	9,937	psf =	9,666	+	271	+	0
$q_{all} =$	9,030	psf = q_{ult} / FS					
$q_{actual} =$	3,530	psf = $(F_v + EQ_v)/(B' \times L')$					
$FS_{actual} =$	2.81	= q_{ult}/q_{actual}			> 1.1	Hence OK	

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Case IVC: 100% N-S, 40% Vertical, 40% E-W

Determine forces and moments due to earthquake.

$EQ_V = 0.4 \times \underset{a_V}{0.533} \times [\underset{Wp}{864\ K} + \underset{Wc}{2{,}852\ K}] = 792\ K$

Normal force at base of the cask = Cask DL = 2,852 K

+ 40% of Cask EQvc = -0.4 x 0.533 x 2,852 K = + 608 K = 40% of a_V x Wc

$\Rightarrow$ Nc = 3,460 K

$\Rightarrow$ $F_{EQ\ \mu=0.8} = 0.8 \times 3{,}460\ K = 2{,}768\ K$

$EQhc = \text{Min of } [\underset{1{,}506\ K}{\underset{a_H}{0.528} \times \underset{Wc}{2{,}852\ K}} \ \&\ \underset{2{,}768\ K}{\underset{\mu}{0.8} \times \underset{Nc}{3{,}460\ K}}] \Rightarrow EQhc = 1{,}506\ K$, since it is $< F_{EQ\ \mu=0.8}$

Using 100% of N-S:

$\Rightarrow$ $EQ_{H\ N\text{-}S} = \underset{100\%\ of}{1.0} \times [\underset{EQhp}{456\ K} + \underset{EQhc}{1{,}506\ K}] = 1{,}962\ K$

Using 40% of E-W:

$\Rightarrow$ $EQ_{H\ E\text{-}W} = \underset{40\%\ of}{0.4} \times [\underset{EQhp}{456\ K} + \underset{EQhc}{1{,}506\ K}] = 785\ K$

$$\Rightarrow \quad F_H = \sqrt{EQ^2_{H\ E\text{-}W} + EQ^2_{H\ N\text{-}S}} = \sqrt{785^2 + 1{,}962^2} = 2{,}113\ K$$

Determine moments acting on pad due to casks

See Figure 6 for identification of Δb. Note: EQvc = 0.533 x 2,852 K = 1,520 K

$$\Delta b_{E-W} = \frac{9.83' \times 40\%\, EQhc}{Wc + EQvc} = \frac{9.83' \times 0.4 \times 1{,}506\ K}{2{,}852\ K + 0.4 \times 0.533 \times 2{,}852\ K} = 1.71\ ft$$

$$\Sigma M_{@N\text{-}S} = 1.5' \times \underset{40\%\ a_H}{0.4 \times 0.528} \times \underset{Wp}{864\ K} + 3' \times \underset{40\%\ EQhc}{0.4 \times 1{,}506\ K} + \underset{\Delta b}{1.71'} \times (\underset{Wc}{2{,}852K} + \underset{40\%\ EQvc}{0.4 \times 1{,}520\ K})$$

$$= 274\ ft\text{-}K + 1{,}807\ ft\text{-}K + 5{,}917\ ft\text{-}K = 7{,}998\ ft\text{-}K$$

$$\Delta b_{N-S} = \frac{9.83' \times EQhc}{Wc + EQvc} = \frac{9.83' \times 1.0 \times 1{,}506\ K}{2{,}852\ K + 0.4 \times 0.533 \times 2{,}852\ K} = 4.28\ ft$$

$$\Sigma M_{@E\text{-}W} = 1.5' \times \underset{100\%\ a_H}{0.528} \times \underset{Wp}{864\ K} + 3' \times \underset{100\%\ EQhc}{1{,}506\ K} + \underset{\Delta b}{4.28'} \times (\underset{Wc}{2{,}852K} + \underset{40\%\ EQvc}{0.4 \times 1{,}520\ K})$$

$$= 684\ ft\text{-}K + 4{,}518\ ft\text{-}K + 14{,}808\ ft\text{-}K = 20{,}010\ ft\text{-}K$$

Determine $q_{allowable}$ for FS = 1.1.

Allowable Bearing Capacity of Cask Storage Pads Inertial Forces

PSHA 2,000-Yr Earthquake: Case IVC **100 % in X, 40 % in Y, 40 % in Z**

Soil Properties:

$c =$ **2,200** Cohesion (psf)

$\phi =$ **0.0** Friction Angle (degrees)

$\gamma =$ **80** Unit weight of soil (pcf)

$\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **26.5** Footing Width - ft (E-W) $L' =$ **55.1** Length - ft (N-S)

$D_f =$ **2.7** Depth of Footing (ft)

$\beta =$ **9.9** Angle of load inclination from vertical (degrees)

$FS =$ **1.1** Factor of Safety required for $q_{allowable}$.

$F_V =$ **3,716** k $EQ_V =$ **792** k

$EQ_{H\,E\text{-}W} =$ **785** k & $EQ_{H\,N\text{-}S} =$ **1,962** k $\rightarrow$ **2,113** k for F_H

$$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi\tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2\,(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.81	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2\,D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.04	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.38 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.37	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'\,L'\,c\,N_c)$	=	0.82	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	10,882	psf =	10,612	+	271	+	0
q_{all} =	9,890	psf = q_{ult} / FS					
q_{actual} =	3,092	psf = $(F_v + EQ_v)/(B' \times L')$					
FS_{actual} =	3.52	= q_{ult}/q_{actual}			> 1.1 Hence OK		

As indicated in Table 2.6-7, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the inertial loads due to the design basis ground motion exceeds 7.7 ksf for all loading cases identified above. The minimum allowable value was obtained for Load Case II, wherein 100% of the earthquake loads act in the N-S and E-W directions and 0% acts in the vertical direction. The actual factor of safety for this condition was 2.3, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1).

BASED ON MAXIMUM CASK DYNAMIC FORCES FROM THE SSI ANALYSIS

The following pages determine the allowable bearing capacity for the cask storage pads with respect to the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 1999) for the pad supporting 2 casks, 4 casks, and 8 casks. These dynamic forces represent the maximum force occurring at any time during the earthquake at each node in the model used to represent the cask storage pads. It is expected that these maximum forces will not occur at the same time for every node. These forces, therefore, represent an upper bound of the dynamic forces that could act at the base of the pad.

The coordinate system used in these analyses is the same as that used for the analyses discussed above, which is shown in Figure 1. Note, this is different than the coordinate system used in Calculation 05996.02-G(PO17)-2 (CEC, 1999), which is shown on Page B11. Therefore, in the following pages, the X direction is N-S, the Y direction is vertical, and the Z direction is E-W.

These maximum dynamic cask driving forces were confirmed to be less than the maximum force that can be transmitted from the cask to the pad through friction acting at the base of the cask for each of these load cases. This friction force was calculated based on the upper-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. These maximum dynamic cask driving forces can be transmitted to the pad through friction only when the inertial vertical forces act downward; therefore, these analyses are performed only for Load Case IV. The analyses conservatively assume that 100% of the horizontal forces act in the E-W and vertical directions at the same time. The width (30 ft) is less in the E-W direction than the length N-S (64 ft); therefore, the E-W direction is the critical direction with respect to a bearing capacity failure.

NOTED ~~JUN 1 9 1997~~ P. J. Trudeau 1-26-00

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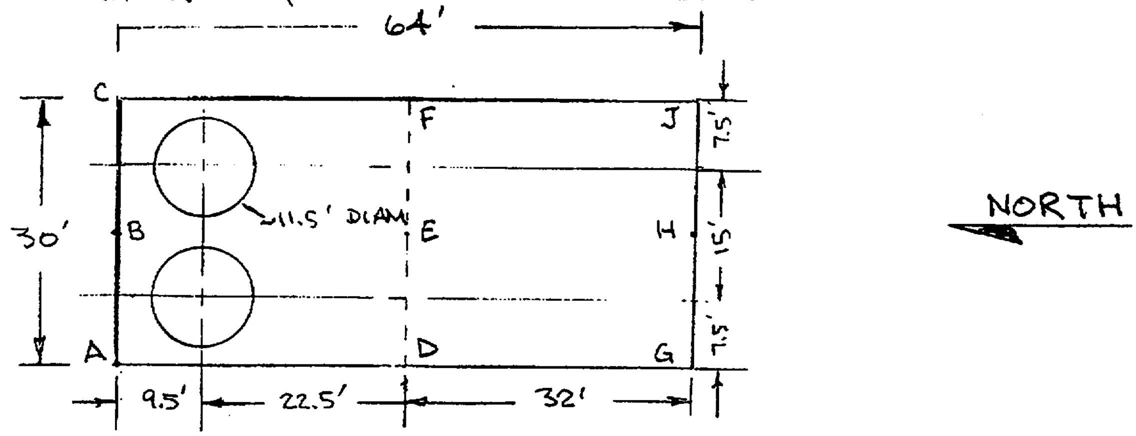
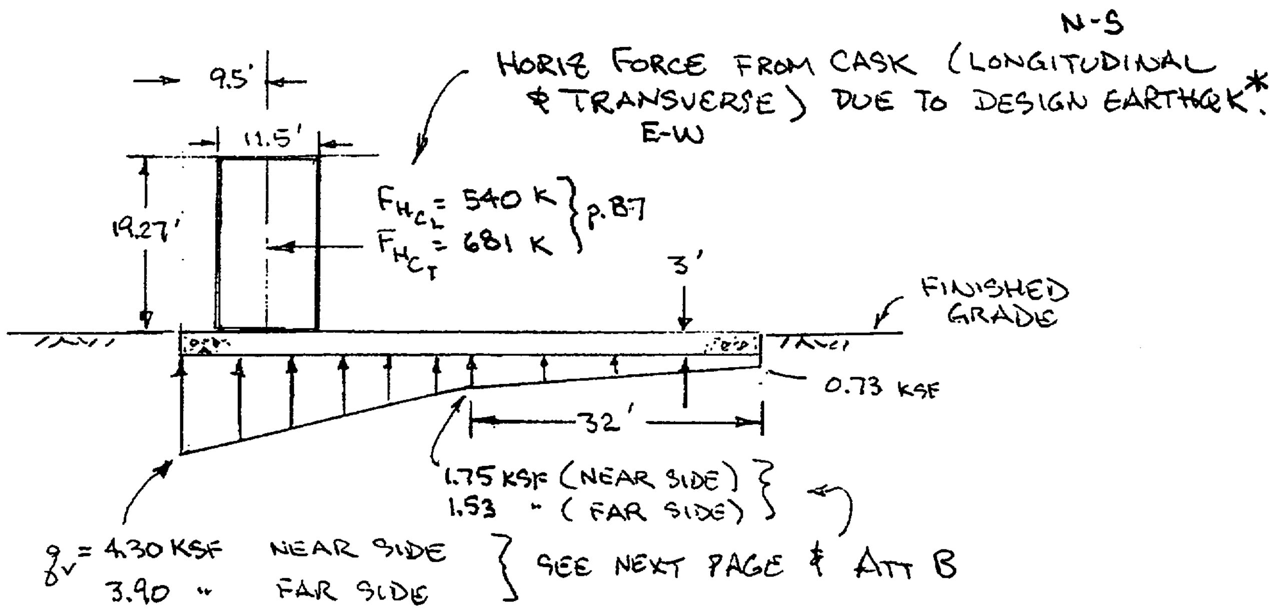
DYN BEARING CAPACITY OF PAD: 2-CASK CASE



PLAN



ELEV

* STRESSES AT PAD/SOIL INTERFACE OBTAINED FROM CEC (1999) CALC 05996.02-G(PO17)-2, REV 1 - COPIES OF PERTINENT PAGES ARE INCLUDED IN ATT B OF THIS CALC

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DYN BEARING CAPACITY OF PAD: 2-CASK CASE

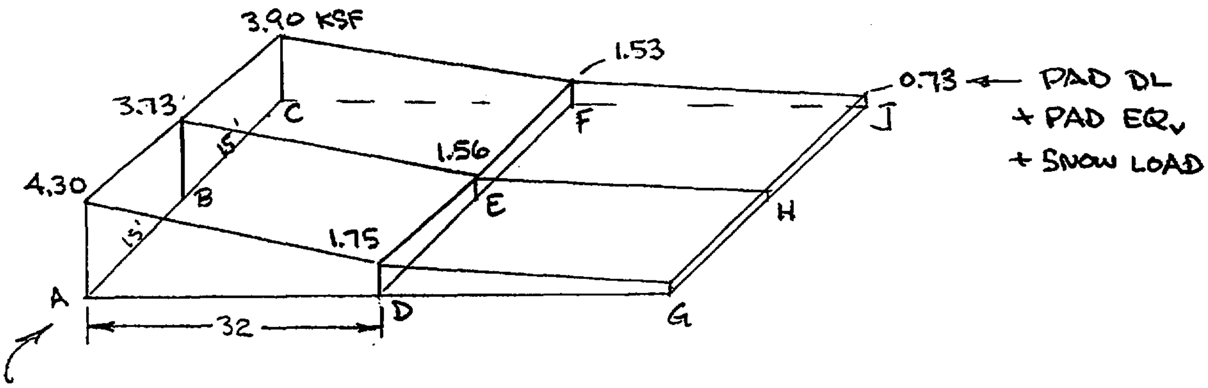
SOIL BEARING PRESSURES ARE BASED ON INFO FROM CEC(1999) INCLUDED IN ATT B AND ARE SUMMARIZED IN TABLE 1.

VERTICAL PRESSURES INCLUDE: PAD DL = 0.45 KSF
PAD EQ = 0.24 KSF
SNOW LOAD = 0.045 KSF

CASK LL = 1.36 KSF ALONG LINE AC & IS ASSUMED TO DECREASE LINEARLY TO 0 ALONG LINE DF.

CASK EQ PRESSURES ARE SHOWN ON TABLE 1.

SUMMING THESE VERTICAL PRESSURES RESULTS IN THE FOLLOWING MAXIMUM TOTAL PRESSURE DISTRIBUTION. NOTE, LOADING FROM CASKS & PAD ARE ESSENTIALLY APPLIED TO ONLY ~1/2 OF THE PAD.



FOR LOADED HALF OF PAD:

$$F_v \doteq \left[\frac{15'}{2}\times\overbrace{(4.30+2\times3.73+3.90)^{KSF}}^{[117.45} + \frac{15'}{2}\overbrace{(1.75+2\times1.56+1.53)^{KSF}}^{48.00]}\right]\frac{32'}{2}$$

$F_v \simeq 2647$ K FOR LOADED 1/2 OF PAD

$A_{AC} \sim 117.45 \frac{K}{FT} = 30' \times q_{AVG_{AC}} \Rightarrow q_{AVG_{AC}} = 3.92$ KSF

$A_{DF} \sim 48.00 \frac{K}{FT} = 30\, q_{AVG_{DF}} \Rightarrow q_{AVG_{DF}} = 1.60$ KSF

~~00867~~

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

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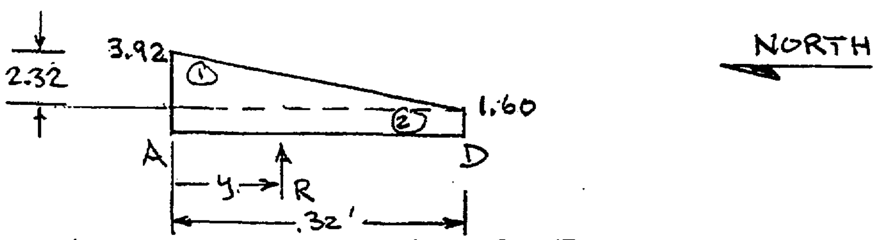
DYN BEARING CAPACITY OF PAD: 2-CASK CASE

DETERMINE ~ ECCENTRICITY OF F_V IN L DIRECTION (N-S) USING $q_{AVG\,AC}$ & $q_{AVG\,DF}$



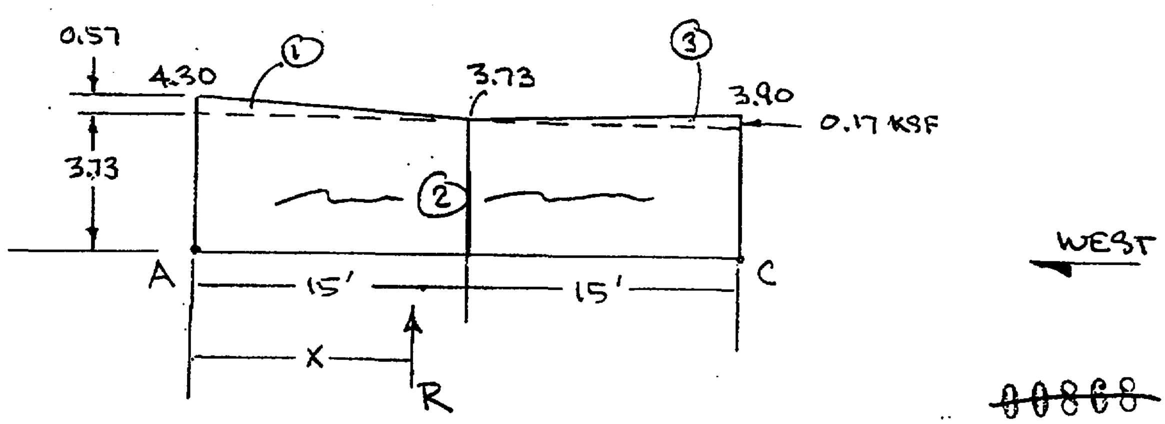
ΣM_A TO FIND LOC'N OF F_V

$$Ry = \frac{1}{2}\,2.32 \times 32 \times \frac{1}{3}32 + 1.60 \times 32 \times \frac{32}{2}$$

$$R = \frac{1}{2}\,2.32 \times 32 + 1.60 \times 32 = 37.12 + 51.20 = 88.32\,K$$

$$y = \frac{395.95 + 819.20}{88.32}\ \text{K-FT} = 13.76\ \text{FT}$$

DETERMINE ECCENTRICITY OF F_V IN B DIRECTION (E-W) USING MAX SOIL PRESSURES ALONG LINE AC



~~00868~~

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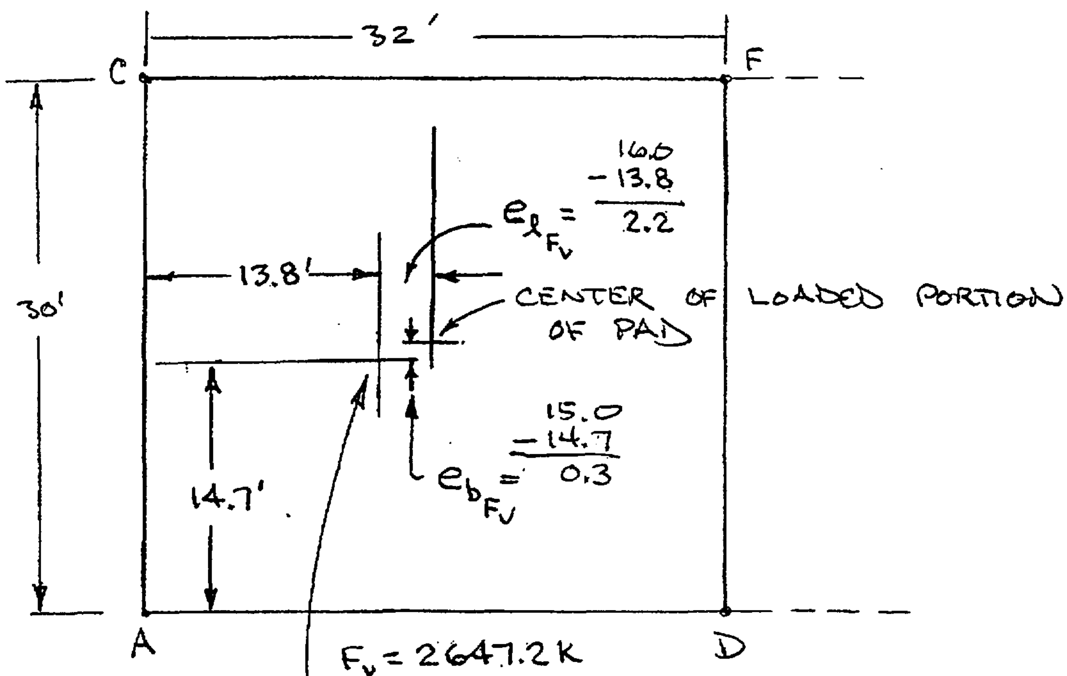
DYN BEARING CAPACITY OF PAD : 2-CASK CASE

ΣM_A

	AREA (K/FT)	MOMENT ARM (FT)	MOMENT K-FT/FT
1	$\frac{1}{2}$ 0.57 KSF × 15′ = 4.28	$\frac{1}{3} \cdot 15' = 5'$	21.40
2	3.73 KSF × 30′ = 111.90	$\frac{1}{2} \cdot 30' = 15'$	1678.50
3	$\frac{1}{2}$ 0.17 KSF × 15′ = 1.28	$15' + \frac{1}{3} 15 = 20'$	25.60

$\Sigma F_V = R = 117.45$ K/FT   1725.50

$$\therefore \quad x = \frac{\Sigma M_A}{\Sigma F_V} = \frac{1725.50 \text{ K-FT/FT}}{117.45 \text{ K/FT}} = 14.69'$$



POINT OF APPLICATION OF F_v DUE TO PAD (DL+EQ) & CASKS (LL+EQ) ~~00809~~ FOR 2-CASK CASE

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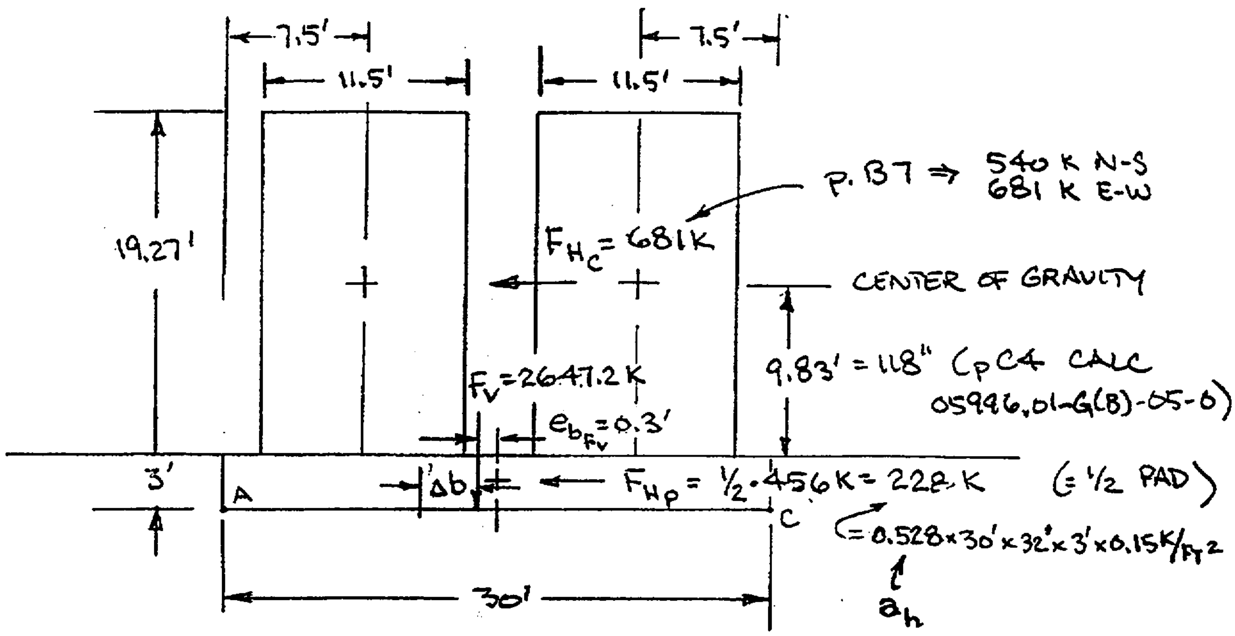
DYN BEARING CAPACITY OF PAD: 2-CASK CASE



$\Sigma M_{F_V @ \text{BOTTOM OF PAD}}$ TO FIND LOC'N OF RESULTANT TO RESIST F_H's

$$R\Delta b = 1.5' \times \underbrace{\frac{456^K}{2}}_{}{}^{\text{½ PAD}*} + (3' + 9.83')\ 681\text{ K} \quad \text{2-CASKS TRANSVERSE LOADING (E-W)}$$

$R = F_V = 2647.2\text{K}$

$$\therefore \Delta b = \frac{342.0 + 8737.2}{2647.2\text{ K}}\ \text{K-FT} = 3.43\text{ FT}$$

ADD $e_{b_{F_V}} = 0.3 \Rightarrow e_b = 3.43' + 0.3' = 3.73'$

$$B' = B - 2e_b = 30' - 2\times 3.73' = 22.54'$$

* NOTE: HORIZ INERTIA OF OTHER ½ OF PAD $\left(\frac{228\text{K}}{30'\times 32'} = 0.24\text{ KSF}\right)$ IS RESISTED BY C = 1.4 KSF & N tan φ w/ φ = 21.3° ALONG BASE OF THAT ½ OF PAD

~~00870~~

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DYN BEARING CAPACITY OF PAD: 2-CASK CASE

CALCULATE L' SIMILARLY FOR LONGITUDINAL DIRECTION

$F_{H_{C_L}} = 540\ K$ ($= Q_{yd_{max}}$ FROM p B7 FOR 2 CASKS)

ΣM_{F_v} — 1/2 PAD — 2-CASKS LONGITUDINAL LOAD'G

$R\,\Delta l = 1.5' \times \frac{456\,k}{2} + (3' + 9.83')(540\ K)$

$R = F_v = 2647.2\,K$

$\therefore \Delta l = \dfrac{342.00\ \text{K-FT} + 6928\ \text{K-FT}}{2647.2\ K} = 2.75\ \text{FT}$

ADD $e_{l_{F_v}} = 2.2' \Rightarrow e_l = 2.75' + 2.2' = 4.95'$

$L' = L - 2e_l = 32' - 2 \times 4.95' = 22.11'$ ← < 22.54' ∴ THIS = B' & L' = 22.54'

$q_{ACTUAL} = \dfrac{F_v}{B' \times L'} = \dfrac{2647.2\ K}{22.11' \times 22.54'} = 5.31\ \text{KSF}$

CALC q_{ALLOW} FOR THE FOLLOWING: $B' = 22.1'$ $L' = 22.5'$

$F_H = 681\,K + 228\,K = 909\,K$ MAX vs 540K N-S + 228K PAD = 768K N-S

2-CASK EQ_h (p B7); 1/2 PAD EQ_h

FS = 1.1

$F_v = 2647.2\,K$ FOR 2-CASK (STATIC + DYN)

ASSUME $\gamma_{SURCH} = 100$ PCF FOR SOIL CEMENT &

$D_f = 3' - \frac{3.5''}{12''/'} = 2.7'$ (TOP OF PAD 3.5" ABOVE GRADE)

~~00871~~

FOR DYN LOADS, $\phi = 0^\circ$ $c = 2.2$ KSF

6

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON MAXIMUM CASK DYNAMIC FORCES FROM THE SSI ANALYSIS

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 2 CASKS

PSHA 2,000-Yr Earthquake: Case IV | **100 % in X, 100 % in Y, 100 % in Z**

Soil Properties:
- $c =$ **2,200** Cohesion (psf)
- $\phi =$ **0.0** Friction Angle (degrees)
- $\gamma =$ **80** Unit weight of soil (pcf)
- $\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

Foundation Properties:
- $B' =$ **22.1** Footing Width - ft (E-W) $L' =$ **22.5** Length - ft (N-S)
- $D_f =$ **2.7** Depth of Footing (ft)
- $\beta =$ **19.0** Angle of load inclination from vertical (degrees)
- $FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **2,647** k (Includes EQ_V)
- $EQ_{H\ E\text{-}W} =$ **909** k & $EQ_{H\ N\text{-}S} =$ 768 k → **1,190** k for F_H

$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$ — **General Bearing Capacity Equation, based on Winterkorn & Fang (1975)**

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.19	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.61	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.87 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.53	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.68	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	9,824	psf =	9,554	+	271	+	0
q_{all} =	8,930	psf = q_{ult} / FS					
q_{actual} =	5,323	psf = $(F_v + EQ_v)/(B' \times L')$					
FS_{actual} =	1.85	= q_{ult}/q_{actual}			> 1.1 Hence OK		

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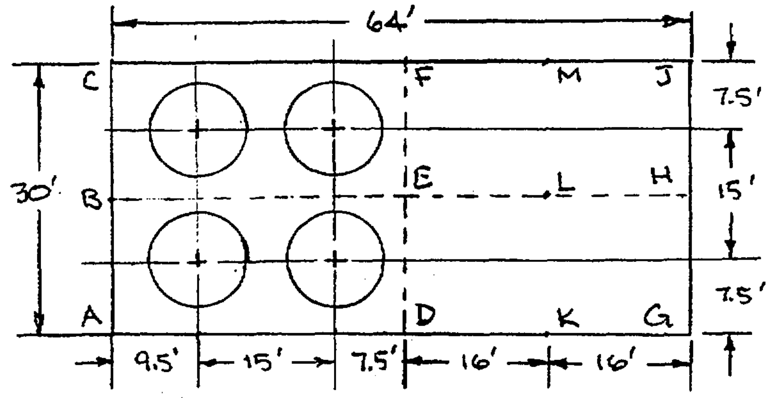
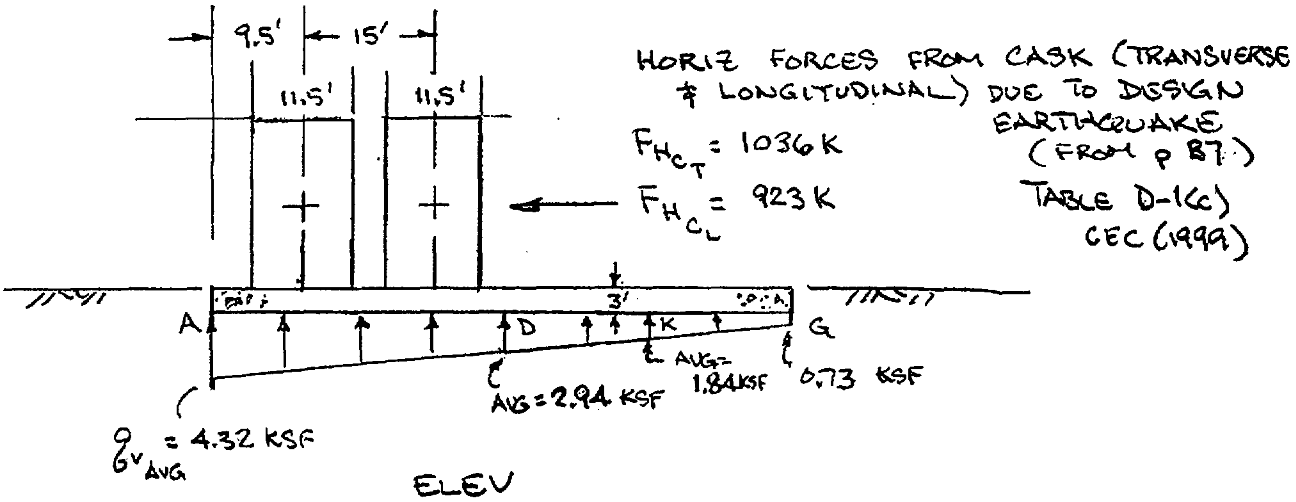
DYN BEARING CAPACITY OF PAD: 4-CASK CASE



PLAN



ELEV

STRESSES AT PAD/SOIL INTERFACE FROM CEC (1999) SEE ATTACHMENT B AND NEXT 2 PAGES.

~~00874~~

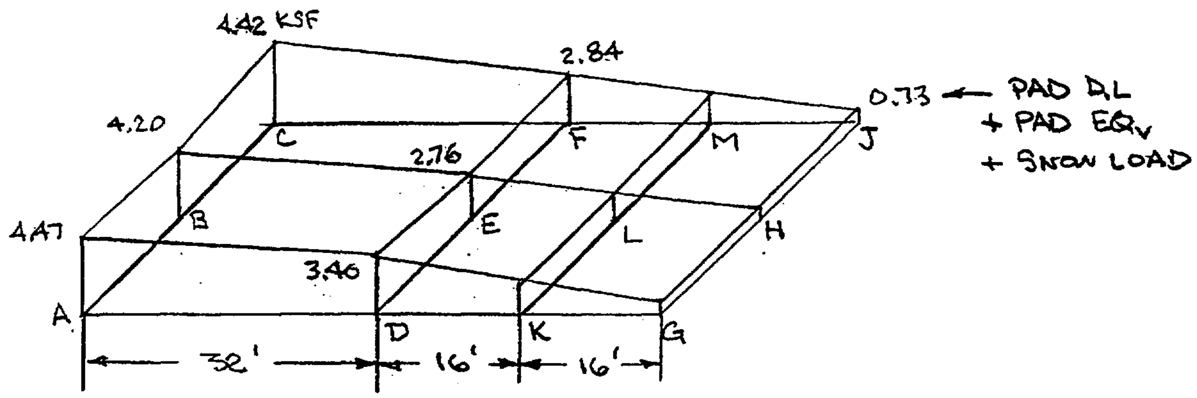
DYN BEARING CAPACITY OF PAD: 4-CASK CASE

SOIL BEARING PRESSURES ARE BASED ON INFO FROM CEC (1999) INCLUDED IN ATT B AND ARE SUMMARIZED IN TABLE 1.

VERTICAL PRESSURES INCLUDE: PAD DL = 0.45 KSF
PAD EQ = 0.24 KSF
SNOW LOAD = 0.045 KSF

LL OF CASKS = 1.77 KSF ALONG LINE AC & IS ASSUMED TO DECREASE LINEARLY TO 0 ALONG LINE GJ.

CASK EQ PRESSURES ARE SHOWN ON TABLE 1
RESULTING PRESSURE DISTRIBUTION:



ASSUME 3/4 OF PAD IS EFFECTIVE IN RESISTING LOADS OF 4-CASK CASE

$\therefore$ B = 30' $L = \frac{3}{4}\,64 = 48'$

LINEARLY DISTRIBUTE STATIC + DYN LOADING FROM LINE DF TO 48' AWAY FROM LINE AC & DETERMINE F_V

VERT STRESSES	KSF	POINT
0.5 (3.40 + 0.73) =	2.07	K
0.5 (2.76 + 0.73) =	1.75	L
0.5 (2.84 + 0.73) =	1.79	M

~~00875~~

B CALCULATION IDENTIFICATION NUMBER				PAGE 56
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DYN BEARING CAPACITY OF PAD : 4-CASK CASE

CALCULATE F_v

ALONG LINE	AREA = K/FT		
AC	$\frac{15'}{2}(4.47 + 2\times4.20 + 4.42)$ KSF	=	129.68 K/FT
DF	$\frac{15'}{2}(3.40 + 2\times2.76 + 2.84)$	=	88.20
KM	$\frac{15'}{2}(2.07 + 2\times1.75 + 1.79)$	=	55.20

$$F_v \sim \frac{32'}{2}(129.68 + 88.20)^{K}/_{1} + \frac{16'}{2}(88.20 + 55.20)^{K}/_{1}$$

$$F_v = 3486.1\text{ K} + 1147.2\text{ K} = \underline{4633.3\text{ K}}$$

ESTIMATE LOCATION WHERE F_v ACTS ON $30' \times 48'$ PORTION OF PAD.

NOTE AVG VERT STRESS ALONG LINES.

LINE

$$AC = \frac{129.68\ K/FT}{30\ FT} = 4.32\ KSF$$

$$DF = \frac{88.20\ K/FT}{30'} = 2.94\ KSF$$

$$KM = \frac{55.20\ K/FT}{30'} = 1.84\ KSF$$

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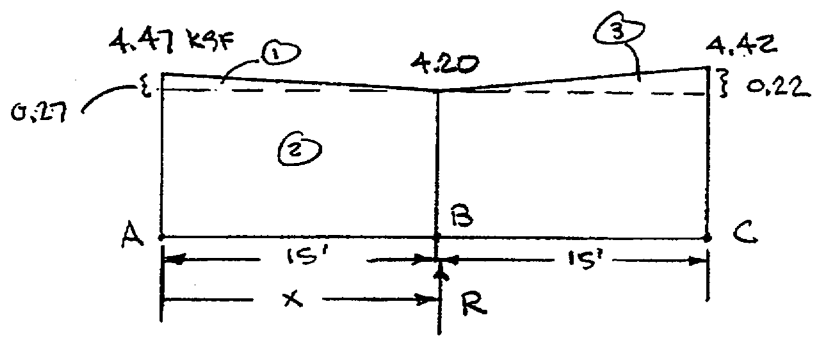
DYN BEARING CAPACITY OF PAD: 4-CASK CASE

DETERMINE ECCENTRICITY OF F_V WRT B, $e_{B_{F_V}}$

ALONG LINE AC



ΣM_A

AREA	K/FT	MOMENT ARM (FT)	MOMENT
①	$\frac{1}{2} \times 0.27 \frac{K}{FT^2} \times 15' = 2.03$	$\frac{1}{3} \times 15' = 5'$	10.13
②	$4.20 \frac{K}{FT^2} \times 30' = 126.00$	$\frac{1}{2} \times 30' = 15'$	1890.0
③	$\frac{1}{2}\, 0.22 \frac{K}{FT^2} \times 15' = 1.65$	$15' + \frac{2}{3}\, 15' = 25'$	41.3

$F_V = \Sigma = 129.68$ K/FT $\qquad Rx = \Sigma = 1941.4$

$$\therefore \quad x = \frac{1941.4 \ \text{K-FT/FT}}{129.68 \ \text{K/FT}} = 15.0'$$

$$e_{B_{F_V}} = \frac{B}{2} - x = 15' - 15.0' = 0'$$

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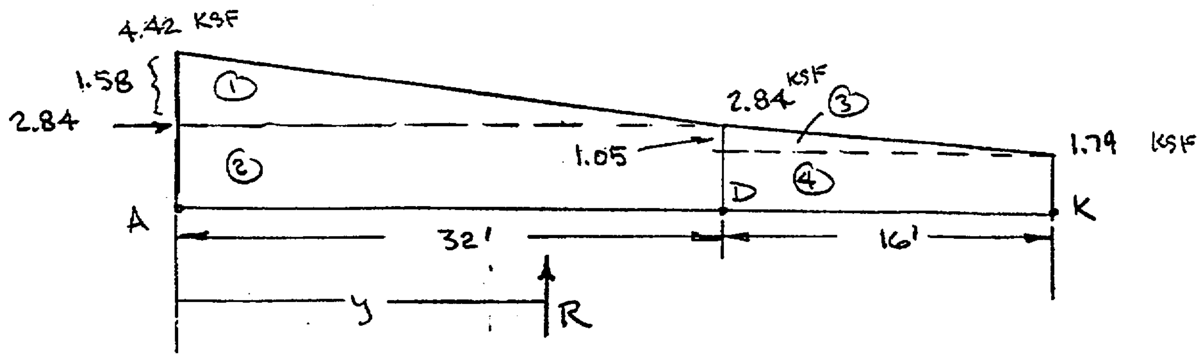
DYN BEARING CAPACITY OF PAD: 4-CASK CASE

DETERMINE ECCENTRICITY OF F_v WRT L, $e_{L_{F_v}}$



ΣM_A TO FIND y

	FORCE (K/')	MOMENT ARM (FT)	MOMENT K·FT/L.F.
①	$\frac{1}{2}$ 1.58 × 32' = 25.28	$\frac{1}{3}$ × 32' = 10.67'	269.7
②	2.84 × 32 = 90.88	$\frac{1}{2}$ × 32' = 16'	1454.0
③	$\frac{1}{2}$ 1.05 × 16 = 8.40	32' + $\frac{1}{3}$16' = 37.33'	313.6
④	1.79 × 16 = 28.64	32' + $\frac{16'}{2}$ = 40'	1145.6

$R = \Sigma F_v = 153.20$ K/' $\qquad Ry = \Sigma M = 3182.9$

$$y = \frac{\Sigma M}{\Sigma F_v} = \frac{3182.9 \text{ K-FT/LF}}{153.20 \text{ K/LF}} = 20.78 \text{ FT}$$

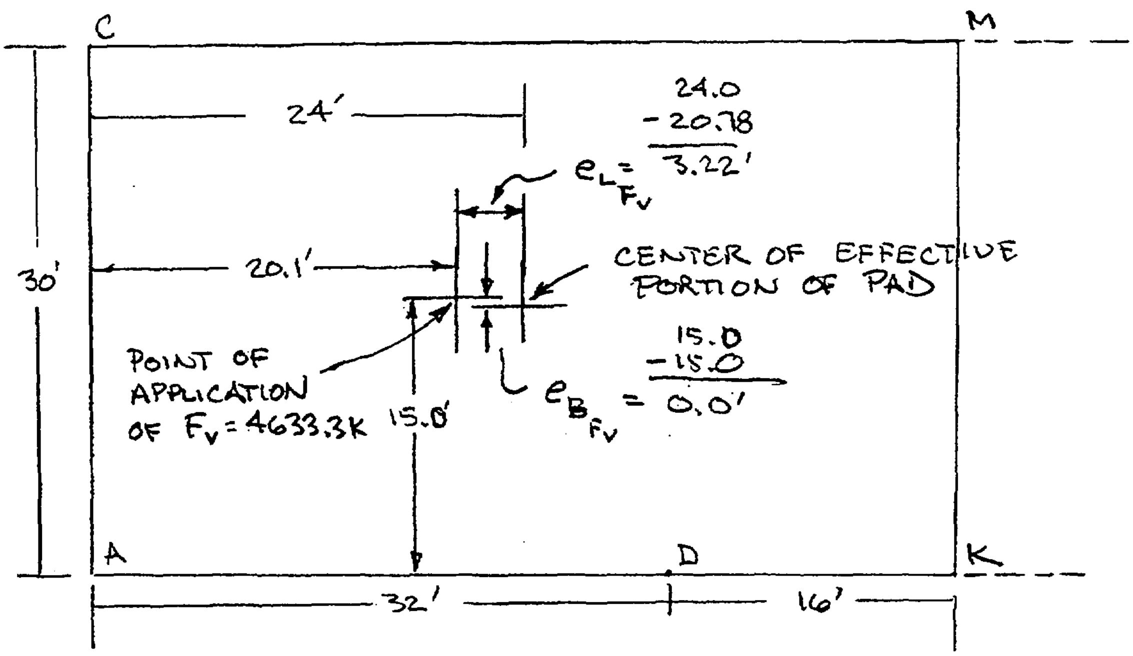
$$e_{L_{F_v}} = \frac{L}{2} - y = 24.0' - 20.78' = 3.22'$$

AS SHOWN ON NEXT PAGE

~~00878~~

DYN BEARING CAPACITY OF PAD: 4-CASK CASE

PLAN VIEW OF PAD SHOWING LOCATION OF VERTICAL FORCE DUE TO VERTICAL STRESSES FOR 4-CASK LOADING CASE



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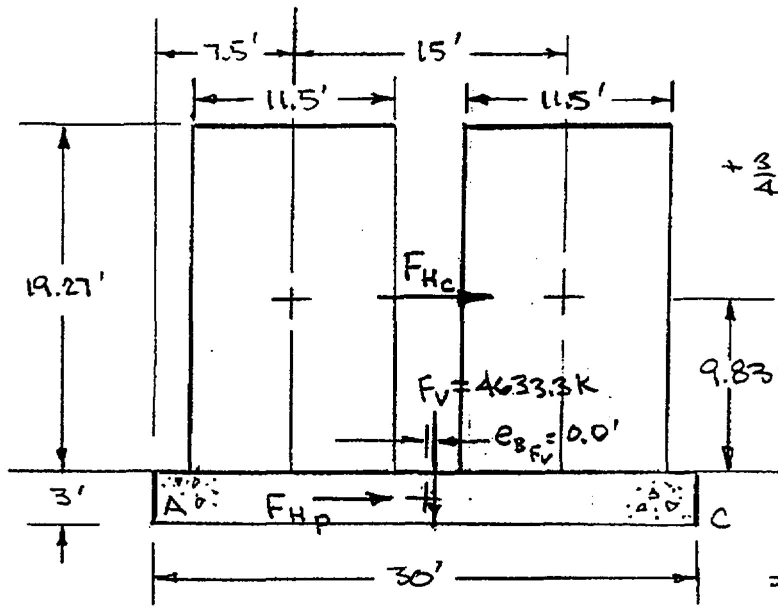
DYN BEARING CAPACITY OF PAD: 4-CASK CASE



4-CASK Q_{xd} IN TABLE D-1(c)

$F_{H_{C_T}} = 1036K$ (p B7)

$+\frac{3}{4}F_{H_P} = \frac{3}{4}456 \Rightarrow 1378K = EQ_{H\ E\text{-}W}$

CENTER OF GRAVITY OF CASK

9.83' = 118" (p C4 CALC 05996.01-G(B)-05-0)

$F_{H_P} = 456^K$ FOR 30'×64'

$= 0.528g \times 30' \times 64' \times 3' \times 0.15^K/FT^3 / g$

ΣM_{F_V} TO FIND LOC'N OF RESULTANT TO RESIST F_H's

4-CASK TRANSVERSE F_H

$$R\,\Delta b = 1.5' \times \underbrace{\tfrac{3}{4} \times 456^K}_{3/4\ PAD} + (3' + 9.83')\ 1036K$$

$R = F_V = 4633.3^K$ ON 30'×48' PORTION OF PAD

$$\therefore\ \Delta b = \frac{513.2 + 13{,}292\ \text{K-FT}}{4633.3\ K} = 2.98' = e_{B_{F_H}}$$

$$e_b = e_{B_{F_V}} + e_{B_{F_H}} = 0.0' + 2.98' = 2.98'$$

$$B' = B - 2e_B = 30' - 2 \times 2.98' = \underline{24.04'}$$

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DYN BEARING CAPACITY OF PAD: 4-CASK CASE

CALCULATE L' SIMILARLY FOR LONGITUDINAL F_H

$F_{H_{C_L}} = 923\ K = Q_{yd_{MAX\ 4\ CASKS}}$ TABLE D-1(c) p B7

$+\frac{3}{4}F_{Hp} = \frac{3}{4}456 \Rightarrow EQ_{H\ N\text{-}S} = 1265\ K$

ΣM_{F_V} $R\Delta l = 1.5' \times \frac{3}{4} \times 456\ K + (3' + 9.83')\ \underbrace{923\ K}_{F_{H_{CL}}\text{ 4 CASKS}}$

$R = F_V = 4633.3\ K$ ON EFFECTIVE PORTION OF PAD (30' x 48')

$$\Delta l = \frac{513.2\ \text{K-FT} + 11{,}842.1\ \text{K-FT}}{4633.3\ K} = 2.67' = e_{L_{F_H}}$$

$$e_L = e_{L_{F_V}} + e_{L_{F_H}} = 3.22' + 2.67' = 5.89'$$

$$L' = L - 2e_L = 48' - 2 \times 5.89' = 36.23'$$

$$q_{ACTUAL} = \frac{F_V}{B' \times L'} = \frac{4633.3k}{24.04' \times 36.23'} = \underline{\underline{5.32\ KSF}}$$

CALC q_{ALLOW} FOR THE FOLLOWING: $B' = 24.04'$ $L' = 36.23'$

$F_V = 4633.3\ K$ FOR 4-CASK CASE (STATIC + DYN)

$EQ_{H\ E\text{-}W} = \overset{PAD}{\frac{3}{4}456} + \overset{F_{HC}}{1036} = 1378\ K$ E-W

$EQ_{H\ N\text{-}S} =$ " $+ 923 = 1265$ N-S

$FS = 1.1$ $\gamma_{SURCH} = 100\ PCF$ $\gamma = 80\ PCF$ $D_f = 3' - \frac{3.5''}{12''/'} = 2.71'$

$\phi = 0°$ $c = 2.2\ KSF$

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DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON MAXIMUM CASK DYNAMIC FORCES FROM THE SSI ANALYSIS

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 4 CASKS

PSHA 2,000-Yr Earthquake: Case IV — **100 % in X, 100 % in Y, 100 % in Z**

Soil Properties:
- $c =$ **2,200** Cohesion (psf)
- $\phi =$ **0.0** Friction Angle (degrees)
- $\gamma =$ **80** Unit weight of soil (pcf)
- $\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

Foundation Properties:
- $B' =$ **24.0** Footing Width - ft (E-W) $L' =$ **36.2** Length - ft (N-S)
- $D_f =$ **2.7** Depth of Footing (ft)
- $\beta =$ **16.6** Angle of load inclination from vertical (degrees)
- $FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **4,633** k (Includes EQ_V)
- $EQ_{H\,E\text{-}W} =$ **1,378** k & $EQ_{H\,N\text{-}S} =$ **1,265** k → **1,871** k for F_H

$$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.13	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.73	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.83 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.52	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.71	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	9,773	psf =	9,503	+	271	+	0
q_{all} =	8,880	psf = q_{ult} / FS					
q_{actual} =	5,320	psf = $(F_v + EQ_v)/(B' \times L')$					
FS_{actual} =	1.84	= q_{ult}/q_{actual}			> 1.1 Hence OK		

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CALCULATION SHEET

▲ 5010.65

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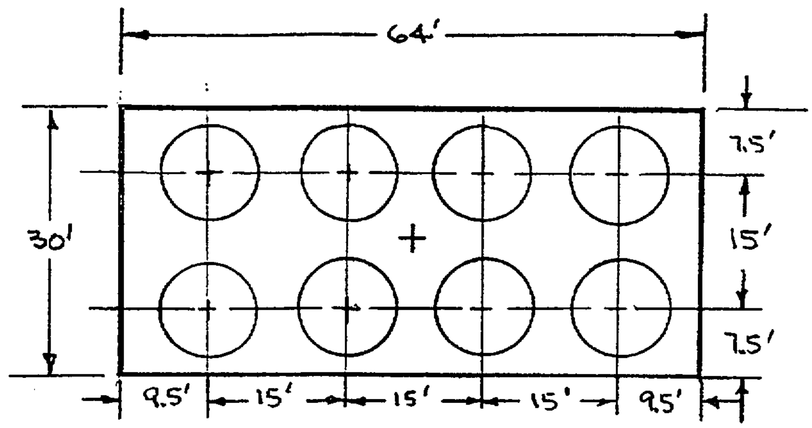
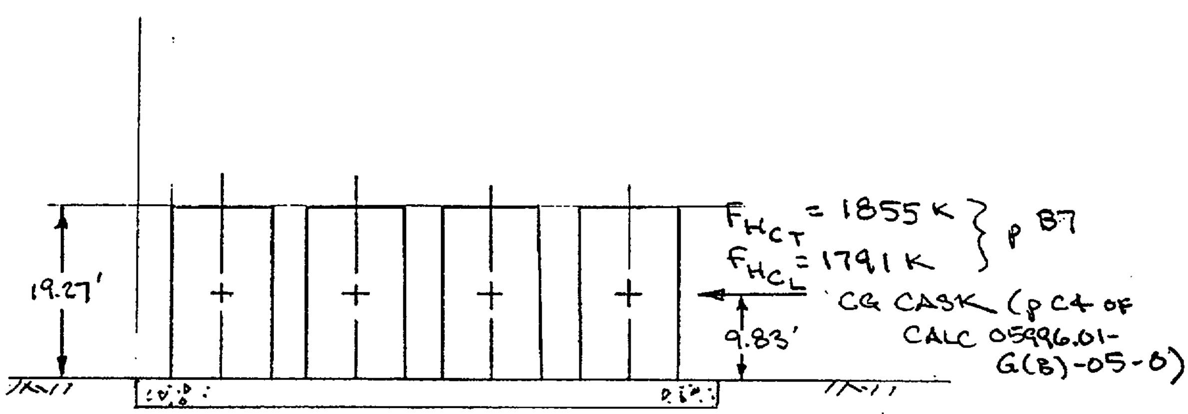
05996.02	G(B)	04-6		

DYN BEARING CAPACITY OF PAD: 8-CASK CASE



PLAN



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CALCULATION SHEET

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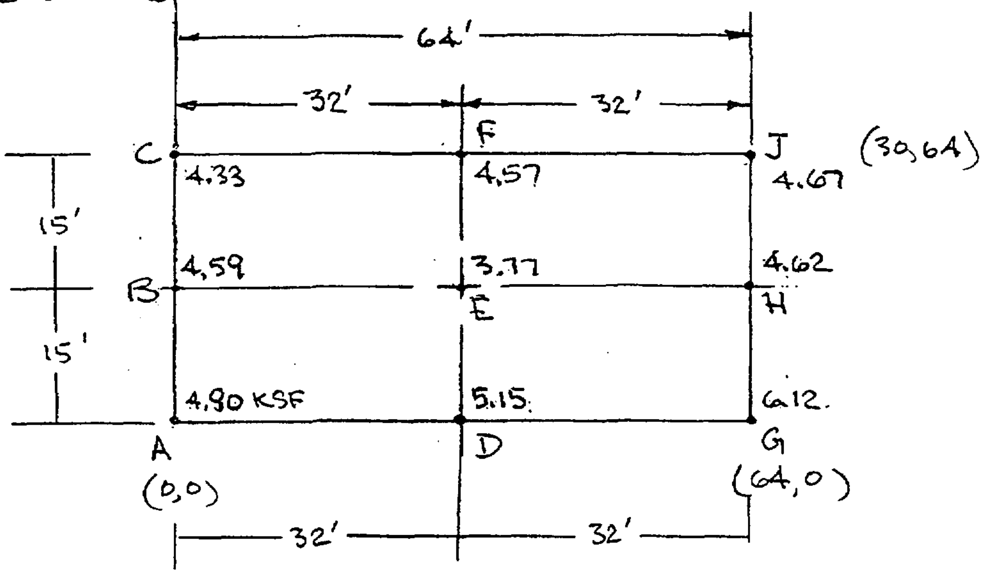
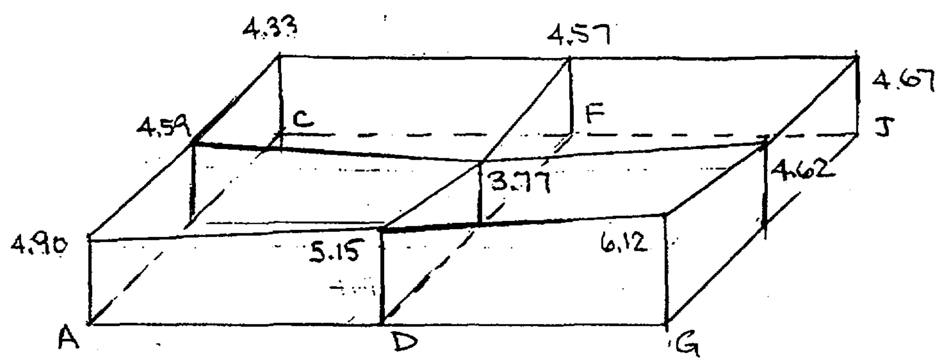
DYN BEARING CAPACITY OF PAD: 8-CASK CASE

SOIL BEARING PRESSURES FROM TABLE 1





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DYN BEARING CAPACITY OF PAD: 8-CASK CASE

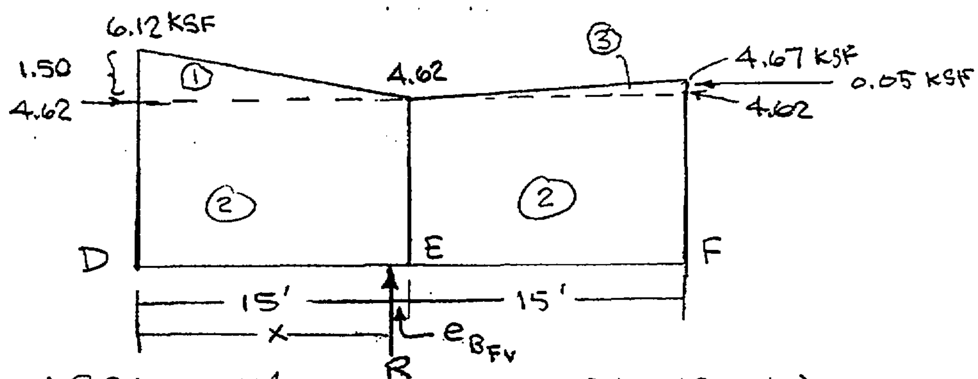
CALCULATE F_v :

ALONG LINE	AREA = K/FT	F_v (K/FT)	q_{AVG} (KSF)
AC	$\frac{15}{2}(4.90 + 2\times 4.59 + 4.33)$ =	138.08	4.60
DF	$\frac{15}{2}(5.15 + 2\times 3.77 + 4.57)$ =	129.45	4.32
GJ	$\frac{15}{2}(6.12 + 2\times 4.62 + 4.67)$ =	150.23	5.01

$F_v \sim \frac{32'}{2}(138.08 + 2\times 129.45 + 150.23) = \underline{8755\ K}$

ESTIMATE LOCATION WHERE F_v ACTS.
DETERMINE ECCENTRICITY OF F_v WRT B, $e_{B_{F_v}} = \frac{B}{2} - x$
ALONG LINE GJ, WHICH HAS THE GREATEST STRESSES

ΣM_D



	AREA	K/FT	MOMENT ARM (FT)	MOMENT $\frac{K\text{-}FT}{FT}$
①	$\frac{1}{2}\, 1.50\ KSF \times 15'$	= 11.25	$\frac{1}{3}\times 15' = 5'$	56.25
②	$4.62 \times 30'$	= 138.60	$\frac{1}{2}\times 30' = 15'$	2079.00
③	$\frac{1}{2}\, 0.05 \times 15'$	= 0.38	$15' + \frac{2}{3}\times 15' = 25'$	9.38
		$R = \Sigma = 150.23$		$Rx = \Sigma = 2144.63$

~~0.0886~~

$\therefore x = \frac{Rx}{R} = \frac{2144.63\ K\text{-}FT/FT}{150.23\ K/FT} = 14.28'$

$e_{B_{F_v}} = \frac{30}{2} - 14.28' = 0.72'$

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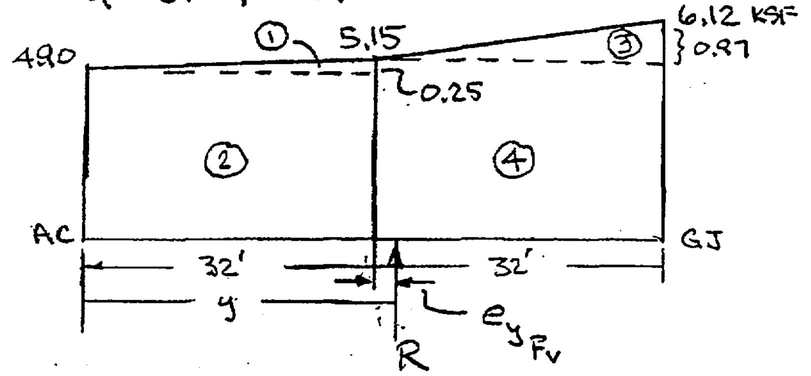
DYNAMIC BEARING CAPACITY OF PAD: 8-CASK CASE

DETERMINE ECCENTRICITY OF F_V WRT L, $e_{L_{F_V}} = \frac{L}{2} - y$
USE AVERAGE VALUES ALONG LINE AG, WHICH ARE GREATER THAN ALONG BH & CJ.



ΣM_{AC}

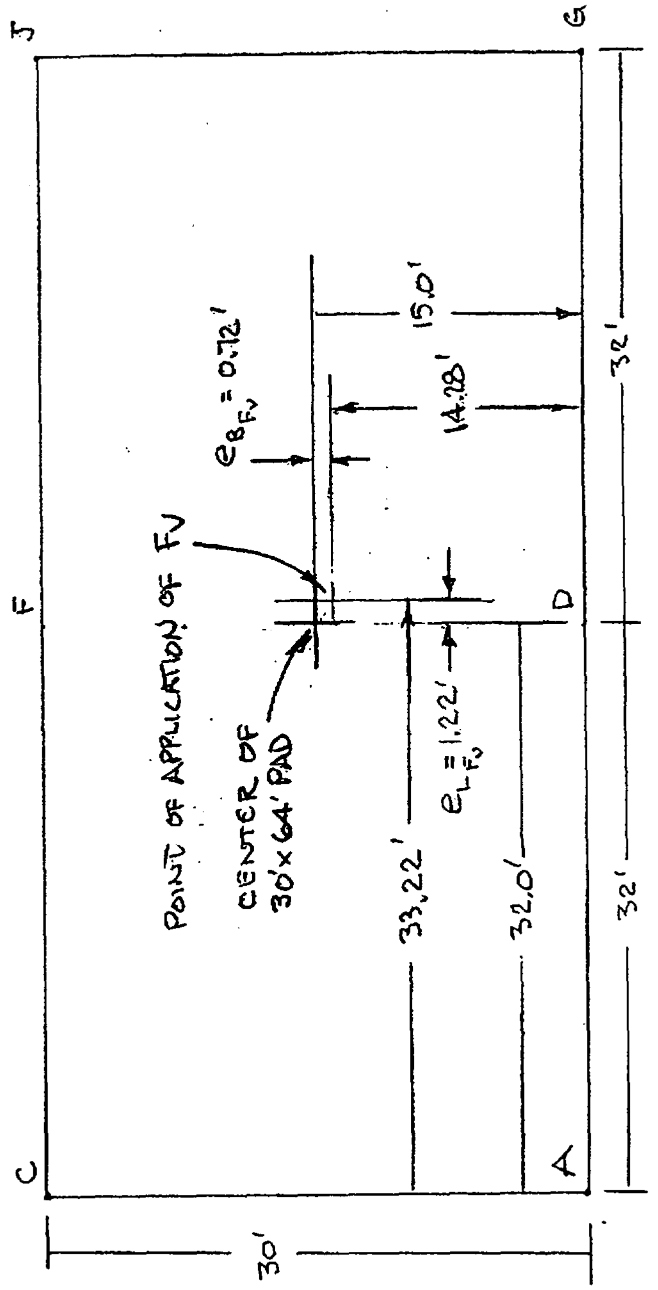
AREA	K/FT	MOMENT ARM FT	MOMENT $\frac{K\text{-}FT}{FT}$
① $\frac{1}{2}(0.25\text{ KSF}) \times 32' =$	4.0	$\frac{2}{3} \times 32' = 21.33'$	85.33
② $4.90\text{ KSF} \times 32' =$	156.80	$\frac{1}{2} \times 32' = 16'$	2508.80
③ $\frac{1}{2}(6.12-5.15)^{KSF} \times 32' =$	15.52	$32' + \frac{2}{3}32' = 53.33'$	827.68
④ $5.15\text{ KSF} \times 32' =$	164.80	$32' + \frac{1}{2}32' = 48.0'$	7910.40
$R =$	341.12 K/FT	$Ry =$	$11,332.2\ \frac{K\text{-}FT}{FT}$

$$\therefore y = \frac{Ry}{R} = \frac{11,332.2\text{ K-FT/FT}}{341.12\text{ K/FT}} = 33.22'$$

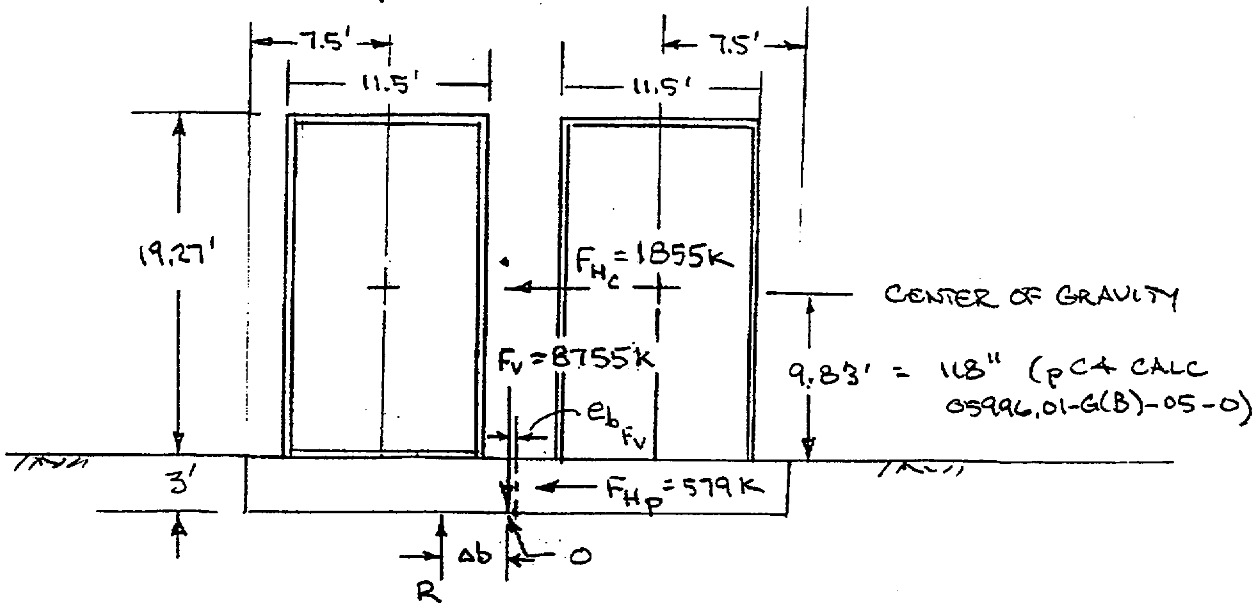
$$e_{L_{F_V}} = y - \frac{L}{2} = 33.22' - \frac{64'}{2} = 1.22'$$

~~00887~~

PLAN VIEW OF PAD SHOWING ~ LOCATION OF VERTICAL FORCES DUE TO VERTICAL STRESSES FROM STATIC AND DYNAMIC LOADS FOR 8-CASK LOADING CASE



DYN BEARING CAPACITY OF PAD: 8-CASK CASE



F_{H_c} = 1855 K FOR 8-CASK CASE (p B7)
TRANVERSE DIRECTION E-W

$F_{H_p} = 0.528\,g \times 30' \times 64' \times 3' \times \frac{0.15\ KCF}{g} = 456\ K$ (0.528 = a_h)

$\Sigma = EQ_{H\,E\text{-}W} = 2311\ K$

ΣM_o TO FIND LOCATION OF R TO RESIST MOMENT DUE TO F_H's

$R\,\Delta b = 1.5' \times 456\,K + (3'+9.83')\,1855\,K$ (R = F_v; PAD; CASKS)

$$\Delta b = \frac{684 + 23{,}800\ \text{K-FT}}{8755\ \text{K}} = 2.80\ \text{FT}$$

ADD $e_{b_{F_v}} = 0.72$ FT $\Rightarrow$ $e_b = 2.80 + 0.72 = 3.52$ FT

$$B' = B - 2e_b = 30' - 2 \times 3.52' = 22.97\ \text{FT}$$

~~00889~~

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DYN BEARING CAPACITY OF PAD: 8-CASK CASE

SIMILARLY FOR LONGITUDINAL DIRECTION

$F_{H_C} = 1791\text{ K}$ ADD $F_{HP} = 456\text{K} \Rightarrow EQ_{H_{N-S}} = 2247\text{K}$

↑ p 87

ΣM_o $\quad \overset{R=F_V}{8755\text{ K}}\ \Delta l = \overset{\text{PAD}}{1.5' \times 456\text{K}} + \overset{\text{CASKS}}{(3' + 9.83')(1791\text{K})}$

$$\Delta l = \frac{684 + 22{,}979 \text{ K-FT}}{8755 \text{ K}} = 2.70'$$

ADD $e_{l_{F_v}} = 1.22$ FT $\Rightarrow e_l = 2.70' + 1.22' = 3.92'$

$L' = L - 2e_l = 64' - 2 \times 3.92' = 56.15$ FT

$$q_{ACTUAL} = \frac{F_V}{B' \times L'} = \frac{8755\text{K}}{22.97' \times 56.15'} = 6.79 \text{ KSF}$$

CALC q_{ALLOW} FOR FS = 1.1 $\quad B' = 22.97' \quad L' = 56.15'$

$F_v = 8755$ K (STATIC + DYN 8 CASKS)

$EQ_{H_{E-W}} = \overset{F_{HP}}{456\text{ K}} + \overset{F_{HC}}{1855\text{ K}} = 2311\text{ K}$

$EQ_{H_{N-S}} = 456\text{K} + 1791\text{ K} = 2247\text{ K}$

$\gamma_{SURCH} = 100$ PCF $\quad \gamma = 80$ PCF $\quad D_f = 3' - \frac{3.5''}{12''/'} = 2.7'$

$\phi = 0°$ $\quad C = 2.2$ KSF

$\boxed{6}$

~~00880~~

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS BASED ON MAXIMUM CASK DYNAMIC FORCES FROM THE SSI ANALYSIS

ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS WITH 8 CASKS

PSHA 2,000-Yr Earthquake: Case IV $\boxed{\textbf{100 \% in X, 100 \% in Y, 100 \% in Z}}$

Soil Properties:
- $c =$ **2,200** Cohesion (psf)
- $\phi =$ **0.0** Friction Angle (degrees)
- $\gamma =$ **80** Unit weight of soil (pcf)
- $\gamma_{surch} =$ **100** Unit weight of surcharge (pcf)

Foundation Properties:
- $B' =$ **23.0** Footing Width - ft (E-W) $L' =$ **56.2** Length - ft (N-S)
- $D_f =$ **2.7** Depth of Footing (ft)
- $\beta =$ **14.8** Angle of load inclination from vertical (degrees)
- $FS =$ **1.1** Factor of Safety required for $q_{allowable}$.
- $F_V =$ **8,755** k (Includes EQ_V)
- $EQ_{H\,E\text{-}W} =$ **2,311** k & $EQ_{H\,N\text{-}S} =$ **2,247** k → **3,223** k for F_H

$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$ **General Bearing Capacity Equation, based on Winterkorn & Fang (1975)**

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.08	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.84	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.05	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.68	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.32	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.80 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.51	Eq 3.18c
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'L'c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a
For $\phi = 0$: $i_c = 1 - (m\,F_H/B'L'c\,N_c)$	=	0.67	Eq 3.16a

			N_c term		N_q term		N_γ term
Gross $q_{ult} =$	8,802	psf =	8,531	+	271	+	0
$q_{all} =$	8,000	psf = q_{ult}/FS					
$q_{actual} =$	6,788	psf = $(F_v + EQ_v)/(B' \times L')$					
$FS_{actual} =$	1.30	= q_{ult}/q_{actual}		> 1.1	Hence OK		

[geot]j05996\calc\brng_cap\Pad\cu_phi.xls

Table 2.6-8 presents a summary of the bearing capacity analyses that were performed using the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 1999) for the pad supporting 2 casks, 4 casks, and 8 casks. Details of these analyses are presented on the preceding pages. As indicated in this table, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the very conservative maximum dynamic cask driving forces due to the design basis ground motion is at least 8.0 ksf for the 2-cask, 4-cask, and 8-cask loading cases. The minimum allowable value (8.0 ksf) was obtained for the 8-cask loading. The actual factor of safety for this case was 1.3, which is greater than the criterion for dynamic bearing capacity (FS $\geq$ 1.1).

CONCLUSIONS

Analyses presented herein demonstrate that the cask storage pads have adequate factors of safety against overturning, sliding, and bearing capacity failure for static and dynamic loadings due to the design basis ground motion. The following load cases are considered:

Case I Static

Case II Static + dynamic horizontal forces due to the earthquake

Case III Static + dynamic horizontal + vertical uplift forces due to the earthquake

Case IV Static + dynamic horizontal + vertical compression forces due to the earthquake

For Case II, 100% of the dynamic lateral forces in both X and Z directions are combined. For Cases III and IV, the effects of the three components of the design basis ground motion are combined in accordance with procedures described in ASCE (1986); i.e., 100% of the dynamic loading in one direction is assumed to act at the same time that 40% of the loading acts in the other two directions.

These results of these stability analyses are discussed in more detail in the following sections.

OVERTURNING STABILITY OF THE CASK STORAGE PADS

Analyses presented above indicate that the factor of safety against overturning due to dynamic loadings from the design basis ground motion is 1.66. This is greater than the criterion of 1.1 for the factor of safety against overturning due to dynamic loadings; therefore, the cask storage pads have an adequate factor of safety against overturning due to loadings from the design basis ground motion.

SLIDING STABILITY OF THE CASK STORAGE PADS

The cask storage pads will be constructed on and within soil cement, as described in Sections 2.6.1.7 and 2.6.4.11 of the SAR and as illustrated in Figure 4.2-7 of the SAR. Analyses presented above demonstrate that, using only the passive resistance of the soil cement above the bottom of the pads, the soil cement can be designed to provide sufficient resistance to sliding of the pads to readily achieve the minimum required factor of safety of 1.1. Thus, embedding the pads in soil cement will greatly enhance their resistance to sliding due to dynamic loads from the design basis ground motion. Additional analyses are included that demonstrate that sliding will not occur along deeper surfaces within the profile underlying the cask storage pads. First, the sliding resistance of the *in situ* silty clay/clayey silt layer is addressed to demonstrate that sliding will not occur along the interface between the bottom of the soil cement and those soils. These analyses demonstrate that if the pads were founded directly on the silty clay/clayey silt layer, the minimum factor of safety against sliding would be ~1.7. Therefore, the cask storage pads, embedded in soil cement, will have an adequate factor of safety against sliding.

Adequate factors of safety against sliding due to maximum forces from the design basis ground motion were obtained assuming that the storage pads were founded directly on the silty clay/clayey silt layer and conservatively ignoring the passive resistance of the soil cement that will be placed under and adjacent to the pads. In this case, much of the shearing resistance is provided by the cohesive portion of the 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, the sliding stability of the cask storage pads was analyzed assuming 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.

Analyses were performed to address the possibility that sliding may occur along a deep slip plane at the clayey soil/sandy soil interface as a result of the earthquake forces. To simplify the analysis, it was assumed that cohesionless soils extend above the 10 ft depth and, thus, the pads are founded directly on cohesionless materials. Because of the magnitude of the peak ground accelerations (0.53g) due to the design basis ground motion at this site, the frictional resistance available when the normal stress is reduced due to the uplift from the inertial forces applicable for the vertical component of the design basis ground motion is not sufficient to resist sliding. However, analyses were performed to estimate the amount of displacement that might occur due to the design basis ground motion for this case. These analyses, based on the method of estimating displacements of dams and embankments during earthquakes developed by Newmark (1965), indicate that even if these soils are cohesionless and even if they are conservatively located directly at the base of the pads, the estimated displacements would be less than ½ inch. Whereas there are no connections between the ground and these pads or between the pads and other structures, this minor amount of displacement would not adversely affect the performance of these structures if it did occur. Furthermore, the pads will be constructed on and within soil cement, which will be strong enough to resist sliding of the pads using only the passive resistance of the soil cement. This soil cement will effectively lock the pads in their respective locations, so that they can not move relative to one another.

ALLOWABLE BEARING CAPACITY OF THE CASK STORAGE PADS

STATIC BEARING CAPACITY OF THE CASK STORAGE PADS

Analyses of bearing capacity for static loads are summarized in Table 2.6-6. As indicated for Case IA, the factor of safety of the cask storage pad foundation is 6.3 using the undrained strength for the cohesive soils that was measured in the UU tests (s_u > 2.2 ksf) that were performed at depths of approximately 10 to 12 feet. The results for Case IB illustrates that the factor of safety against a bearing capacity failure increases to greater than 14 when the effective-stress strength of $\phi = 30°$ is used. Therefore, cases result in factors of safety against a bearing capacity failure that exceed the minimum allowable value of 3 for static loads. The minimum gross allowable bearing capacity exceeds 4 ksf for static loads.

DYNAMIC BEARING CAPACITY OF THE CASK STORAGE PADS

Analyses of bearing capacity for dynamic loads are summarized in Tables 2.6-7 and 2.6-8. Table 2.6-7 presents the results of the bearing capacity analyses based on the inertial forces applicable for the peak ground accelerations from the design basis ground motion. Table 2.6-8 presents the results of the analyses based on the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 1999) for the pad supporting 2 casks, 4 casks, and 8 casks. These latter dynamic forces represent the maximum force occurring at any time during the earthquake at each node in the model used to represent the cask storage pads. It is expected that these maximum forces will not occur at the same time for every node. These forces, therefore, represent an upper bound of the dynamic forces that could act at the base of the pad.

Table 2.6-7 presents the results of the dynamic bearing capacity analyses for the following cases, which include static loads plus inertial forces due to the earthquake.

Case II	100%	N-S direction,	0%	Vertical direction,	100%	E-W direction.
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.
Case IVA	40%	N-S direction,	100%	Vertical direction,	40%	E-W direction.
Case IVB	40%	N-S direction,	40%	Vertical direction,	100%	E-W direction.
Case IVC	100%	N-S direction,	40%	Vertical direction,	40%	E-W direction

As indicated in Table 2.6-7, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the inertial loads due to the design basis ground motion exceeds 7.7 ksf for all loading cases identified above. The minimum allowable value was obtained for Load Case II, wherein 100% of the earthquake loads act in the N-S and E-W directions and 0% acts in the Vertical direction, tending to rotate the cask storage pad about the N-S axis. The actual factor of safety for this condition was 2.3, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1).

Table 2.6-8 presents a summary of the bearing capacity analyses that were performed using the maximum dynamic cask driving forces developed for use in the design of the pads in Calculation 05996.02-G(PO17)-2 (CEC, 1999) for the pad supporting 2 casks, 4 casks, and 8 casks. As indicated in this table, the gross allowable bearing pressure for the cask storage pads to obtain a factor of safety of 1.1 against a shear failure from static loads plus the very conservative maximum dynamic cask driving forces due to the design basis ground motion is at least 8 ksf for the 2-cask, 4-cask, and 8-cask loading cases. The minimum allowable value (8.0 ksf) was obtained for the 8-cask loading. The actual factor of safety for this case was 1.3, which is greater than the criterion for dynamic bearing capacity (FS ≥ 1.1).

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STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE
05996.02	G(B)	04 - 6		78

TABLE 1

Summary of Vertical Soil Bearing Pressures (ksf) from Calc 05996.02-G(PO17)-2, Rev 1

[After adjusting snow loads to 0.045 ksf]

Loading	Point	A (287)	B (293)	C (299)	D (144)	E (150)	F (156)	G (1)	H (7)	J (13)
2-Cask	**Pad DL**	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.35	1.36	1.36	0.35	0.35	0.35	0.00	0.00	0.00
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24
	Cask EQ	2.22	1.64	1.81	0.67	0.48	0.45	0.00	0.00	0.00
	100% Vert	**4.30**	**3.73**	**3.90**	**1.75**	**1.56**	**1.53**	**0.73**	**0.73**	**0.73**
4-Cask	**Pad DL**	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.77	1.77	1.77	0.80	0.80	0.80	0.00	0.00	0.00
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24
	Cask EQ	1.97	1.70	1.92	1.87	1.23	1.31	0.00	0.00	0.00
	100% Vert	**4.47**	**4.20**	**4.42**	**3.40**	**2.76**	**2.84**	**0.73**	**0.73**	**0.73**
8-Cask	**Pad DL**	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45
	Snow LL	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045	0.045
	Cask LL	1.47	1.47	1.47	1.60	1.60	1.60	1.47	1.47	1.47
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24
	Cask EQ	2.70	2.39	2.13	2.82	1.44	2.24	3.92	2.42	2.47
	100% Vert	**4.90**	**4.59**	**4.33**	**5.15**	**3.77**	**4.57**	**6.12**	**4.62**	**4.67**

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TABLE 2.6-6

SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Static Loads

Case	F_V	$EQ_{H\ N\text{-}S}$	$EQ_{H\ E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\ E\text{-}W}$	β_L $EQ_{H\ N\text{-}S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
IA - Static Undrained Strength	3,716	0	0	0	0	0.0	0.0	13.05	**4.35**	0.0	0.0	30.0	64.0	**1.94**	**6.7**
IB - Static Effective Strength	3,716	0	0	0	0	0.0	0.0	28.34	**9.44**	0.0	0.0	30.0	64.0	**1.94**	**14.6**

ϕ =	**30**	Effective stress friction angle (deg), c=0.
c =	**2,200**	Undrained strength (psf), f=0.
γ =	**80**	Unit weight of soil (pcf)
B =	**30**	Footing width (ft)
L =	**64**	Footing length (ft)
D_f =	**2.7**	Depth of footing (ft)
γ_{surch} =	**100**	Unit weight of surcharge (pcf)
FS =	**3**	Factor of safety for static loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\ E\text{-}W}$ or $EQ_{H\ N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\ E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(

$\beta_L = \tan^{-1}[(EQ_{H\ N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(l

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2\ e_B$ $L' = L - 2\ e_L$

$q_{actual} = F_V / (B' \times L')$

[geot]\05996\calc\brng_cap\Pad\cu_phi.xls Table 2.6-6

STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET
CALCULATION IDENTIFICATION NUMBER
J.O. OR W.O. NO. 05996.02
DIVISION & GROUP G(B)
CALCULATION NO. 04 - 6
OPTIONAL TASK CODE

5010.65

TABLE 2.6-7

SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Inertial Forces Due to Design Earthquake: PSHA 2,000-Yr Return Period

Case	F_V	$EQ_{H\text{-}NS}$	$EQ_{H\text{-}EW}$	$\Sigma M_{EW\text{-}S}$	$\Sigma M_{EW\text{-}W}$	β_B $EQ_{H\text{-}EW}$	β_L $EQ_{H\text{-}NS}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
II	3,716	1,962	1,962	20,006	20,006	27.8	27.8	8.46	7.68	5.4	5.4	19.2	53.2	3.63	2.3
IIIA	1,736	785	785	8,002	8,002	24.3	24.3	11.39	10.35	4.6	4.6	20.6	54.8	1.52	7.5
IIIB	2,924	785	1,962	20,006	8,002	33.9	15.3	8.92	8.10	6.8	2.7	16.5	58.5	3.06	2.9
IIIC	2,924	1,962	785	8,002	20,006	15.3	33.9	10.52	9.56	2.7	6.6	24.5	50.3	2.37	4.4
IVA	5,697	785	785	8,002	8,002	7.8	7.8	11.91	10.83	1.4	1.4	27.2	61.2	3.42	3.5
IVB	4,538	785	1,962	20,006	8,002	23.5	9.9	9.93	9.03	4.4	1.8	21.1	60.5	3.53	2.8
IVC	4,538	1,962	785	8,002	20,006	9.9	23.5	10.99	9.89	1.8	4.4	26.5	55.1	3.06	3.5

c = 2,200 Total stress cohesion (psf)
ϕ = 0.0 Total stress friction angle (deg)
B = 30 Footing width (ft)
L = 64 Footing length (ft)
D_f = 2.7 Depth of footing (ft)
γ = 80 Unit weight of soil (pcf)
γ_{surch} = 100 Unit weight of surcharge (pcf)
FS = 1.1 Factor of safety for dynamic loads.

F_V = Vertical load (Static + EQ_V)
EQ_H = Earthquake Horizontal force. $F_H = EQ_{H\,E\text{-}W}$ or $EQ_{H\,N\text{-}S}$
$\beta_B = \tan^{-1}[(EQ_{H\,E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).
$\beta_L = \tan^{-1}[(EQ_{H\,N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).
$e_B = \Sigma M_{EW\text{-}S} / F_V$ $e_L = \Sigma M_{EW\text{-}W} / F_V$
$B' = B - 2\,e_B$ $L' = L - 2\,e_L$
$q_{actual} = F_V / (B' \times L')$

lgeo:\05996\calc\bearing_cap\Pad\wlughtxls Table 2.6-7

TABLE 2.6-8

SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Maximum Cask Driving Forces Due to Design Earthquake: PSHA 2,000-Yr Return Period for Loading Case IV: 100% N-S, 100% Vertical, and 100% E-W

Case IV	F_V	$EQ_{H\,N\text{-}S}$	$EQ_{H\,E\text{-}W}$	$\Sigma M_{B\,N\text{-}S}$	$\Sigma M_{B\,E\text{-}W}$	β_B $EQ_{H\,E\text{-}W}$	β_L $EQ_{H\,N\text{-}S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
2 Casks	2,647	768	909	9,873	13,103	19.0	16.2	9.62	8.93	3.73	4.95	22.1	22.5	5.32	1.8
4 Casks	4,633	1,265	1,373	13,867	27,290	16.6	15.3	9.77	8.88	2.99	5.89	24.0	36.2	5.32	1.8
8 Casks	8,755	2,247	2,511	30,818	34,320	14.8	14.4	8.30	8.00	3.52	3.92	23.0	56.2	6.79	1.3

c = 2,200 Total stress cohesion (psf)

ϕ = 0.0 Total stress friction angle (deg)

B = 30 Footing width (ft)

L = Varies Footing length (ft)

D_f = 2.7 Depth of footing (ft)

γ = 80 Unit weight of soil (pcf)

γ_{surch} = 100 Unit weight of surcharge (pcf)

FS = 1.1 Factor of safety for dynamic loads.

F_V = Vertical load (Static – EQ_V)

EQ_H = Earthquake: Horizontal force $F_H = EQ_{H\,E\text{-}W}$ or $EQ_{H\,N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\,E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\,N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$\Sigma M_{B\,N\text{-}S} = e_B \times F_V$ $\quad \Sigma M_{B\,E\text{-}W} = e_L \times F_V$

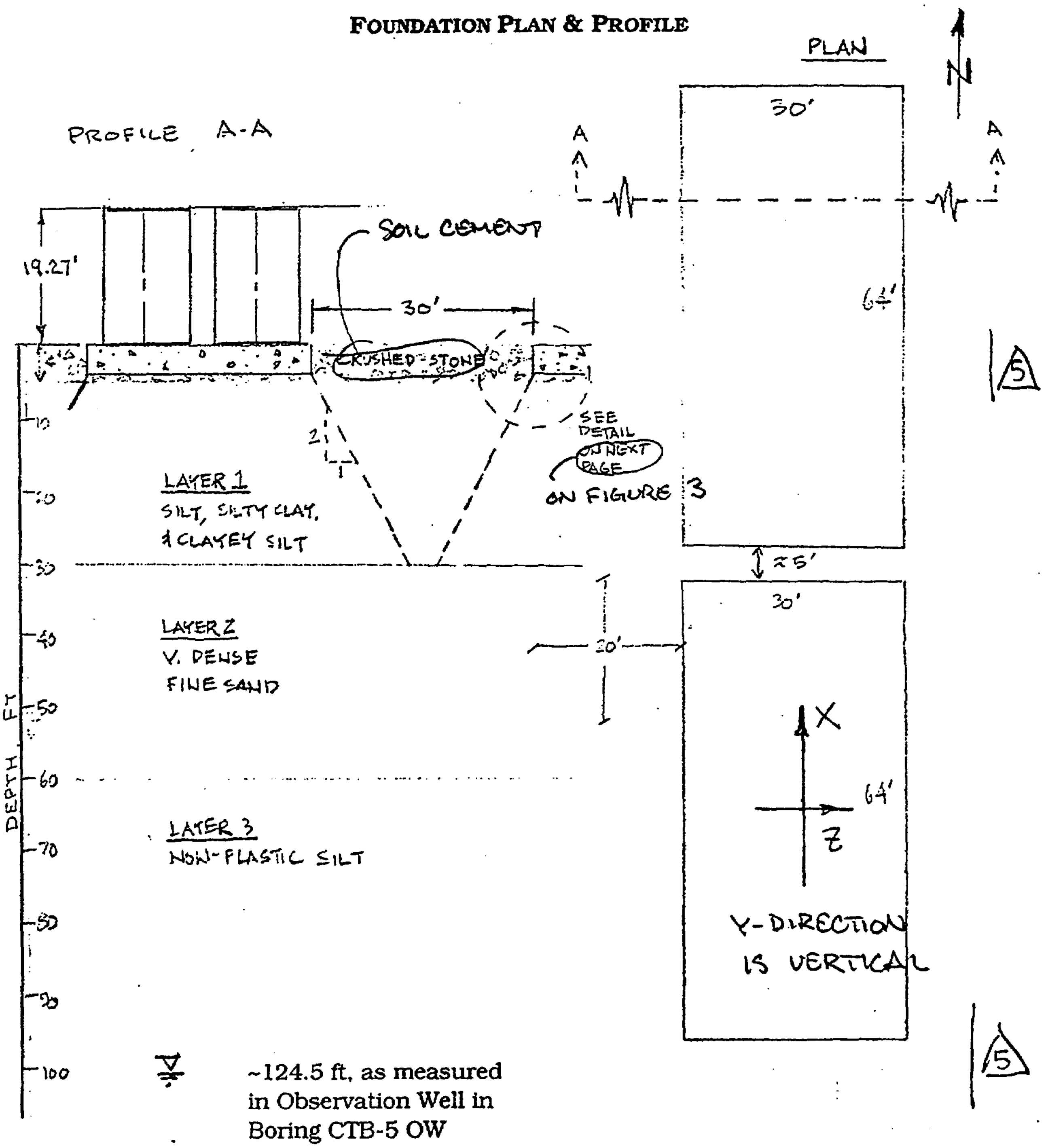
$B' = B - 2\,e_B$ $\quad L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

g:\...\05996\calcs\geng_cap\Pad\cu_phi.xls Table 2.6-8

FIGURE 1

FOUNDATION PLAN & PROFILE



Note: Plan view of pad from SWEC Drawing 0599601-EY-2E.
Cask details from Attachment C of Calc 05996.02-G(B)-05-1.

FIGURE 2

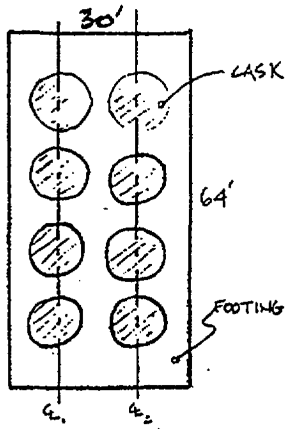
STATIC FOUNDATION LOAD / PRESSURE

TOTAL LOAD:

8 CASKS @ 356.5 K = 2852 K

30' × 64' × 3' × 0.15 K/ft³ = 864 K

∴ TOTAL LOAD = 3716 K



PLAN VIEW OF FOOTING

BEARING PRESSURE:

P_{actual} = LOAD/AREA

P_{actual} = 3716 K / 30' × 64'

P_{actual} = 1.94 KSF

Cask weight = 356.5K based on heaviest assembly weight shown on HI-STORM TSAR Table 3.2.1 (overpack with fully loaded MPC-32). See p C3 of Calc 05996.02-G(B)-05-1 for copy.

FIGURE 3

DETAIL OF SOIL CEMENT UNDER & ADJACENT TO CASK STORAGE PADS

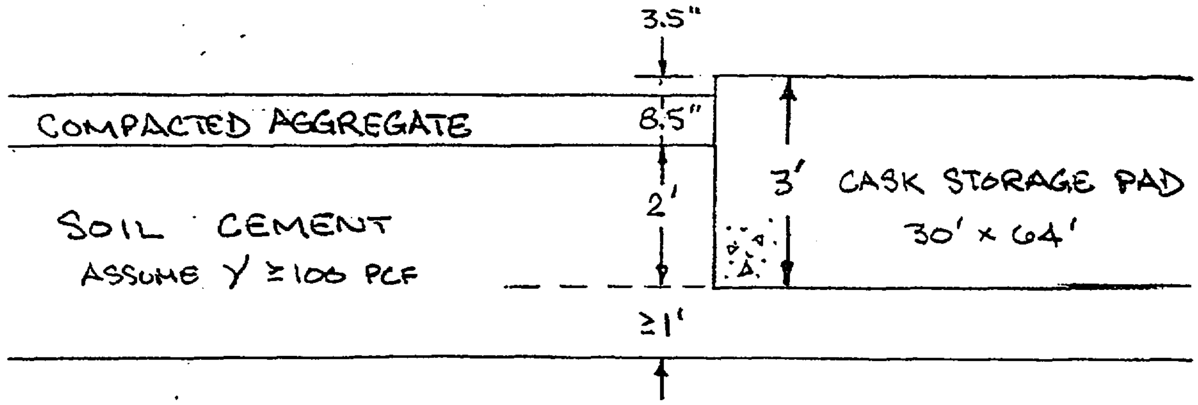
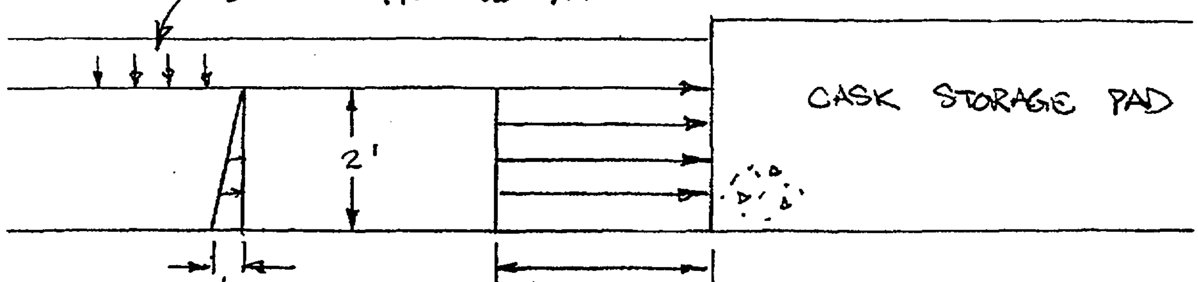


FIGURE 4

PASSIVE PRESSURE ACTING ON CASK STORAGE PADS

$$q_{DS} = 0.125 \frac{K}{FT^3} \times \frac{8.5 \text{ IN.}}{12 \text{ IN./FT}} = 0.09 \text{ KSF}$$



$p_p = \gamma H K_p$

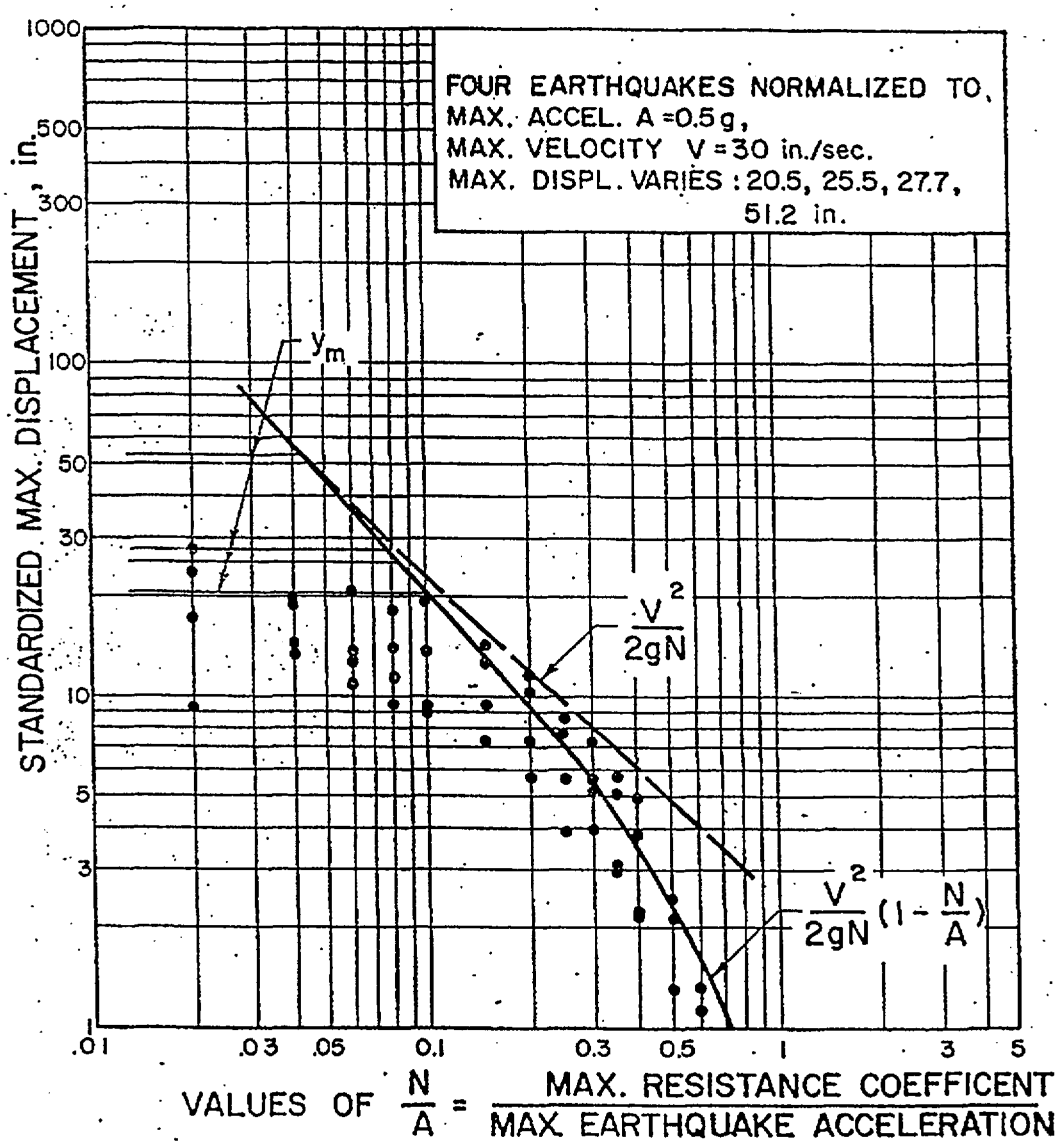
$$p_p = (2\bar{c} + q_{DS}) H \sqrt{K_p}$$

$$= (2\bar{c} + 0.09) \text{ KSF} \times 2 \text{ FT} \quad \text{FOR } \phi = 0$$

$$K_p = 1.0$$

FIGURE 5

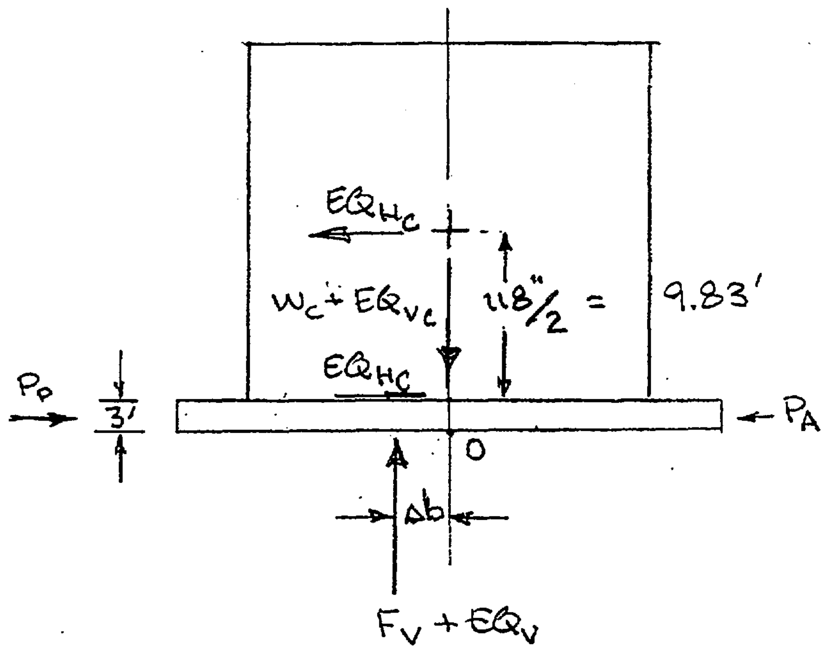
STANDARDIZED DISPLACEMENT FOR NORMALIZED EARTHQUAKES (SYMMETRICAL RESISTANCE)



From Newmark (1965)

FIGURE 6

DETERMINATION OF MOMENTS ACTING ON PAD DUE TO EARTHQUAKE LOADS FROM CASKS



$P_A << P_P$; therefore, It's conservative to Ignore both in ΣM.

Vertical reaction of cask load acts on the pad at an offset = Δb from the centerline of the cask.

$\sum M_{@\,\text{centerline}}$ to find Δb.

$$\Delta b \times (W_c + EQ_{VC}) = 9.83\text{ ft} \times EQ_{HC}$$

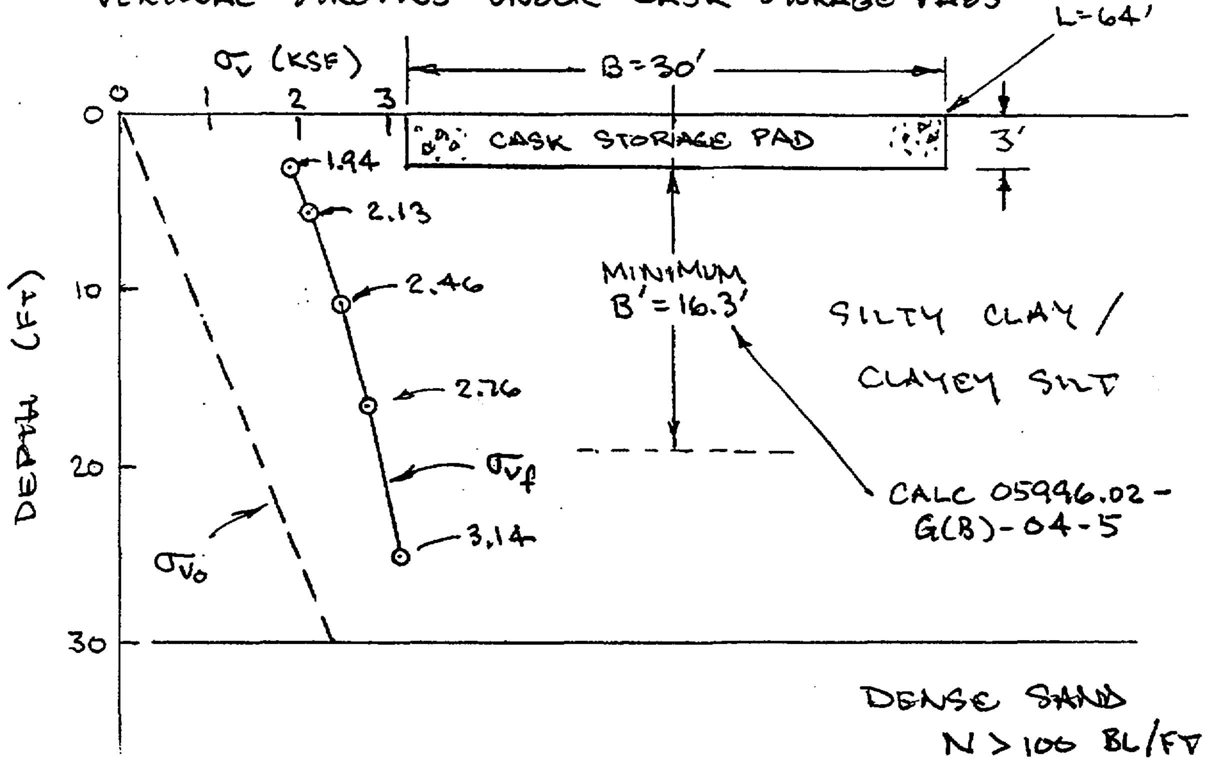
$\sum M_{@O}$ to find $\sum M_{@N\text{-}S}$

$$\sum M_{@N\text{-}S} = \underbrace{1.5\text{ ft} \times EQ_{HP}}_{\text{pad}} + \underbrace{3\text{ ft} \times EQ_{HC}}_{\text{cask horiz}} + \underbrace{\Delta b \times (W_c + EQ_{VC})}_{\text{cask vert}}.$$

Note: Moment arm of 3 ft is used for determining moment due to cask horizontal force, because casks are only resting on the pads — No connection exists to transmit moment to the pad.

FIGURE 7

VERTICAL STRESSES UNDER CASK STORAGE PADS



NOTE: AVG $\gamma_m \sim 80$ PCF FOR SOILS THAT WERE TESTED UNDER THE PAD EMPLACEMENT AREA, AS INDICATED ON p 4 OF CALC 05996.02-G(B)-05-1.

REF: CALC 05996.02-G(B)-03-3:

TABLE 3 => σ_{vf} AS $f(z)$

FIG 1 => σ_v AT BOTTOM OF 3' PAD = 1.935 KSF

FIG 7 => SUBLAYERS USED IN DET'G σ_v AS $f(z)$

ATTACHMENT A TO CALC 05996.01-G(B)-04-5 p A1/1

NOTES OF TELEPHONE CONVERSATION

JO No. 05996.01

PRIVATE FUEL STORAGE, LLC
PRIVATE FUEL STORAGE FACILITY

Date: 06-19-97
Time: 2:45 PM EDT

FROM:	Stan M. Macie	SWEC-Denver 1E		Tie Line 321-7305
	Wen Tseng	(ICEC)	Voice	(510) 841-7328
			(FAX)	(510) 841-7438
TO:	Paul J. Trudeau	SWEC-Boston 245/03		(617) 589-8473

SUBJECT: DYNAMIC BEARING CAPACITY OF PAD

DISCUSSION:

WTseng reported that his pad design analyses are being prepared for three loading cases: 2 casks, 4 casks, and 8 casks. The dynamic loads that he is using are based on the forcing time histories he received from Holtec. These forcing time histories were developed using a coefficient of friction between the cask and the pad of 0.2 and 0.8, where 0.2 provides the lower bound and 0.8 provides the upper bound loads from the cask to the pad.

He indicated that the bearing pressures at the base of the pad are greatest for the 2-cask dynamic loading case for $\mu = 0.8$ between the cask and the pad, because of eccentricity of the loading. For this case, the vertical pressures at the 30' wide loaded end of the pad are 5.77 ksf at one corner and 3.87 ksf at the other. He reported that it is reasonable to assume this pressure decreases linearly to 0 at a distance of ~32 ft; i.e., approximately half of the pad is loaded in this case. He also indicated that the horizontal pressure at the base of the pad is 1.04 ksf at the 30' wide end of the pad that is loaded by the 2 casks, and that this pressure decreases linearly over a distance of ~40' from the loaded end. He noted that the vertical pressures include the loadings (DL + dynamic loadings) of the casks and the pad, but the horizontal pressures apply only to the casks. Therefore, the inertia force of the whole pad must be added to the horizontal loads calculated based on the horizontal pressure distribution described above.

Since the table of allowable bearing pressures as a function of coefficient of friction between the cask and the pad that is in the design criteria does not include a value for $\mu = 0.8$, WTseng asked PJTrudeau to provide the allowable bearing pressure for this case.

SUPERSEDED BY ATT B 3

ACTION ITEMS:

PJTrudeau to determine the dynamic allowable bearing pressure for the 2-cask loading case.

COPY TO: NTGeorges Boston 245/03
SMMacie Denver 1E

~~00901~~

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
ORIGINATOR		DATE 9/20/99	CHECKED	DATE	9-20-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	234

5.3 Soil Pressures

5.3.1 Static Soil Pressure

Calculations of static soil pressure due to dead load (DL) and cask live load (LL) are given in Table S-1 and S-2, respectively.

Post-it® Fax Note 7671

Date 10-28-99	# of pages 11
To Paul Trudeau	From J. L. Cooper
Co./Dept.	Co.
Phone #	Phone #
Fax # 619-588-2959	Fax #

NOTE: CALC 05996.02-G(PO17)-2 REV 1
DATED 12-6-99 PER CALC INDEX

NOTED OCT 28 1999 P. J. Inlow

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
ORIGINATOR	kw	DATE 9/20/99	CHECKED	DATE	9-20-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	235

Table S - 1
Maximum Vertical Displacements and Soil Bearing Pressures
Dead Load

	k_s = 2.75 kcf	k_s = 26.2 kcf
Z_w(ft) =	0.164	0.0172
q_{zw}(ksf) =	0.45	0.45

Notes: 1. Z_w = maximum vertical displacement due to dead load (wt. of the pad only).

2. q_{zw} = vertical soil bearing pressure = $k_s \times Z_w$, where k_s = subgrade moduli = 2.75 and 26.2 kcf for lower-bound and upper-bound soils, respectively, and Z_w are obtained from CECSAP analysis results (Att. A).

CALCULATION SHEET

			CALC. NO.	G(PO17)-2	REV. NO. 0
ORIGINATOR	KV	DATE 9/20/99	CHECKED	[illegible]	DATE 9-20-99
PROJECT	Private Fuel Storage Facility				JOB NO. 1101-000
SUBJECT	Storage Pad Analysis and Design				SHEET 236

Table S - 2
Maximum Vertical Displacements and Soil Bearing Pressures
Live Load

Node No.	(Z_i)max (x10^{-2} ft.)							
	subgrade modulus = 2.75 kcf				subgrade modulus = 26.2 kcf			
	2 Casks	4 Casks	8 Casks	7 Casks + OLT	2 Casks	4 Casks	8 Casks	7 Casks + OLT
1	13.54	11.2	-53.28	-60.55	0.7244	1.22	-4.959	-5.451
7	13.5	11.19	-53.27	-44	0.7026	1.206	-4.966	-4.481
13	13.54	11.2	-53.28	-27.42	0.7244	1.22	-4.959	-3.479
144	-12.65	-27.63	-55.27	-81.67	-0.8428	-3.061	-6.121	-8.451
150	-12.74	-27.62	-55.24	-63.97	-0.8975	-3.061	-6.119	-6.723
156	-12.65	-27.63	-55.27	-46.31	-0.8428	-3.061	-6.121	-5.01
287	-43.58	-64.48	-53.28	-103	-5.152	-6.179	-4.959	-11.85
293	-43.63	-64.46	-53.27	-83.3	-5.178	-6.172	-4.966	-8.549
299	-43.58	-64.48	-53.28	-63.94	-5.152	-6.179	-4.959	-5.58
	Maximum Soil Bearing Pressure q_{zi}[1] (ksf)							
1	0	0	-1.465	-1.665	0	0	-1.299	-1.428
7	0	0	-1.465	-1.210	0	0	-1.301	-1.174
13	0	0	-1.465	-0.754	0	0	-1.299	-0.911
144	0.348	-0.760	-1.520	-2.246	0.221	0.802	-1.604	-2.214
150	0.350	-0.760	-1.519	-1.759	0.235	-0.802	-1.603	-1.761
156	0.348	-0.760	-1.520	-1.274	0.221	-0.802	-1.604	-1.313
287	-1.198	-1.773	-1.465	-2.833	-1.350	-1.619	-1.299	-3.105
293	-1.200	-1.773	-1.465	-2.291	-1.357	-1.617	-1.301	-2.240
299	-1.198	-1.773	-1.465	-1.758	-1.350	-1.619	-1.299	-1.462

Note:

1. $q_{zi} = k_s \times Z_i$ where k_s = 2.75 and 26.2 kcf for lower-bound and upper-bound subgrade moduli, respectively, and Z_i are obtained from CECSAP analysis results (Att. A)
2. Negative displacements imply downward movements.
3. The displacement values listed are taken from the selected 9 nodes. They are Node 1, 7, 13, 144, 150, 156, 287, 293, and 299. The locations of these nodes are shown in Figure 1. Their maximum displacement values may not be the local maxima. By close examination, it is determined that the nine values taken for each loading case have encompassed the maximum value for that case.
4. For snow load, the soil bearing pressure is .45 ksf (Ref. 5).

45 psf = 0.045 KSF
PER REV 3 OF
DESIGN CRITERIA

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
ORIGINATOR	[illegible]	DATE 9/20/99	CHECKED [illegible]	DATE	9-20-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	237

5.3.2 Dynamic Horizontal and Vertical Soil Pressures

Calculations of horizontal and vertical soil pressures due to dynamic cask driving forces resulting from earthquake motions are given in the following tables:

Table D-1(a) shows calculation of total maximum horizontal dynamic soil reactions in the X-direction (short direction of pad).

Table D-1(b) shows calculation of total maximum horizontal dynamic soil reactions in the Y-direction (long direction of pad).

Table D-1(c) shows a summary of total maximum horizontal dynamic soil reactions.

Table D-1(d) shows calculation of maximum vertical dynamic soil bearing pressures.

CALCULATION SHEET

				CALC. NO. G(PO17)-2	REV. NO. 0
ORIGINATOR		DATE	9/20/99	CHECKED	DATE 9-20-99
PROJECT	Private Fuel Storage Facility				JOB NO. 1101-000
SUBJECT	Storage Pad Analysis and Design				SHEET 238

Table D-1(a)
Total Maximum Horizontal Soil Reactions in the X Direction
Dynamic Load

	Maximum Displacement Xd ($\times 10^{-3}$ ft.)								
Node Number	LB			BE			UB		
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
1	6.106	3.738	33.63	3.256	1.974	17.72	1.673	1.380	10.29
7	6.110	3.736	33.68	3.256	1.975	17.73	1.674	1.379	10.31
13	6.106	3.739	33.64	3.256	1.972	17.73	1.673	1.377	10.30
144	8.131	15.69	16.72	4.406	8.923	17.88	2.335	5.129	10.75
150	8.130	15.69	16.72	4.408	8.928	17.89	2.333	5.097	10.76
156	8.137	15.69	16.70	4.409	8.933	17.89	2.338	5.061	10.75
287	22.76	34.77	34.90	12.26	19.48	18.14	6.776	10.68	10.89
293	22.76	34.78	34.92	12.27	19.48	18.16	6.777	10.70	10.90
299	22.76	34.76	34.91	12.27	19.46	18.16	6.776	10.68	10.89
Average	12.333	18.066	28.424	6.643	10.125	17.922	3.595	5.720	10.65
Kxd (kips/ft)	55188	55188	55188	102288	102288	102288	174240	174240	174240
Qxd (kips)	681	997	1569	680	1036	1833	626	997	1855

Notes:

1. Average = {sum(Xd)$_i$}/N; Xd=max. x-displ.; i=nodes 1,7,13,144,150,156,287,293,299; and N=9.
2. Qxd = Kxd x Average = total maximum horizontal-x soil reaction in Kips due to dynamic loading.
3. Kxd for LB, BE, and UB soils are dynamic horizontal-x soil spring stiffnesses given below:

(Kxd)LB =	4.60E+06 lb/in	Kxd)BE =	8.52E+06 lb/in	Kxd)UB =	1.45E+07 lb/in
	5.52E+04 Kips/ft		1.02E+05 Kips/ft		1.74E+05 Kips/ft

4. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.
5. Xd are obtained from CECSAP analysis results given in Att. A.
6. The maximum nodal displacements listed may not be concurrent. However, they are assumed to be concurrent for conservatism.
7. Node numbers are shown in Figure 1.

CALCULATION SHEET

			CALC. NO.	G(PO17)-2	REV. NO. 0
ORIGINATOR	KW	DATE 9/20/99	CHECKED	[signature]	DATE 9-20-99
PROJECT	Private Fuel Storage Facility				JOB NO. 1101-000
SUBJECT	Storage Pad Analysis and Design				SHEET 239

Table D-1(b)
Total Maximum Horizontal Soil Reactions in the Y Direction
Dynamic Load

Node Number	Max. Displacement Yd ($x10^{-3}$ ft.)								
	LB			BE			UB		
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
1	9.382	17.42	29.04	5.446	10.100	17.04	3.550	5.444	10.67
7	7.698	14.54	17.42	4.581	8.865	17.23	2.829	5.085	10.80
13	9.788	14.65	20.90	5.119	9.150	17.41	3.116	5.711	10.92
144	9.472	17.51	29.08	5.563	10.240	17.07	3.588	5.602	10.71
150	7.746	14.66	17.40	4.660	8.984	17.24	2.889	5.226	10.83
156	9.856	14.76	20.72	5.225	9.310	17.42	3.245	5.874	10.95
287	9.570	17.54	29.13	5.671	10.380	17.06	3.767	5.734	10.71
293	7.833	14.72	17.39	4.803	9.120	17.23	3.001	5.348	10.81
299	10.000	14.89	20.54	5.348	9.366	17.41	3.370	5.890	10.93
Average	9.036	15.632	22.402	5.157	9.502	17.234	3.262	5.546	10.814
Kyd (kips/ft)	52428	52428	52428	97176	97176	97176	165600	165600	165600
Qyd (kips)	474	820	1175	501	923	1675	540	918	1791

Notes:

1. Average=(sum(Yd)i)/N; Yd=max. y-displ.; i=nodes 1,7,13,144,150,156,287,293,299; and N=9.

2. Qyd = Kyd x Average = total maximum horizontal-y soil reaction in Kips due to dynamic loading.

3. Kyd for LB, BE, and UB soils are dynamic horizontal-y soil spring stiffnesses for entire pad given below:

(Kyd)LB =	4.37E+06 lb/in	(Kyd)BE	8.10E+06 lb/in	(Kyd)UB	1.38E+07 lb/in
	5.24.E+04 Kips/ft		9.72.E+04 Kips/ft		1.66.E+05 Kips/ft

4. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.

5. Yd are obtained from CECSAP analysis results given in Att. A.

6. The maximum nodal displacement listed may not be concurrent. However, they are assumed to be concurrent for conservatism.

7. Node numbers are shown in Figure 1.

CALCULATION SHEET

			CALC. NO.	G(PO17)-2	REV. NO. 0
ORIGINATOR	[signature]	DATE 9/20/99	CHECKED	[signature]	DATE 9-23-99
PROJECT	Private Fuel Storage Facility				JOB NO. 1101-000
SUBJECT	Storage Pad Analysis and Design				SHEET 240

Table D-1(c)
Summary of Total Maximum Horizontal Soil Reactions
Dynamic Load

	Max. Soil Reaction (Kips)									
	LB			BE			UB			
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	
Qxd =	681	997	1569	680	1036	1833	626	997	1855	E-W
Qyd =	474	820	1175	501	923	1675	540	918	1791	N-S

Notes:

1. Qxd and Qyd in Kips are calculated in Tables D-1(a) and (b), respectively.

2. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.

CEC

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
ORIGINATOR	[signature]	DATE 9/20/99	CHECKED [signature]	DATE	9-20-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	241

Table D-1(d)
Maximum Vertical Soil Bearing Pressures
Dynamic Load

Node Number	Maximum Displacement Zd ($\times 10^{-3}$ ft.)								
	LB			BE			UB		
	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks	2 Casks	4 Casks	8 Casks
1	6.046	13.58	-30.77	4.002	7.599	-50.25	1.945	5.852	-33.37
7	6.421	9.074	-29.91	3.341	5.761	-24.61	1.955	3.728	-20.64
13	9.799	14.73	-47.10	4.855	10.53	-27.68	2.379	6.073	-21.03
144	-12.78	-24.37	-30.63	-9.079	-22.41	-29.56	-5.715	-15.900	-23.99
150	-6.301	-12.57	-16.70	-5.213	-12.41	-15.86	-4.055	-10.450	-12.29
156	-10.13	-25.14	-21.34	-5.896	-13.95	-29.82	-3.801	-11.180	-19.07
287	-26.50	-35.51	-69.21	-23.67	-27.08	-25.68	-18.900	-16.760	-14.97
293	-21.77	-32.04	-61.38	-17.39	-22.58	-21.37	-14.010	-14.500	-15.10
299	-26.01	-37.77	-54.79	-29.69	-22.41	-26.55	-15.430	-16.340	-16.84
	Maximum Soil Bearing Pressure q_{zd} (Kips/ft^2)								
1	0.00	0.00	-1.20	0.00	0.00	-3.53	0.00	0.00	-3.92
7	0.00	0.00	-1.16	0.00	0.00	-1.73	0.00	0.00	-2.42
13	0.00	0.00	-1.83	0.00	0.00	-1.94	0.00	0.00	-2.47
144	-0.50	-0.95	-1.19	-0.64	-1.57	-2.08	-0.87	1.87	-2.82
150	-0.25	-0.49	-0.65	-0.37	-0.87	-1.11	-0.48	-1.23	-1.44
156	-0.39	-0.98	-0.83	-0.41	-0.98	-2.09	-0.45	-1.31	-2.24
287	-1.03	-1.38	-2.70	-1.66	-1.90	-1.80	-2.22	-1.97	-1.76
293	-0.85	-1.25	-2.39	-1.22	-1.69	-1.50	-1.64	-1.70	-1.77
299	-1.01	-1.47	-2.13	-2.09	-1.57	-1.87	-1.81	-1.92	-1.98

Notes:

1. q_{zd} = maximum soil bearing pressure = (Kzd x Z_d)/A, where A = 64' x 30' = 1920 ft^2.
2. Kzd for LB, BE, and UB soils are vertical-z dynamic soil spring stiffnesses given below:

(Kzd)LB =	6.23E+06 lb/in	(Kzd)BE =	1.12E+07 lb/in	(Kzd)UB =	1.88E+07 lb/in
	7.48.E+04 Kips/ft		1.35.E+05 Kips/ft		2.25.E+05 Kips/ft

3. LB = lower-bound soil, BE = best-estimate soil, UB = upper-bound soil.
4. Zd are obtained from CECSAP analysis results given in Att. A.
5. Negative displacements imply downward movements.
6. The maximum Zd values listed above may not be concurrent. However they are assumed to be concurrent values and concurrent signs are assigned to them.
7. Node numbers are shown in Figure 1.

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
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SUBJECT	Storage Pad Analysis and Design			SHEET	311

6.2 Vertical Soil Bearing Pressures and Horizontal Soil Shear Stresses

Vertical soil bearing pressures for individual loadings and combined loadings are summarized as shown in Table 5.

Horizontal soil shear stresses are shown in Tables D-1(a) and (b), and the total horizontal soil reactions (shear forces) in both the short (x) and long (y) directions of the pad are summarized in Table D-1(c).

CALCULATION SHEET

			CALC. NO. G(PO17)-2	REV. NO.	0
ORIGINATOR	mr	DATE 9/20/99	CHECKED	DATE	9-20-99
PROJECT	Private Fuel Storage Facility			JOB NO.	1101-000
SUBJECT	Storage Pad Analysis and Design			SHEET	312

Table 5

Summary of Vertical Soil Bearing Pressures (ksf)

Node Number		287	293	299	144	150	156	1	7	13	
2-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	
	Snow LL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	*
	Cask LL	1.35	1.36	1.36	0.35	0.35	0.35	0	0	0	
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	
	Cask EQ	2.22	1.64	1.81	0.67	0.48	0.45	0	0	0	
	100% Ve	4.71	4.14	4.31	2.16	1.97	1.94	1.14	1.14	1.14	
4-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	
	Snow LL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	*
	Cask LL	1.77	1.77	1.77	0.80	0.80	0.80	0	0	0	
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	
	Cask EQ	1.97	1.70	1.92	1.87	1.23	1.31	0	0	0	
	100% Ve	4.88	4.61	4.83	3.81	3.17	3.25	1.14	1.14	1.14	
8-Cask	Pad DL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	
	Snow LL	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	0.45	*
	Cask LL	1.47	1.47	1.47	1.60	1.60	1.60	1.47	1.47	1.47	
	Pad EQ	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	
	Cask EQ	2.70	2.39	2.13	2.82	1.44	2.24	3.92	2.42	2.47	
	100% Ve	5.31	5.00	4.74	5.66	4.18	4.98	6.53	5.03	5.08	

Notes:
(1) Values for Pad DL are obtained from Table S-1.
(2) Values for Snow LL are obtained from Table S-2.
(3) Values for Cask LL are obtained from Table S-2.
(4) Pad EQ pressure = (pad wt.) x a_v, where pad wt. = 864 kips, and a_v = 0.533g.
(5) Values for Cask EQ are obtained from Table D-1(d).
(6) EQ pressures listed are the envelopes of results for all soil conditions.
(7) Node numbers are shown in Figure 1.

* SNOW LOAD SHOULD BE 0.045 KSF (i.e., 45 psf); ∴ ADJUST "100% Ve" LINES ACCORDINGLY.

CALCULATION SHEET

ORIGINATOR	[illegible]	DATE 9-23-99	CALC. NO. G(PO17)-2 / REV. NO. 0
		CHECKED DH	DATE 9/23/99
PROJECT	Private Fuel Storage Facility		JOB NO. 1101-000
SUBJECT	Storage Pad Analysis and Design		SHEET NO. 23

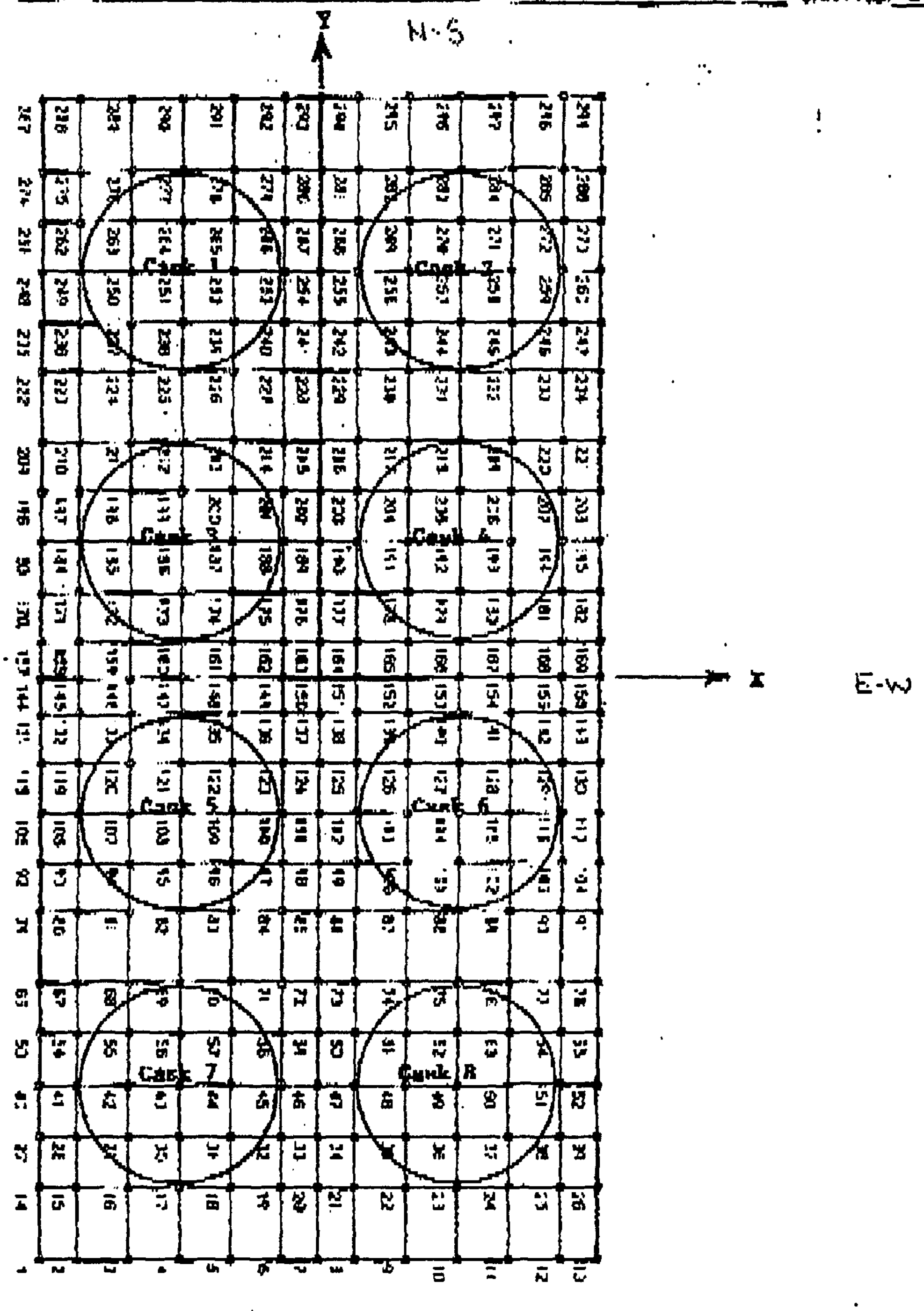


Figure 1 CECSAP Finite-Element Model with Node Numbers

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION SHEET

J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE
05996.02	G(B)	05-2		25K

CALCULATION IDENTIFICATION NUMBER

TABLE 6

SUMMARY OF TRIAXIAL TEST RESULTS FOR SOILS WITHIN ~10 FT OF GROUND SURFACE AT THE SITE

Boring	Sample	Depth ft	Elev ft	w %	ATTERBERG LIMITS LL	PL	PI	USC Code	γ_m pcf	γ_d pcf	e_o	σ_c ksf	s_u ksf	ε_a %	Type	Date
B-1	U-2C	5.9	4453.9	47.1	66.1	33.4	32.7	MH	79.3	53.9	2.15	0.3	2.03	1.7	CU	Nov '99
B-1	U-2B	5.3	4454.5	52.9	80.6	40.9	39.7	MH	70.6	46.3	2.67	1.0	2.21	6.0	CU	Nov '98
B-4	U-3D	10.4	4462.1	27.4	42.5	24.7	17.8	CL	85.5	67.1	1.53	1.3	2.18	4.0	CU	Jan '97
C-2	U-2D	11.1	4453.4	35.6	See U-2C & E[1]			CL	78.5	57.9	1.93	1.3	2.39	11.0	CU	Jan '97
CTB-1	U-3D	6.7	4463.7	47.9	See U-3C[2]			CH	91.9	62.1	1.73	1.7	2.84	5.0	CU	June '99
CTB-4	U-2D	9.5	4465.5	45.2	See U-2E[2]			CH	87.7	60.4	1.31	1.7	3.11	6.0	CU	June '99
CTB-6	U-3D	8.3	4457.9	52.7				CH	85.7	56.2	2.02	1.7	2.70	7.0	CU	June '99
CTB-N	U-1B	5.7	4456.4	30.1	41.3	22.5	18.8	CL	100.6	77.3	1.20	1.7	3.00	8.0	CU	Nov '98
CTB-N	U-2B	7.7	4456.4	65.4	See U-2A[2]			MH	74.6	45.1	2.76	1.7	2.41	13.0	CU	June '99
CTB-X	U-3D	10.5	4433.6	52.2	61.1	30.8	30.3	CH	86.3	56.7	1.98	1.7	2.73	7.0	CU	June '99
CTB-S	U-1B	5.6	4435.7	73.6	66.2	40.9	25.3	MH	78.0	44.9	2.78	1.7	2.05	12.0	CU	Nov '98
CTB-S	U-2D	8.4	4436.1	54.8	57.9	28.9	29.0	CH	90.0	58.2	1.92	1.7	2.43	5.0	CU	June '99
B-1	U-2D	6.5	4453.3	45.2	59.8	34.7	25.1	MH	76.7	52.8	2.22	2.1	3.28	15.0	CU	Mar '99
B-3	U-1B	5.2	4463.0	35.5	52.4	25.2	27.2	MH	90.6	67.9	1.50	2.1	3.55	5.0	CU	Mar '99
C-2	U-1D	6.2	4459.2	50.5	70.8	41.8	29.0	MH	74.5	49.5	2.48	2.1	3.03	12.0	CU	Mar '99

NOTES 1 Attachment 2 of SAR Appendix 2A.

2 Attachment 6 of SAR Appendix 2A.

Attachment C p C1/3

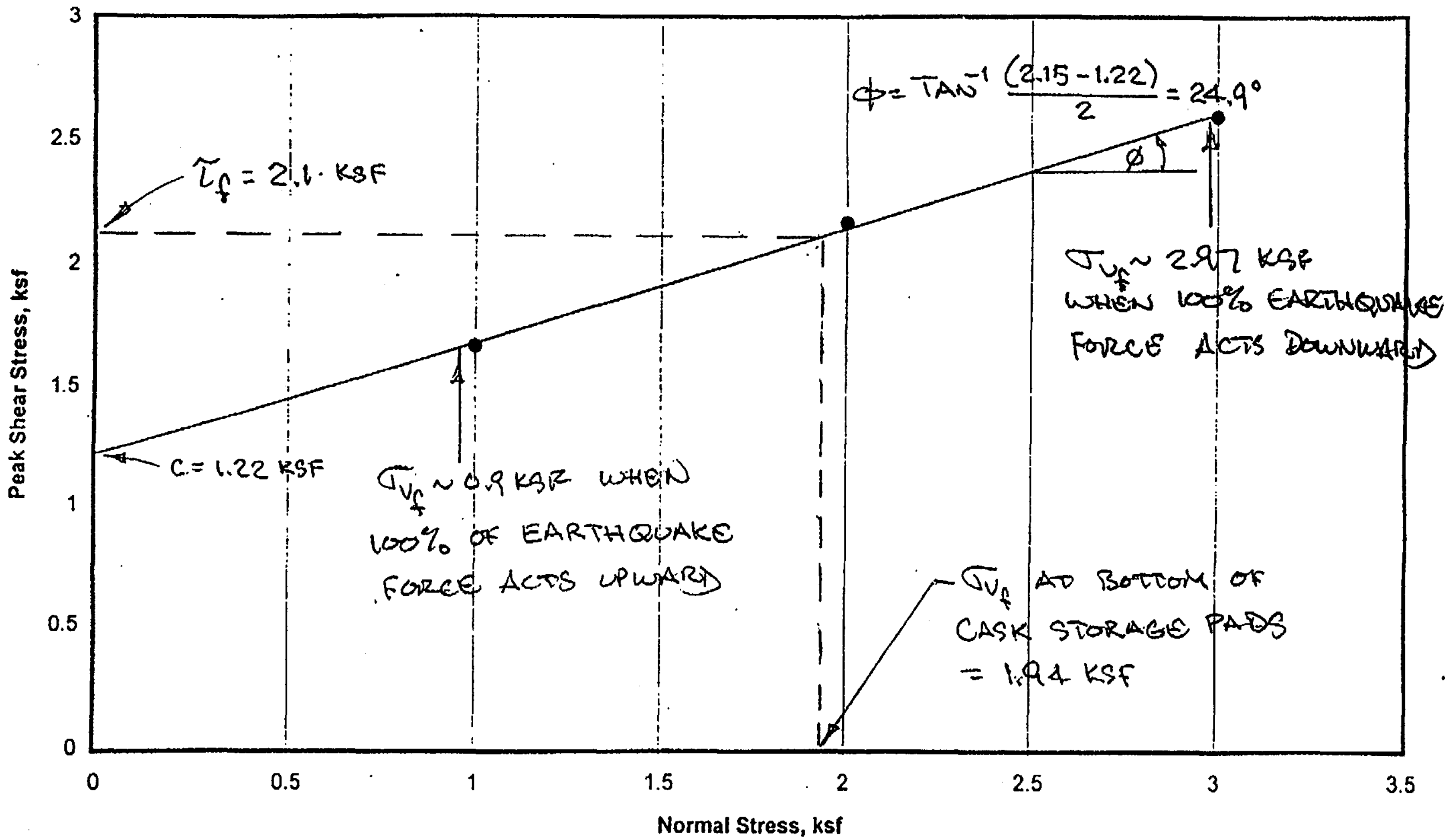
CALC 05996.02-G(B)-04-6

FIGURE 7

DIRECT SHEAR TEST

Boring C-2, Sample U-1C

PAD EMPLACEMENT AREA



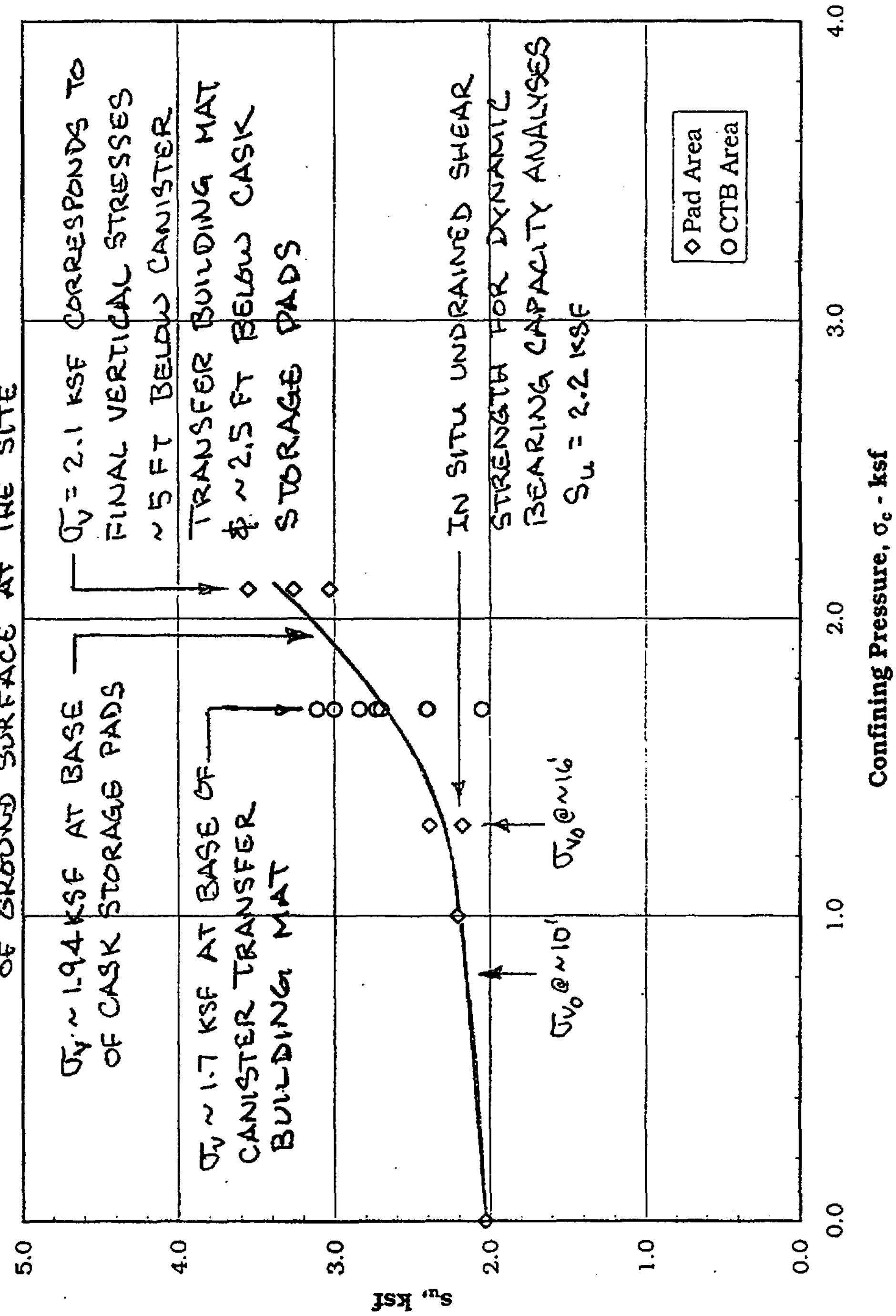
CALC 05996.02-G(B)-05-½ p 32

ATTACHMENT C p C2
CALC 05996.02-G(B)-04-6

REF SAR APP 2A ATT 7

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 36
05996.02	G(B)	05-2		

Figure 11
Summary of Triaxial Test Results for Soils Within Depth of ~10 ft
OF GROUND SURFACE AT THE SITE



ATTACHMENT C P C3/3
CALC 05996.02-G(B)-04-6

[geot]\05996\lab\triax-1.xls on 6/15/2000

QA CATEGORY I
CALCULATION CHECKLIST

Calculation No. 05996.02-G(B)-04
Revision No. 6

Project No. 05996.02
Job Book File Location Q2.9

	Yes	No	N/A
Method			
Identify the method used to verify the "Method" of the calculation			
• By design review	✓		
• Compare the Method with another calculation			✓
• Alternate calculation			✓
If the compare method was used, is the statement identifying the other calculation identified in this calculation?			✓
If an alternate calculation was used for a QA Category I calculation, is it included with the calculation?			✓
Is the calculation method acceptable?	✓		
Assumptions			
Affirmative answers to the following questions are required:			
• Are all assumptions uniquely identified as assumptions and adequately described?	✓		
• Are all assumptions reasonable?	✓		
• Are all assumptions that require confirmation at a later date specifically identified as assumptions that must be confirmed?	✓		
For Revisions to the Calculation			
• Are changes clearly identified?	✓		
• For QA Category I calculations, is a reason for the revision given?	✓		
• Does the calculation identify the calculation, including revision, when applicable, which is superseded?	✓		

QA CATEGORY I
CALCULATION CHECKLIST

Calculation No. 05996.02-G(B)-04
Revision No. 6

Project No. 05996.02
Job Book File Location Q2.9

	Yes	No	N/A
• Are affected pages identified with the new calculation number or revision number?	✓		
• When applicable, is an alternate calculation included as part of the calculation?			✓
• When applicable, is a statement identifying the calculation to which the method was compared included as part of the revision?			✓

Thomas Y. Chang	Thomas Y. Chang	6-16-2000
Printed Name	Signature	Date

Exhibit A

Attachment 2

STONE & WEBSTER ENGINEERING CORPORATION

CALCULATION TITLE PAGE

*SEE INSTRUCTIONS ON REVERSE SIDE

▲5010.64 (FRONT)

CLIENT & PROJECT	PAGE 1 OF 52 + 6 pp OF ATTACH'S
PRIVATE FUEL STORAGE, LLC - PRIVATE FUEL STORAGE FACILITY	
CALCULATION TITLE (Indicative of the Objective): STABILITY ANALYSES OF THE CANISTER TRANSFER BUILDING SUPPORTED ON A MAT FOUNDATION	QA CATEGORY (✓) ☒ I - NUCLEAR SAFETY RELATED ☐ II ☐ III ☐ ____ OTHER

CALCULATION IDENTIFICATION NUMBER				
J. O. OR W.O. NO.	DIVISION & GROUP	CURRENT CALC. NO.	OPTIONAL TASK CODE	OPTIONAL WORK PACKAGE NO.
05996.02	G(B)	13	—	345AD

*APPROVALS - SIGNATURE & DATE: PREPARER(S)/DATE(S)	REVIEWER(S)/DATE(S)	INDEPENDENT REVIEWER(S)/DATE(S)	REV. NO. OR NEW CALC NO.	SUPERSEDES * CALC. NO. OR REV. NO.	CONFIRMATION *REQUIRED (✓) YES	NO
L.P. SINGH (signature) 12-9-98	D.L. Aloysius (signature) 12-10-98	D.L. Aloysius (signature) 12-10-98	0	-		✓
DLALOYSIUS 9-3-99 (signature) SYBOAKYE 9-3-99 (signature)	SYBOAKYE 9-3-99 (signature) DLALOYSIUS 9-3-99 (signature)	TY CHANG 9-3-99 Thomas Y. Chang T Y Chang 9-3-99 Thomas Y. Chang	1	G(C)-13 REV 0		✓
SEE PAGE 2-1 FOR ID OF PREPARED/REVIEWED BY						
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RECORD OF REVISIONS

REVISION 0

Original Issue

REVISION 1

Page count increased from 37 to 63.

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

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

REVISION 2

Major re-write of the calculation.

1. Renumbered pages and figures to make the calculation easier to follow.
2. Changed effective length of mat to 265 ft to make it consistent with Calculation 05996.02-SC-4, Rev 1 (SWEC, 1999a).
3. Added overturning analysis.
4. Corrected calculation of moments for joints 3 and 6 in Table 1 and incorporated revised seismic loads in calculations of overturning stability and dynamic bearing capacity.
5. Revised dynamic bearing capacity analyses to utilize only total strength parameters because these partially saturated soils will not have time to drain fully during the rapid cycling associated with the design basis ground motion. See Calculation 05996.02-G(B)-05-1 (SWEC, 1999b) for additional details.
6. Updated references to current issues of drawings.
7. Added references to foundation profiles through Canister Transfer Building area presented in SAR Figures 2.6-21 through 23.

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

REVISION 3

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

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

OBJECTIVE

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

ASSUMPTIONS/DATA

The footprint of the Canister Transfer Building foundation mat is shown on SWEC Drawing 0599601-EA-8-D, Canister Transfer Building – Floor Plan, and Drawing 0599601-EM-1-D, Canister Transfer Building – General Arrangement Sheet 1. The elevation view of the structure is shown on Drawing 0599601-EA-9-D, Canister Transfer Building – Elevations Sheet 1, and Drawing 0599601-EM-1-D, Canister Transfer Building – General Arrangement Sheet 2. As indicated in SAR Section 4.7.1.5.1, Structural Design, the mat foundation is 5 ft thick. The foundation mat is modeled as 165 ft x 265 ft x 5 ft thick. These are the effective dimensions that were developed and used in Calculation 05996.02-SC-4, Rev 1 (SWEC, 1999a).

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

The various static and dynamic loads and load combinations used in these analyses were obtained from Calculation 05996.02-SC-5-1 (SWEC, 1999b). All loads are transferred to the bottom of the mat. Moments, when transferred to the bottom of the mat, result in eccentricity of the applied load with respect to the center of gravity of the mat. Lateral loads, when combined with the vertical load, result in inclination of the vertical load, which decreases the allowable bearing capacity.

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

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

Geotechnical Properties

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

Table 6 of Calc 05996.02-G(B)-05-2 (copy included in Attachment A) summarizes the results of the triaxial tests that were performed within depths of ~10 ft. The undrained shear strengths (s_u) measured in these tests are plotted vs confining pressure in Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment A). This figure is annotated to indicate the vertical stresses existing prior to construction and following completion of construction.

The undrained shear strengths measured in the triaxial tests are used for the dynamic bearing capacity analyses because the soils are partially saturated and they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. As indicated in Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment A), the undrained strength of the soils within ~10 ft of grade is assumed to be 2.2 ksf. This value is the lowest strength measured in the UU tests, which were performed at confining stresses of 1.3 ksf. This confining stress corresponds to the in situ vertical stress existing near the middle of the upper layer, prior to construction of these structures. It is much less than the final stresses that will exist under the cask storage pads and the Canister Transfer Building following completion of construction. Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment A) illustrates that the undrained strength of these soils increase as the loadings of the structures are applied; therefore, 2.2 ksf is a very conservative value for use in the bearing capacity analyses of these structures.

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

In determining the bearing capacity of the foundation, the average shear strength of the soils along the anticipated bearing capacity failure slip surface should be used. This slip surface is normally confined to the zone within a depth below the footing equal to the minimum width of the footing. For the Canister Transfer Building, the effective width of the footing is decreased because of the large eccentricity of the load on the mat due to the seismic loading. As indicated in Table 2.6-10, the minimum effective width of the Canister Transfer Building occurs for Load Case IIIA, where B' = 38.2 ft. This is greater than the depth of the upper layer (~30 ft). Therefore, it is reasonable to use the average strength of the soils in the upper layer in the bearing capacity analyses, since all of the soils in the upper layer will be effective in resisting failure along the anticipated bearing capacity slip surface.

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

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

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

Further evidence that this is a conservative value of s_u for the soils in the upper layer is presented in Figure 11 of Calc 05996.02-G(B)-05-2 (copy included in Attachment A). This plot of s_u vs confining pressure illustrates that this value is slightly less than the average value of s_u measured in the CU triaxial tests that were performed on specimens obtained from depths of ~10 ft at confining stresses of 2.1 ksf. As indicated in this figure, the confining stress of 2.1 ksf used to test these specimens is comparable to the vertical stress that will exist ~5 ft below the Canister Transfer Building mat following completion of construction. Since these tests were performed on specimens of the weakest soils underling the Canister Transfer Building mat (the deeper lying soils are stronger based on the SPT and the cone penetration test data), it is conservative to use the weighted average value of s_u of 3.18 ksf for the soils in the entire upper layer of the profile in the bearing capacity analyses.

Direct shear tests were performed on undisturbed specimens of the silty clay/clayey silt obtained from Borings CTB-6 and CTB-S, which were drilled in the locations shown in SAR Figure 2.6-18. These specimens were obtained from Elevation ~4469, the elevation of the bottom of the 1-ft deep perimeter key proposed at the base of Canister Transfer Building mat. Note, this key is being constructed around the perimeter of the mat to ensure that the full shear strength of the clayey soils is available to resist sliding of the structure due to loads from the design basis ground motion. These direct shear tests were performed at normal stresses that ranged from 0.25 ksf to 3.0 ksf. This range of normal stresses bounds the ranges of stresses expected for static and dynamic loadings from the design basis ground motion.

The results of these tests are presented in Attachments 7 and 8 of the Appendix 2A of the SAR and they are plotted in Figures 9 and 10 of Calc 05996.02-G(B)-05-2 (copies included in Attachment A). Because of the fine grained nature of these soils, they will not drain completely during the rapid cycling of loadings associated with the design basis ground motion. Therefore, sliding stability analyses included below of the Canister Transfer Building constructed directly on the silty clay are performed using the average shear strength measured in these direct shear tests for a normal stress equal to the vertical stress under the building following completion of construction, but prior to imposition of the dynamic loading due to the earthquake. As shown in Figures 9 and 10 of Calc 05996.02-G(B)-05-2 (copies included in Attachment A), this average shear strength is 1.8 ksf and the friction angle is set equal to 0°.

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

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

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

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

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

METHOD OF ANALYSIS

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

The following load combinations are analyzed:

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

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

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

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

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

ANALYSIS OF OVERTURNING STABILITY

The factor of safety against overturning is defined as:

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

The overturning stability of the Canister Transfer Building is determined using the dynamic loads for the building due to the PSHA 2,000-yr return period earthquake. These loads are listed in Table 1 (SAR Table 2.6-11), and they were developed based on the dynamic analysis performed in Calculation 05996.02-SC-5 (SWEC, 1999b) and described in SAR Section 4.7.1.5.3. The masses and accelerations of the joints (see Figure 2 for locations of the joints) used in the model of the Canister Transfer Building in Calculation 05996.02-SC-5 are listed on the left side of Table 1, and the resulting inertial forces and associated moments are listed on the right. Based on building geometry shown schematically in Figure 1 and the forces and moments shown in Table 1, overturning is more critical about the N-S axis (~265 ft) than about the E-W axis (~165 ft).

The resisting moment is calculated as the weight of the building x the distance from one edge of the mat to the center of the mat. The weight of the building is 72,988 K, as shown in Table 1. For overturning about the N-S axis, the moment arm for the resisting moment equals ½ of ~165 ft, or 82.5 ft. Therefore,

$\Sigma M_{Resisting}$ = 72,988 K x 82.5 ft = 6,021,510 ft-K.

The driving moments include the ΣM acting about the N-S axis, ΣM_X in Table 1, which is 2,513,041 ft-K, and the moment due to the uplift force ($\Sigma F_{V\,dyn}$ = 57,139 K) x ½ the width of the mat. The vertical force due to the earthquake can act upward or downward. However, when it acts downward, it acts in the same direction as the weight, tending to stabilize the structure. Therefore, the minimum factor of safety against overturning will occur when the dynamic vertical force acts in the upward direction, tending to unload the mat.

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

$$\sum M_{Driving} = \sqrt{2{,}513{,}041^2 + (57{,}139\text{ K} \times 82.5\text{ ft})^2} = 5{,}341{,}991 \quad \text{ft} - \text{K}$$

and FS_{OT} = 6,021,510 ÷ 5,341,991 = 1.13 about the N-S axis.

Checking overturning about the E-W axis (~165 ft), the resisting moment is calculated as the weight of the building x the distance from one edge of the mat to the center of the mat. The weight of the building is 72,988 K, as shown in Table 1. For overturning about the E-W axis, the moment arm for the resisting moment equals ½ of ~265 ft, or 132.5 ft. Therefore,

$\Sigma M_{Resisting}$ = 72,988 K x 132.5 ft = 9,670,910 ft-K.

The driving moments include the ΣM acting about the E-W axis, ΣM_Y in Table 1, which is 1,961,325 ft-K, and the moment due to the uplift force ($\Sigma F_{V\,dyn}$ = 57,139 K) x ½ the length of the mat. The vertical force due to the earthquake can act upward or downward. However, when it acts downward, it acts in the same direction as the weight, tending to stabilize the structure. Therefore, the minimum factor of safety against overturning will occur when the dynamic vertical force acts in the upward direction, tending to unload the mat.

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

$$\sum M_{Driving} = \sqrt{1{,}961{,}325^2 + (57{,}139\ \text{K} \times 132.5\ \text{ft})^2} = 7{,}820{,}843 \quad \text{ft}-\text{K}$$

and $\quad FS_{OT} = 9{,}670{,}910 \div 7{,}820{,}843 = 1.24$ about the E-W axis.

These values are greater than the criterion of 1.1; therefore, the Canister Transfer Building has an adequate factor of safety against overturning due to dynamic loadings from the design basis ground motion.

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."

$B = 165$ feet

$L = 265$ 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, SWEC, 1999b). In this case, the strength of the clayey soils at the bottom of the 1-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.

Conservatively assume the backfill to be placed around the Canister Transfer Building mat and 1-ft deep key will be the eolian silt that was excavated from the area. For these soils, it is reasonable to assume the lower bound value of γ is 80 pcf, $\phi = 30°$ & c = 0.

$$K_p = \left(\frac{1+\sin\phi}{1-\sin\phi}\right) = 3.0 \text{ for } \phi = 30°$$

For cohesionless soils, $P_p = 0.5 \times \gamma H^2 K_p$

$P_p = 0.5 \times 0.080 \text{ kcf} \times (6 \text{ ft})^2 \times 3.0 = 4.32 \text{ k/LF}$

Based on Drawing 0599602-EC-2-A (See Figure 5), the CTB mat is actually 35' + 145' + 35' = 215' wide in the E-W direction and 182' + 60' + 30' = 272' long in the N-S direction. Therefore, the total passive force available to resist sliding is at least 215' x 4.32 k/LF = 929 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 6 ft height of the mat + 1-ft deep key = 0.005 x 6 ft x 12 in./ft = 0.36 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, and they indicate that the factors of safety are acceptable for all load combinations examined. The lowest factor of safety was 1.10, which applies for Cases IIIB and IVB, where 100% of the dynamic earthquake forces act in the east-west direction and 40% act in the other two directions. 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.

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 Load Cases IIIA and IIIC and they illustrate that it is ~1.1 for Load Case IIIB. 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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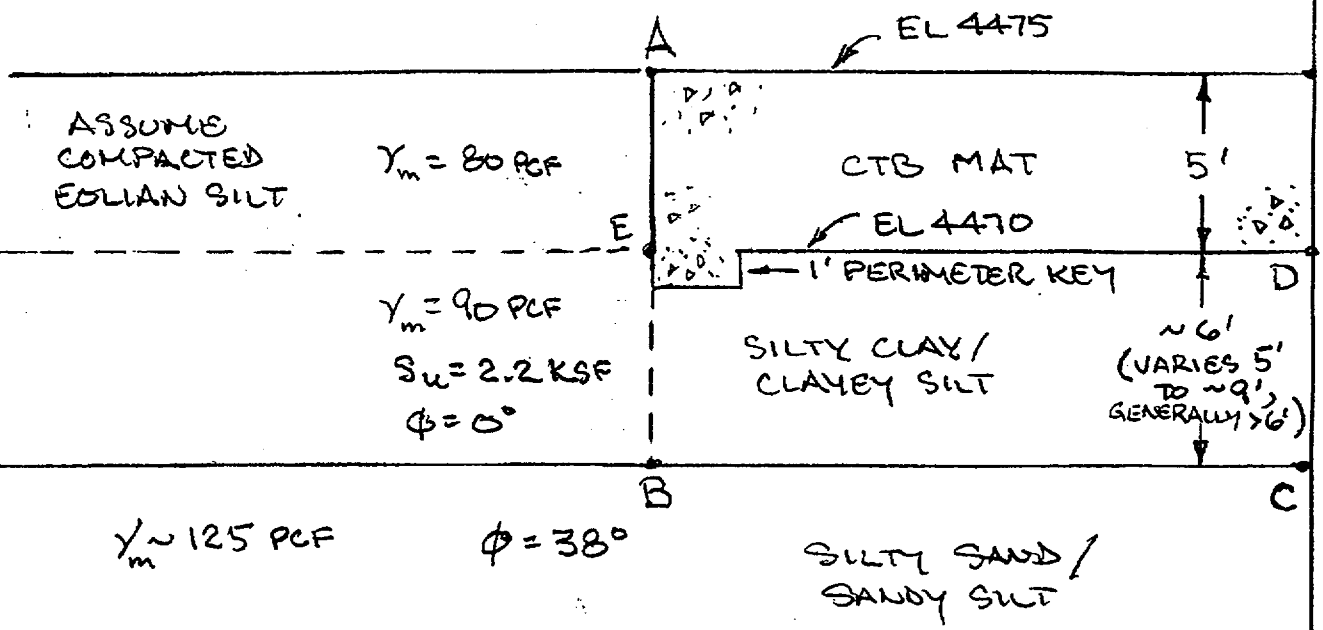
SLIDING ON DEEP PLANE AT TOP OF SILTY SAND/ SANDY SILT LAYER



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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CALCULATION SHEET

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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 $\frac{1}{2} \times (0.080 \frac{K}{FT^3}) \times (6')^2 \times 3.0 = 4.32$ K/LF FOR COMPACTED EOLIAN SILT ADJACENT TO 5' MAT + 1' KEY

$+ \frac{1}{2} \gamma H^2 K_p + q_s H K_p + 2cH\sqrt{K_p}$ FOR 5' BLOCK OF SILTY CLAY UNDERLYING THE COMPACTED SILT.

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

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

∴ TOTAL PASSIVE RESISTANCE AVAILABLE ALONG AB

$= 4.32 + 25.52 = 29.84$ 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 15.

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

AREA BCDE $= 6' FT \times 215\ FT_{E-W} = 1290\ \frac{FT^2}{END}$

$\therefore\ \Delta T_{ENDS_{E-W}} = 2 \text{ ENDS} \times 1290 \frac{FT^2}{END} \times 2.2 \frac{K}{FT^2} = 5{,}676\ K_{E-W}$

$\Delta T_{END_{N-S}} = 2 \text{ ENDS} \times 6' \times \underset{L_{N-S}}{272'} \times 2.2 \frac{K}{FT^2} = 7{,}181\ K_{N-S}$

(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.

$$\Delta N_{CLAY} = \underset{\Delta H}{6'} \times \underset{\gamma}{0.090 \frac{K}{FT^3}} \times \underset{B}{165'} \times \underset{L}{265'} = 23{,}612\ K$$

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

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SLIDING ON DEEP PLANE

	N-S	VERT	E-W
CASE IIIA:	40% IN X	−100% IN Y	40% IN Z
FROM TABLE 1	$0.4 \times 62{,}040^k$	−57,139 K	$\underbrace{0.4 \times 67{,}572}_{L = V_{E\text{-}W}} = 27{,}029\text{k}$

$$\therefore N = \underset{\text{CTB DL}}{72{,}988} - \underset{F_{VD}\text{ or }E_{Qv}}{57{,}139\text{k}} + \underset{\Delta N_{CLAY}}{23{,}612^k} = 39{,}460\text{ K}$$

$$N \tan\phi = 39{,}460\text{k} \tan 38^\circ = 30{,}830\text{ K}$$

$$\therefore FS_{SLIDING_{N\text{-}S}} = \frac{29.84\,\text{K/LF} \times \underset{B_{E\text{-}W}}{215'} + 7{,}181\text{K} + 30{,}830\text{K}}{0.4 \times 62{,}040\text{K}} = 1.78$$

$$FS_{SLIDING_{E\text{-}W}} = \frac{29.84\,\frac{K}{LF} \times \overset{L}{272'} + 5{,}676\text{K} + 30{,}830\text{K}}{0.4 \times 67{,}572\text{K}} = 1.65 > 1.1$$

$\therefore$ OK

	N-S	VERT	E-W
CASE IIIB	40% IN X	−40% IN Y	100% IN Z
FROM TABLE 1	$0.4 \times 62{,}040^k$	$-0.4 \times 57{,}139^k$	$67{,}572^K$ $L = V_{E\text{-}W}$

$$\therefore N = \underset{\text{CTB DL}}{72{,}988\text{K}} - \underset{22{,}856}{\underset{F_{VD}}{0.4 \times 57{,}139\text{K}}} + \underset{\Delta N_{CLAY}}{23{,}612^k} = 73{,}744\text{ K}$$

$$\Rightarrow T = \Big(\underset{P_p}{4.32\,\frac{K}{L.F.}} + 25.52\,\frac{K}{L.F.}\Big) \times \overset{L_{N\text{-}S}}{272'} + 5{,}676\text{ K}_{E\text{-}W}$$

8116 K

$$+ \underset{57{,}615}{\overset{N}{73{,}744\text{K}} \tan \overset{\phi}{38^\circ}} = 71{,}407\text{ K}$$

$$FS = \frac{\text{RESISTING}}{\text{DRIVING}} = \frac{71{,}407\text{ K}}{67{,}572\text{ K}} = 1.06 \quad \sim 1.1$$

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SLIDING ON DEEP PLANE

	N-S	VERT	E-W
<u>CASE III C</u>	100% IN X	−40% IN Y	40% IN Z
FROM TABLE 1	62,040 K	−0.4 × 57,139 K	0.4 × 67,572

$$\therefore\ N = \underset{\text{CTB DL}}{72{,}988} - \underset{\underset{22{,}856}{F_{VD}}}{0.4 \times 57{,}139\,K} + \underset{\Delta N_{CLAY}}{23{,}612} = 73{,}744\,K$$

$$T_{N\text{-}S} = 29.84\,\frac{K}{LF} \times \underset{B}{215'} + \underset{\Delta T_{N\text{-}S}}{7{,}181\,K} + 73{,}744 \tan 38^\circ = 71{,}212\,K$$

$$FS_{SLIDING} = \frac{T}{V_{N\text{-}S}} = \frac{71{,}212\,K}{62{,}040\,K} = 1.15 \quad > 1.1 \ \therefore OK$$

THE FACTOR OF SAFETY AGAINST SLIDING ON A DEEP PLANE OF COHESIONLESS SOIL IS > 1.1 FOR LOAD CASES III A & III C, AND IT IS ESSENTIALLY 1.1 FOR LOAD CASE III B. THIS CASE, HOWEVER, IS LESS CRITICAL THAN THE CASE PRESENTED IN THE FOLLOWING SECTION, WHICH DEMONSTRATES THAT EVEN IF THE COHESIONLESS SOIL WAS LOCATED DIRECTLY BENEATH THE CTB MAT, THE ESTIMATED HORIZONTAL DISPLACEMENTS OF THE FOUNDATION DUE LOADS FROM THE DESIGN BASIS GROUND MOTION ARE ≤ 1.2 INCHES.

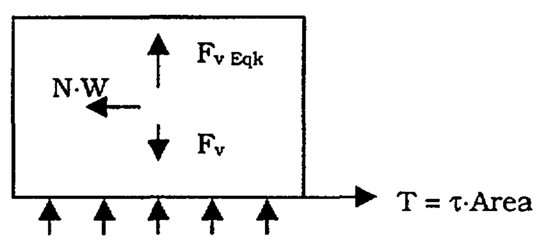
SLIDING STABILITY OF THE CANISTER TRANSFER BUILDING ON COHESIONLESS SOILS (CONT'D)

An additional analysis of sliding on cohesionless soils was performed to define the upper bound of potential movement that might occur due to the earthquake if the mat was founded directly on cohesionless soils. In this analysis it was postulated that the cohesionless soils extend above the depth of about 10 ft and the structure is founded directly on the cohesionless materials. These analyses conservatively assumed that ϕ = 35° and c = 0 for these soils.

The higher value of ϕ used here, compared to that used in the cask storage pad sliding analysis, is based on the fact that the cohesionless soils underlying the Canister Transfer Building area are sandier than those in the pad emplacement area. Further, this higher value is justified by the results of the cone penetration testing, which indicate that the average and median ϕ range from 40° to 44° for the cohesionless soils underlying the Canister Transfer Building. The high values reported in the CPT results likely are the manifestation of cementation that was observed in many of the specimens obtained in split-barrel sampling. Possible cementation of these soils is also ignored in this analysis, adding to the conservatism.

Because of the magnitude of the dynamic forces resulting from the soil-structure interaction analyses, the factor of safety against sliding of this building would be less than 1 if it were founded directly on cohesionless soils. For this case, the displacements the building may experience were calculated using the method proposed by Newmark (1965) for estimating displacements of dams and embankments during earthquakes.

NEWMARK'S METHOD OF ESTIMATING DISPLACEMENTS DUE TO EARTHQUAKES



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.

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

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

σ_n = Normal Stress

ϕ = Friction angle of sand layer

σ_n = Net Vertical Force/Area

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

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

$$N \cdot W = T$$

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

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

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

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 6 , 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.

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

ESTIMATION OF HORIZONTAL DISPLACEMENT USING NEWMARK'S METHOD

1. Maximum Ground Motions

The maximum ground accelerations and velocities at the Canister Transfer Building are based on Calculation 05996.02-SC-5, Rev. 1, p. 37 (SWEC, 1999b), which indicates:

	North-South	**Vertical**	**East-West**
Acceleration	0.805g	0.720g	0.769g
Velocity	21.7 in./sec	Not Required	19.8 in./sec

2. Load Combinations

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.

3. Ground Motions for Analysis

Case	North-South		Vertical	East-West	
	Accel (g)	**Velocity (in./sec)**	**Accel (g)**	**Accel (g)**	**Velocity (in./sec)**
IIIA	0.322	8.68	0.720	0.308	7.92
IIIB	0.322	8.68	0.288	0.769	19.8
IIIC	0.805	21.7	0.288	0.308	7.92

LOAD CASE IIIA: 40% N-S DIRECTION, -100% VERTICAL DIRECTION, 40% E-W DIRECTION.

Static Vertical Force, $F_v = W = 72{,}988$ kips (Calculation 05996.02-SC-5, Rev 1 (SWEC, 1999b), p37)

Earthquake Vertical Force, $F_{v\,Eqk} = 57{,}139$ kips (Calculation 05996.02-SC-5, Rev 1, p37)

$$\phi = 35^\circ$$

$$N = [(72{,}988 - 57{,}139)\tan 35^\circ]\,/72{,}988$$

$$N = 0.152$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\underbrace{0.322^2}_{40\%\ N\text{-}S} + \underbrace{0.308^2}_{40\%\ E\text{-}W})}\,g$

$$A = 0.446g$$

Resultant velocity in horizontal direction, $V = \sqrt{(\underbrace{8.68^2}_{40\%\ N\text{-}S} + \underbrace{7.92^2}_{40\%\ E\text{-}W})}$

$$V = 11.75\ \text{in./sec}$$

$$\Rightarrow \frac{N}{A} = \frac{0.152}{0.446} = 0.34$$

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

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

where g is in units of inches/sec².

$$\Rightarrow \quad u_m = \left(\frac{(11.75\,\text{in./sec})^2 \cdot (1 - 0.34)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.152} \right) = 0.8"$$

As shown in Figure 6, 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 where there is symmetrical resistance to sliding. Within the range of values of N/A between 0.15 to 0.5, the following expression gives an upper bound for all data:

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

Substituting the relevant parameters,

$$\Rightarrow \quad u_m = \left(\frac{(11.75\,\text{in./sec})^2}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.152} \right) = 1.2"$$

Therefore, the maximum relative displacement ranges from 0.8" to 1.2" for Load Case IIIA.

LOAD CASE IIIB: 40% N-S DIRECTION, -40% VERTICAL DIRECTION, 100% E-W DIRECTION.

Static Vertical Force, F_v = W = 72,988 kips (Calculation 05996.02-SC-5, Rev 1 (SWEC, 1999b), p37)

Earthquake Vertical Force, $F_{v\,Eqk}$ = 57,139 kips x 0.40 = 22,856 kips acting upward.

$$\phi = 35°$$

$$N = [(72{,}988 - 22{,}856)\tan 35°] / 72{,}988$$

$$N = 0.48$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\underset{\text{40\% N-S}}{0.322^2} + \underset{\text{100\% E-W}}{0.769^2})}\,g$

$$A = 0.834g$$

Resultant velocity in horizontal direction, $V = \sqrt{(\underset{\text{40\% N-S}}{8.68^2} + \underset{\text{100\% E-W}}{19.8^2})}$

$$V = 21.6\ \text{in./sec}$$

$$\Rightarrow \quad \frac{N}{A} = \frac{0.48}{0.834} = 0.576$$

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

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

where g is in units of inches/sec².

$$\Rightarrow \quad u_m = \left(\frac{(21.6\,\text{in./sec})^2 \cdot (1 - 0.576)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.48} \right) = 0.5"$$

As shown in Figure 6, 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 where there is symmetrical resistance to sliding. For Case IIIB, N/A > 0.5; therefore, u_m = 0.5" for this case.

LOAD CASE IIIC: 100% N-S DIRECTION, -40% VERTICAL DIRECTION, 40% E-W DIRECTION.

Static Vertical Force, F_v = W = 72,988 kips (Calculation 05996.02-SC-5, Rev 1 (SWEC, 1999b), p37)

Earthquake Vertical Force, $F_{v\,Eqk}$ = 57,139 kips x 0.40 = 22,856 kips acting upward.

$$\phi = 35°$$

$$N = [(72{,}988 - 22{,}856) \tan 35°] / 72{,}988$$

$$N = 0.48$$

Resultant acceleration in horizontal direction, $A = \sqrt{(\overset{100\%\ \text{N-S}}{0.805^2} + \overset{40\%\ \text{E-W}}{0.308^2})}\,g$

$$A = 0.862g$$

Resultant velocity in horizontal direction, $V = \sqrt{(\overset{100\%\ \text{N-S}}{21.7^2} + \overset{40\%\ \text{E-W}}{7.92^2})}$

$$V = 23.1 \text{ in./sec}$$

$$\Rightarrow \quad \frac{N}{A} = \frac{0.48}{0.862} = 0.558$$

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

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

where g is in units of inches/sec².

$$\Rightarrow \quad u_m = \left(\frac{(23.1\,\text{in./sec})^2 \cdot (1 - 0.558)}{2 \cdot 386.4\,\text{in./sec}^2 \cdot 0.48} \right) = 0.6"$$

As shown in Figure 6, 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 where there is symmetrical resistance to sliding. For Case IIIC, N/A > 0.5; therefore, u_m = 0.6" for this case.

Summary of Horizontal Displacement Calculated Using Newmark's Method

The following table presents a summary of the Newmark's analysis of sliding of the Canister Transfer Building, assuming it is founded directly on cohesionless soils.

Load Combination				**Displacement**
Case IIIA	40% N-S	-100% Vertical	40% E-W	0.8 to 1.2 inches
Case IIIB	40% N-S	-40% Vertical	100% E-W	0.5 inches
Case IIIC	100% N-S	-40% Vertical	40% E-W	0.6 inches

These analyses indicate that there is an adequate factor of safety against sliding along the surface of the soils underlying the building that may be cohesionless within the depth zone of about 10 to 20 ft, especially near the southern portion of the building. The analysis that postulated that these cohesionless soils exist higher in the profile, such that the building was constructed directly on them, includes several conservative assumptions. Even with this high level of conservatism, the estimated relative displacement of the building ranged from 0.5 inches to 1.2 inches.

Motions of this magnitude, occurring at the depth of the silty sand/sandy silt layer, would likely not even be evident at the ground surface. For the building to slide, a surface of sliding must be established between the horizontal sliding surface in the silty sand/sandy silt layer and through the overlying clayey layer. In the simplified model used to estimate these displacements, the contribution of this surface of sliding through the overlying clayey layer to the dynamic resistance to sliding motion is ignored, as is the passive resistance that would act on the embedded portion of the building foundation and the block of soil that is postulated to be moving with it. It is likely, that should such slippage occur within the cohesionless soils underlying the building, it would minimize the level of the accelerations that would be transmitted through the soil and into the structure. In this manner, the cohesionless soils would act as a built-in base-shear isolation system. Any decrease in these accelerations as a result of this would increase the factor of safety against sliding, which would decrease the estimated displacements as well. Further, since there are no Important to Safety systems that would be severed or otherwise impacted by movements of this small amount as a result of the earthquake, such movements do not adversely affect the performance of the Canister Transfer Building.

ALLOWABLE BEARING CAPACITY

Bearing capacity calculations are performed using the method for determining general bearing capacity failure, as presented in Winterkorn and Fang (1975. Local bearing capacity (punching shear) failure is ruled out due to the large size of the mat, 165' x 265'.

The general bearing capacity equation is a modification of Terzaghi's bearing capacity equation, which was developed for strip footings and which indicates that $q_{ult} = cN_c+qN_q+1/2\ \gamma BN_\gamma$. For this relationship, the ultimate bearing capacity of soil consists of three components: 1) cohesion, 2) surcharge, and 3) friction, which are represented by bearing capacity factors N_c, N_q, and N_γ. Terzaghi's bearing capacity equation has been enhanced by various investigators to incorporate shape, depth, and load inclination factors for different foundation geometries and loads as follows:

$$q_{ult} = c\, N_c\, s_c\, d_c\, i_c + q\, N_q\, s_q\, d_q\, i_q + \tfrac{1}{2}\, \gamma B\, N_\gamma\, s_\gamma\, d_\gamma\, i_\gamma$$

where

q_{ult} = ultimate bearing capacity

c = cohesion or undrained strength

q = effective surcharge at bottom of foundation, = γD_f

γ = unit weight of soil

B = foundation width

s_c, s_q, s_γ = shape factors, which are a function of foundation width to length

d_c, d_q, d_γ = depth factors, which account for embedment effects

i_c, i_q, i_γ = load inclination factors

N_c, N_q, N_γ = bearing capacity factors, which are a function of ϕ.

γ in the third term is the unit weight of soil below the foundation, whereas the unit weight of the soil above the bottom of the footing is used in determining q in the second term.

BEARING CAPACITY FACTORS

Bearing capacity factors computed based on relationships proposed by Vesic (1973), which are presented in Chapter 3 of Winterkorn and Fang (1975).

$$N_q = e^{\pi \tan\varphi} \tan^2\left(45 + \frac{\phi}{2}\right)$$

$$N_c = (N_q - 1) \ \cot\phi \text{, but} = 5.14 \text{ for } \phi = 0.$$

$$N_\gamma = 2 \ (N_q + 1) \ \tan\phi$$

SHAPE FACTORS

$$s_c = 1 + \frac{B}{L} \cdot \frac{N_q}{N_c}$$

$$s_q = 1 + \frac{B}{L} \tan\phi$$

$$s_\gamma = 1 - 0.4 \cdot \frac{B}{L}$$

DEPTH FACTORS

For $\frac{D_f}{B} \leq 1$:

$$d_c = d_q - \frac{(1 - d_q)}{N_q \cdot \tan\phi} \text{ for } \phi > 0 \text{ and } d_c = 1 + 0.4\left(\frac{D_f}{B}\right) \text{ for } \phi = 0.$$

$$d_q = 1 + 2\tan\phi \cdot (1 - \sin\phi)^2 \cdot \left(\frac{D_f}{B}\right)$$

$$d_\gamma = 1$$

INCLINATION FACTORS

$$i_q = \left(1 - \frac{F_H}{F_V + B' L' c \cot\phi}\right)^m$$

$$i_c = i_q - \frac{(1 - i_q)}{N_c \cdot \tan\phi} \text{ for } \phi > 0 \text{ and } i_c = 1 - \left(\frac{m F_H}{B' L' c N_c}\right) \text{ for } \phi = 0$$

$$i_\gamma = \left(1 - \frac{F_H}{F_V + B' L' c \cot\phi}\right)^{m+1}$$

where F_H and F_V are the total horizontal and vertical forces acting on the footing.

Static Bearing Capacity of the Canister Transfer Building

The following pages present the details of the bearing capacity analyses for the static load cases. These cases are identified as follows:

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

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

Table 2.6-9 presents the results of the bearing capacity analyses for these static load cases. The minimum factor of safety required for static load cases is 3.

As indicated in this table, the gross allowable bearing pressure for the Canister Transfer Building to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 6 ksf. However, loading the foundation to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume $\phi = 0°$ and $c = 3.18$ ksf, the average undrained strength for the soils in the upper layer at the site, to model the end of construction. Using the estimated effective-stress strength of $\phi = 30°$ and $c = 0$ results in higher allowable bearing pressures. As shown in Table 2.6-9, the gross allowable bearing capacity of the Canister Transfer Building for static loads for these soil strengths is 45 ksf.

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ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Static Analysis: **Case IA**

Soil Properties:

s_u =	**3,180**	Average undrained strength (psf) in upper ~30' layer	
ϕ =	**0.0**	Friction Angle (degrees)	
γ =	**90**	Unit weight of soil (pcf)	
γ_{surch} =	**80**	Unit weight of surcharge (pcf)	

Foundation Properties:

B' =	**165.0**	Footing Width - ft (E-W)	L' = **265.0** Length - ft (N-S)
D_f =	**5**	Depth of Footing (ft)	
β =	**0.0**	Angle of load inclination from vertical (degrees)	
FS =	**3**	Factor of Safety required for $q_{allowable}$.	
F_V =	**72,988** k	EQ_V =	**0** k
$EQ_{H\ E\text{-}W}$ =	**0** k	+ $EQ_{H\ N\text{-}S}$ =	**0** k = **0** k for F_H

$q_{ult} = c\, N_c\, s_c\, d_c\, i_c + \gamma_{surch}\, D_f\, N_q\, s_q\, d_q\, i_q + 1/2\, \gamma\, B\, N_\gamma\, s_\gamma\, d_\gamma\, i_\gamma$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

	$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	= 5.14	Eq 3.6 & Table 3.2
	$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2)$	= 1.00	Eq 3.6
	$N_\gamma = 2(N_q + 1)\tan(\phi)$	= 0.00	Eq 3.8
	$s_c = 1 + (B/L)(N_q/N_c)$	= 1.12	Table 3.2
	$s_q = 1 + (B/L)\tan\phi$	= 1.00	"
	$s_\gamma = 1 - 0.4(B/L)$	= 0.75	"
For $D_f/B \leq 1$:	$d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	= 1.00	Eq 3.26
	$d_\gamma = 1$	= 1.00	"
For $\phi > 0$:	$d_c = d_q - (1-d_q)/(N_q \tan\phi)$	= N/A	
For $\phi = 0$:	$d_c = 1 + 0.4(D_f/B)$	= 1.01	Eq 3.27

No inclined loads; therefore, $i_c = i_q = i_\gamma = 1.0$.

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**18,947**	**psf =**	**18,547**	+	400	+	0
q_{all} =	**6,310**	**psf = q_{ult} / FS**					
q_{actual} =	**1,669**	**psf = $(F_v + EQ_v) / (B' \times L')$**					
FS_{actual} =	**11.35**	**= q_{ult} / q_{actual}**			> 3	**Hence OK**	

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ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

Static Analysis: **Case IB**

Soil Properties:

s_u = **0** Cohesion (psf)

ϕ = **30.0** Friction Angle (degrees)

γ = **90** Unit weight of soil (pcf)

γ_{surch} = **80** Unit weight of surcharge (pcf)

Foundation Properties:

B' = **165.0** Footing Width - ft (E-W) L' = **265.0** Length - ft (N-S)

D_f = **5** Depth of Footing (ft)

β = **0.0** Angle of load inclination from vertical (degrees)

FS = **3** Factor of Safety required for $q_{allowable}$.

F_V = **72,988** k EQ_V = **0** k

$EQ_{H\,E\text{-}W}$ = **0** k + $EQ_{H\,N\text{-}S}$ = **0** k = **0** k for F_H

$$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	30.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi\tan\phi}\tan^2(\pi/4 + \phi/2)$	=	18.40	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	22.40	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.38	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.36	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.75	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.01	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	1.01	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	N/A	Eq 3.27

No inclined loads; therefore, $i_c = i_q = i_\gamma = 1.0$.

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**135,005**	**psf =**	**0**	**+**	**10,094**	**+**	**124,911**

q_{all} = 45,000 psf = q_{ult} / FS

q_{actual} = 1,669 psf = $(F_V + EQ_V) / (B' \times L')$

FS_{actual} = 80.88 = q_{ult} / q_{actual} > **3** **Hence OK**

DYNAMIC BEARING CAPACITY OF THE CANISTER TRANSFER BUILDING

The following pages present the details of the bearing capacity analyses for the dynamic load cases. These analyses use the dynamic loads for the building that were developed in Calculation 05996.02-SC-5, (SWEC, 1999b). The development of these dynamic loads is described in Section 4.7.1.5.3 of the SAR. As in the structural analyses discussed in SAR Section 4.7.1.5.3., the seismic loads used in these analyses were combined using 100% of the enveloped zero period accelerations (ZPA) in one direction with 40% of the enveloped ZPA in each of the other two directions. The resulting dynamic loading cases are identified as follows:

Case II	100% N-S direction,	0% Vertical direction,	100% E-W direction.
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.
Case IVA	40% N-S direction,	100% Vertical direction,	40% E-W direction.
Case IVB	40% N-S direction,	40% Vertical direction,	100% E-W direction.
Case IVC	100% N-S direction,	40% Vertical direction,	40% E-W direction.

Table 2.6-10 presents the results of the bearing capacity analyses for these cases, which include static loads plus dynamic loads due to the earthquake. Because the *in situ* fine-grained soils are not expected to fully drain during the rapid cycling of load during the earthquake, these cases are analyzed using the average undrained strength applicable for the soils within the upper layer ($\phi = 0°$ and c = 3.18 ksf). As indicated above, for these cases including dynamic loads from the design basis ground motion, the minimum acceptable factor of safety is 1.1.

Table 2.6-10 indicates the minimum factor of safety against a dynamic bearing capacity failure was obtained for Load Case IIIB, the load combination of full static with 40% of the earthquake loading acting in the N-S direction, 40% acting in the vertical direction, tending to unload the mat, and 100% acting in the E-W horizontal direction. This load case resulted in an actual soil bearing pressure of 3.31 kips per square foot (ksf), compared with an ultimate bearing capacity of ~9 ksf. The resulting factor of safety against a bearing capacity failure for this load case is ~3, which is much greater than 1.1, the minimum allowable factor of safety for seismic loading cases. In these analyses, no credit was taken for the fact that strength of cohesive soil increases as the rate of loading increases. Therefore, the Canister Transfer Building has an adequate factor of safety against a dynamic bearing capacity failure.

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ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case II

$\boxed{\textbf{100 \% in X, 0 \% in Y, 100 \% in Z}}$

Soil Properties:

$s_u =$ **3,180** Average undrained strength (psf) in upper ~30' layer

$\phi =$ **0.0** Friction Angle (degrees)

$\gamma =$ **90** Unit weight of soil (pcf)

$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **96.1** Footing Width - ft (E-W) $L' =$ **211.3** Length - ft (N-S)

$D_f =$ **5** Depth of Footing (ft)

$\beta =$ **42.8** Angle of load inclination from vertical (degrees)

$FS =$ **1.1** Factor of Safety required for $q_{allowable}$.

$F_V =$ **72,988** k $EQ_V =$ **0** k

$EQ_{H\ E\text{-}W} =$ **67,572** k + $EQ_{H\ N\text{-}S} =$ **62,040** k = **91,733** k for F_H

$$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.82	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.02	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.83 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.51	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.58	Eq 3.16a
$i_q = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	10,984	psf =	10,584	+	400	+	0

$q_{all} =$ **9,980** psf = q_{ult} / FS

$q_{actual} =$ **3,594** psf = $(F_v + EQ_v) / (B' \times L')$

$FS_{actual} =$ **3.06** = q_{ult} / q_{actual} **> 1.1 Hence OK**

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ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IIIA **40 % in X, -100 % in Y, 40 % in Z**

Soil Properties:	s_u =	**3,180**	Average undrained strength (psf) in upper ~30' layer
	ϕ =	**0.0**	Friction Angle (degrees)
	γ =	**90**	Unit weight of soil (pcf)
	γ_{surch} =	**80**	Unit weight of surcharge (pcf)
Foundation Properties:	B' =	**38.2**	Footing Width - ft (E-W) L' = **166.0** Length - ft (N-S)
	D_f =	**5**	Depth of Footing (ft)
	β =	**59.6**	Angle of load inclination from vertical (degrees)
	FS =	**1.1**	Factor of Safety required for $q_{allowable}$.
	F_V =	**72,988** k	EQ_V = **-57,139** k
	$EQ_{H\ E\text{-}W}$ =	**27,029** k	+ $EQ_{H\ N\text{-}S}$ = **24,816** k = **36,693** k for F_H

$q_{ult} = c\ N_c\ s_c\ d_c\ i_c + \gamma_{surch}\ D_f\ N_q\ s_q\ d_q\ i_q + 1/2\ \gamma\ B\ N_\gamma\ s_\gamma\ d_\gamma\ i_\gamma$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

	$N_c = (N_q - 1)\ \cot(\phi)$, but = 5.14 for $\phi = 0$	= 5.14	Eq 3.6 & Table 3.2
	$N_q = e^{\pi \tan\phi}\ \tan^2(\pi/4 + \phi/2)$	= 1.00	Eq 3.6
	$N_\gamma = 2\ (N_q + 1)\ \tan(\phi)$	= 0.00	Eq 3.8
	$s_c = 1 + (B/L)(N_q/N_c)$	= 1.04	Table 3.2
	$s_q = 1 + (B/L) \tan\phi$	= 1.00	"
	$s_\gamma = 1 - 0.4\ (B/L)$	= 1.00	"
For $D_f/B \le 1$:	$d_q = 1 + 2 \tan\phi\ (1 - \sin\phi)^2\ D_f/B$	= 1.00	Eq 3.26
	$d_\gamma = 1$	= 1.00	"
For $\phi > 0$:	$d_c = d_q - (1-d_q) / (N_q \tan\phi)$	= N/A	
For $\phi = 0$:	$d_c = 1 + 0.4\ (D_f/B)$	= 1.05	Eq 3.27
	$m_B = (2 + B/L) / (1 + B/L)$	= 1.62	Eq 3.18a
	$m_L = (2 + L/B) / (1 + L/B)$	= 1.38	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$:	$\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W} / EQ_{H\ N\text{-}S})$	= 0.83 rad	
	$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	= 1.51	Eq 3.18c
For $\phi = 0$:	$i_c = 1 - (m\ F_H / B'\ L'\ c\ N_c)$	= 0.46	Eq 3.16a
	$i_q = \{ 1 - F_H / [(F_v + EQ_v) + B'\ L'\ c \cot\phi] \}^m$	= 1.00	Eq 3.14a
	$i_\gamma = \{ 1 - F_H / [(F_v + EQ_v) + B'\ L'\ c \cot\phi] \}^{m+1}$	= 0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**8,753**	**psf =**	**8,353**	+	**400**	+	**0**
q_{all} =	**7,950**	**psf = q_{ult} / FS**					
q_{actual} =	**2,503**	**psf = $(F_v + EQ_v) / (B' \times L')$**					
FS_{actual} =	**3.50**	**= q_{ult} / q_{actual}**					

> 1.1 Hence OK

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ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IIIB **40 % in X, -40 % in Y, 100 % in Z**

Soil Properties:
s_u = **3,180** Average undrained strength (psf) in upper ~30' layer
ϕ = **0.0** Friction Angle (degrees)
γ = **90** Unit weight of soil (pcf)
γ_{surch} = **80** Unit weight of surcharge (pcf)

Foundation Properties:
B' = **64.7** Footing Width - ft (E-W) L' = **233.7** Length - ft (N-S)
D_f = **5** Depth of Footing (ft)
β = **53.4** Angle of load inclination from vertical (degrees)
FS = **1.1** Factor of Safety required for $q_{allowable}$.
F_V = **72,988** k EQ_V = **-22,856** k
$EQ_{H\ E\text{-}W}$ = **67,572** k + $EQ_{H\ N\text{-}S}$ = **24,816** k = **71,985** k for F_H

$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.05	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.89	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q\tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.03	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	1.22 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.59	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.54	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**9,947**	**psf =**	**9,547**	+	**400**	+	**0**

q_{all} = 9,040 psf = q_{ult} / FS

q_{actual} = 3,313 psf = $(F_v + EQ_v)/(B' \times L')$

FS_{actual} = 3.00 = q_{ult}/q_{actual} **> 1.1 Hence OK**

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ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IIIC **100 % in X, -40 % in Y, 40 % in Z**

Soil Properties:

$s_u =$ **3,180** Average undrained strength (psf) in upper ~30' layer

$\phi =$ **0.0** Friction Angle (degrees)

$\gamma =$ **90** Unit weight of soil (pcf)

$\gamma_{surch} =$ **80** Unit weight of surcharge (pcf)

Foundation Properties:

$B' =$ **124.9** Footing Width - ft (E-W) $L' =$ **186.8** Length - ft (N-S)

$D_f =$ **5** Depth of Footing (ft)

$\beta =$ **28.3** Angle of load inclination from vertical (degrees)

$FS =$ **1.1** Factor of Safety required for $q_{allowable}$.

$F_V =$ **72,988** k $EQ_V =$ **-22,856** k

$EQ_{H\,E\text{-}W} =$ **27,029** k + $EQ_{H\,N\text{-}S} =$ **62,040** k = **67,672** k for F_H

$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.13	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4(B/L)$	=	0.73	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4(D_f/B)$	=	1.02	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W}/EQ_{H\,N\text{-}S})$	=	0.41 rad	
$m_n = m_L\cos^2\theta_n + m_B\sin^2\theta_n$	=	1.42	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.75	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	14,435	psf =	14,035	+	400	+	0

$q_{all} =$ **13,120** psf = q_{ult} / FS

$q_{actual} =$ **2,149** psf = $(F_V + EQ_V)/(B' \times L')$

$FS_{actual} =$ **6.72** = q_{ult}/q_{actual} > **1.1 Hence OK**

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05996.02	G(B)	13-3	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IVA — **40 % in X, 100 % in Y, 40 % in Z**

Soil Properties:

s_u = **3,180** Average undrained strength (psf) in upper ~30' layer

ϕ = **0.0** Friction Angle (degrees)

γ = **90** Unit weight of soil (pcf)

γ_{surch} = **80** Unit weight of surcharge (pcf)

Foundation Properties:

B' = **149.6** Footing Width - ft (E-W) L' = **252.9** Length - ft (N-S)

D_f = **5** Depth of Footing (ft)

β = **11.7** Angle of load inclination from vertical (degrees)

FS = **1.1** Factor of Safety required for $q_{allowable}$.

F_V = **72,988** k EQ_V = **57,139** k

$EQ_{H\,E\text{-}W}$ = **27,029** k + $EQ_{H\,N\text{-}S}$ = **24,816** k = **36,693** k for F_H

$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2 (N_q + 1) \tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.12	Table 3.2
$s_q = 1 + (B/L) \tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4 (B/L)$	=	0.76	"
For $D_f/B \le 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q) / (N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4 (D_f/B)$	=	1.01	Eq 3.27
$m_B = (2 + B/L) / (1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B) / (1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W} / EQ_{H\,N\text{-}S})$	=	0.83 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.51	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.91	Eq 3.16a
$i_q = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H / [(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**17,214**	**psf =**	**16,814**	+	**400**	+	**0**

q_{all} = 15,640 psf = q_{ult} / FS

q_{actual} = 3,440 psf = $(F_v + EQ_v) / (B' \times L')$

FS_{actual} = 5.00 = q_{ult} / q_{actual} > 1.1 Hence OK

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IVB | **40 % in X, 40 % in Y, 100 % in Z**

Soil Properties:	s_u =	**3,180**	Average undrained strength (psf) in upper ~30' layer
	ϕ =	**0.0**	Friction Angle (degrees)
	γ =	**90**	Unit weight of soil (pcf)
	γ_{surch} =	**80**	Unit weight of surcharge (pcf)
Foundation Properties:	B' =	**112.6**	Footing Width - ft (E-W) L' = **248.6** Length - ft (N-S)
	D_f =	**5**	Depth of Footing (ft)
	β =	**35.2**	Angle of load inclination from vertical (degrees)
	FS =	**1.1**	Factor of Safety required for $q_{allowable}$.
	F_V =	**72,988** k	EQ_V = **22,856** k
	$EQ_{H\,E\text{-}W}$ =	**67,572** k	+ $EQ_{H\,N\text{-}S}$ = **24,816** k = **71,985** k for F_H

$q_{ult} = c\, N_c\, s_c\, d_c\, i_c + \gamma_{surch}\, D_f\, N_q\, s_q\, d_q\, i_q + 1/2\, \gamma\, B\, N_\gamma\, s_\gamma\, d_\gamma\, i_\gamma$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi} \tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2 (N_q + 1) \tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.09	Table 3.2
$s_q = 1 + (B/L) \tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4 (B/L)$	=	0.82	"
For $D_f/B \le 1$: $d_q = 1 + 2 \tan\phi\, (1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q) / (N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4 (D_f/B)$	=	1.02	Eq 3.27
$m_B = (2 + B/L) / (1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B) / (1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\,N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\,E\text{-}W} / EQ_{H\,N\text{-}S})$	=	1.22	rad
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.59	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\, F_H / B'\, L'\, c\, N_c)$	=	0.75	Eq 3.16a
$i_q = \{ 1 - F_H / [(F_v + EQ_v) + B'\, L'\, c \cot\phi] \}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{ 1 - F_H / [(F_v + EQ_v) + B'\, L'\, c \cot\phi] \}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross q_{ult} =	**13,976**	**psf =**	**13,576**	+	**400**	+	**0**
q_{all} =	**12,700**	**psf = q_{ult} / FS**					
q_{actual} =	**3,425**	**psf = $(F_v + EQ_v) / (B' \times L')$**					
FS_{actual} =	**4.08**	**= q_{ult} / q_{actual}**			> **1.1**	**Hence OK**	

CALCULATION IDENTIFICATION NUMBER				PAGE 38
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05996.02	G(B)	13-3	N/A	

ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING

PSHA 2,000-Yr Earthquake: Case IVC **100 % in X, 40 % in Y, 40 % in Z**

Soil Properties:

$s_u =$	**3,180**	Average undrained strength (psf) in upper ~30' layer	
$\phi =$	**0.0**	Friction Angle (degrees)	
$\gamma =$	**90**	Unit weight of soil (pcf)	
$\gamma_{surch} =$	**80**	Unit weight of surcharge (pcf)	

Foundation Properties:

$B' =$	**144.0**	Footing Width - ft (E-W)	$L' =$ **224.1** Length - ft (N-S)
$D_f =$	**5**	Depth of Footing (ft)	
$\beta =$	**15.7**	Angle of load inclination from vertical (degrees)	
$FS =$	**1.1**	Factor of Safety required for $q_{allowable}$.	
$F_V =$	**72,988** k	$EQ_V =$	**22,856** k
$EQ_{H\ E\text{-}W} =$	**27,029** k	+ $EQ_{H\ N\text{-}S} =$	**62,040** k = **67,672** k for F_H

$q_{ult} = c\,N_c\,s_c\,d_c\,i_c + \gamma_{surch}\,D_f\,N_q\,s_q\,d_q\,i_q + 1/2\,\gamma\,B\,N_\gamma\,s_\gamma\,d_\gamma\,i_\gamma$

General Bearing Capacity Equation, based on Winterkorn & Fang (1975)

$N_c = (N_q - 1)\cot(\phi)$, but = 5.14 for $\phi = 0$	=	5.14	Eq 3.6 & Table 3.2
$N_q = e^{\pi \tan\phi}\tan^2(\pi/4 + \phi/2)$	=	1.00	Eq 3.6
$N_\gamma = 2(N_q + 1)\tan(\phi)$	=	0.00	Eq 3.8
$s_c = 1 + (B/L)(N_q/N_c)$	=	1.13	Table 3.2
$s_q = 1 + (B/L)\tan\phi$	=	1.00	"
$s_\gamma = 1 - 0.4\,(B/L)$	=	0.74	"
For $D_f/B \leq 1$: $d_q = 1 + 2\tan\phi\,(1 - \sin\phi)^2 D_f/B$	=	1.00	Eq 3.26
$d_\gamma = 1$	=	1.00	"
For $\phi > 0$: $d_c = d_q - (1-d_q)/(N_q \tan\phi)$	=	N/A	
For $\phi = 0$: $d_c = 1 + 0.4\,(D_f/B)$	=	1.01	Eq 3.27
$m_B = (2 + B/L)/(1 + B/L)$	=	1.62	Eq 3.18a
$m_L = (2 + L/B)/(1 + L/B)$	=	1.38	Eq 3.18b
If $EQ_{H\ N\text{-}S} > 0$: $\theta_n = \tan^{-1}(EQ_{H\ E\text{-}W}/EQ_{H\ N\text{-}S})$	=	0.41 rad	
$m_n = m_L \cos^2\theta_n + m_B \sin^2\theta_n$	=	1.42	Eq 3.18c
For $\phi = 0$: $i_c = 1 - (m\,F_H / B'\,L'\,c\,N_c)$	=	0.82	Eq 3.16a
$i_q = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^m$	=	1.00	Eq 3.14a
$i_\gamma = \{1 - F_H/[(F_v + EQ_v) + B'\,L'\,c\cot\phi]\}^{m+1}$	=	0.00	Eq 3.17a

			N_c term		N_q term		N_γ term
Gross $q_{ult} =$	15,646	psf =	15,246	+	400	+	0
$q_{all} =$	14,220	psf = q_{ult} / FS					
$q_{actual} =$	2,970	psf = $(F_v + EQ_v)/(B' \times L')$					
$FS_{actual} =$	5.27	= q_{ult}/q_{actual}			> 1.1 Hence OK		

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CONCLUSIONS

Overturning Stability of the Canister Transfer Building

The overturning stability of the Canister Transfer Building is analyzed on Pages 8 & 9 using the dynamic loads for the building due to the PSHA 2,000-yr return period earthquake. These loads, listed in Table 1 (SAR Table 2.6-11), were developed based on the dynamic analysis performed in Calculation 05996.02-SC-5 (SWEC, 1999b) and are described in SAR Section 4.7.1.5.3. This calculation demonstrates that the factor of safety against overturning of the Canister transfer Building is > 1.1; therefore, the Canister Transfer Building has an adequate factor of safety against overturning due to dynamic loadings from the design basis ground motion.

Sliding Sliding Stability of the Canister Transfer Building

The Canister Transfer Building (CTB) will be founded on clayey soils. The sliding stability of the CTB was evaluated using the loads developed in Calculation 05996.02-SC-5 (SWEC, 1999b). The static strength of the clayey soils at the bottom of the CTB mat was based on the average of two sets of direct shear tests performed on samples of soils obtained from beneath the Canister Transfer Building at the elevation proposed for founding the mat.

The results of the sliding stability analysis are presented in Table 2 of this calculation, and they indicate that for all load combinations examined, the factors of safety were acceptable. The lowest factor of safety was 1.10, which applies for Cases IIIB and IVB, where 100% of the dynamic earthquake forces act in the east-west direction and 40% act in the other two directions. These results assume that only one-half of the passive pressures are available 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.

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 were performed to 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.

These analyses included the passive resistance acting on a plane extending from grade down to the top of the cohesionless layer, 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 was included in the normal force used to calculate the frictional resistance acting along the top of the cohesionless layer. The factor of safety against sliding along the top of this layer was found to be ≥ 1.1 for all of the dynamic load cases; therefore, there is an adequate factor of safety against sliding along the surface of the cohesionless soils underlying the Canister Transfer Building.

Additional analyses of sliding on cohesionless soils, based on Newmark's method for estimating displacements of dams and embankment due to earthquakes, were performed to define the upper bound of potential movement that might occur due to the earthquake if the mat was founded directly on cohesionless soils. In these analyses it was postulated that the cohesionless soils extend above the depth of about 10 ft and the structure is founded directly on the cohesionless materials. Several conservative assumptions were made in these analyses, and even with this high level of conservatism, the estimated relative displacement of the building ranged from 0.5 inches to 1.2 inches.

Motions of this magnitude, occurring at the depth of the silty sand/sandy silt layer, would likely not even be evident at the ground surface. For the building to slide, a surface of sliding must be established between the horizontal sliding surface in the silty sand/sandy silt layer and through the overlying clayey layer. In the simplified model used to estimate these displacements, the contribution of this surface of sliding through the overlying clayey layer to the dynamic resistance to sliding motion is ignored, as is the passive resistance that would act on the embedded portion of the building foundation and the block of soil that is postulated to be moving with it. It is likely, moreover, that should such slippage occur within the cohesionless soils underlying the building, it would minimize the level of the accelerations that would be transmitted through the soil and into the structure. In this manner, these cohesionless soils would act as a built-in base-shear isolation system. Any decrease in these accelerations as a result of this would increase the factor of safety against sliding, which would decrease the estimated displacements as well. Further, since there are no important-to-safety systems that would be severed or otherwise impacted by movements of this small amount as a result of the earthquake, such movements do not adversely affect the performance of the CTB.

BEARING CAPACITY

STATIC BEARING CAPACITY OF THE CANISTER TRANSFER BUILDING

Table 2.6-9 presents the results of the bearing capacity analyses for the following static load cases. The minimum factor of safety required for static load cases is 3.

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

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

As indicated in this table, the gross allowable bearing pressure for the Canister Transfer Building to obtain a factor of safety of 3.0 against a shear failure from static loads is greater than 6 ksf. However, loading the foundation to this value may result in undesirable settlements. This minimum allowable value was obtained in analyses that conservatively assume $\phi = 0°$ and c = 2.2 ksf, as measured in the UU tests that are reported in Attachment 2 of Appendix 2A of the SAR, to model the end of construction. Using the estimated effective-stress strength of $\phi = 30°$ and c = 0 or the total-stress strength of $\phi = 21.1°$ and c = 1.1 ksf, as measured in the consolidated undrained triaxial shear tests performed on samples obtained from the Canister Transfer Building area (Attachment 6 of Appendix 2A of the SAR), results in higher allowable bearing pressures (> 20 ksf).

DYNAMIC BEARING CAPACITY OF THE CANISTER TRANSFER BUILDING

The dynamic bearing capacity was analyzed using the dynamic loads for the building that were developed in Calculation 05996.02-SC-5, (SWEC, 1999b). The development of these dynamic loads is described in SAR Section 4.7.1.5.3. As in the structural analyses discussed in Section 4.7.1.5.3., the seismic loads used in these analyses were combined using 100% of the enveloped zero period accelerations (ZPA) in one direction with 40% of the enveloped ZPA in each of the other two directions.

Table 2.6-10 presents the results of the bearing capacity analyses for the following cases, which include static loads plus dynamic loads due to the earthquake. The minimum factor of safety required for dynamic load cases is 1.1.

Case II	100%	N-S direction,	0%	Vertical direction,	100%	E-W direction.
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.
Case IVA	40%	N-S direction,	100%	Vertical direction,	40%	E-W direction.
Case IVB	40%	N-S direction,	40%	Vertical direction,	100%	E-W direction.
Case IVC	100%	N-S direction,	40%	Vertical direction,	40%	E-W direction.

Table 2.6-10 indicates the minimum factor of safety against a dynamic bearing capacity failure was obtained for Load Case IIIB, the load combination of full static with 40% of the earthquake loading acting in the N-S direction, 40% acting in the vertical direction, tending to unload the mat, and 100% acting in the E-W horizontal direction. This load case resulted in an actual soil bearing pressure of 3.31 kips per square foot (ksf), compared with an ultimate bearing capacity of ~9 ksf. The resulting factor of safety against a bearing capacity failure for this load case is ~3, which is much greater than 1.1, the minimum allowable factor of safety for seismic loading cases. In these analyses, no credit was taken for the fact that strength of cohesive soil increases as the rate of loading increases. Therefore, the Canister Transfer Building has an adequate factor of safety against a dynamic bearing capacity failure.

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Table 1
Foundation Loadings for the Canister Transfer Building

JOINT	EL.	MASS X	MASS Y	MASS Z	Ax	Ay	Az	N-S SHEAR X	UPLIFT	E-W SHEAR Z	ΣM	
								F_{Hx}	F_{V}	F_{Hz}	M_{ax}	M_{az}
	ft	k-sec²/ft	k-sec²/ft	k-sec²/ft	g	g	g	k	k	k	ft-k	ft-k
1	85	1257.0	1257.0	1257.0	0.805	0.720	0.759	32,583	29,142	51,126	156,828	132,913
2	130	490.7	490.7	490.7	0.864	0.764	0.834	15,652	12,072	13,179	461,278	477,808
3	170	299.2	299.2	157.0	0.939	0.823	0.966	9,047	7,987	4,684	366,264	676,491
4	190	219.8	166.3	219.8	0.955	0.839	1.067	6,759	4,509	7,552	717,417	642,112
5	190	0.0	52.9	0.0	0.000	2.013	0.000	0	3,429	0	0	0
6	170	0.0	0.0	142.2	0.000	0.000	2.368	0	0	10,834	812,515	0
							TOTAL	62,040	57,139	67,572	2,513,041	1,961,325
							WEIGHT		72,988			

Based on sliding and uplift forces from p 67 of Calc 05996.02-SC-5, Rev 1, which are applicable for "High" Moduli received from Geomatrix Calc 05996.02-G(PO18)-2, Rev 0.

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STONE & WEBSTER ENGINEERING CORPORATION
CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER

J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE
05996.02	G(B)	13-3	N/A	43

5010.65

Table 2

Sliding Stability of Canister Transfer Building Using Average Shear Strength from Direct Shear Tests Under Building and Half of Passive Resistance With a 1-ft Deep Perimeter Key

Joint	MASS X	MASS Y	MASS Z	N-S a_x	Vert a_y	E-W a_z	Static F_v	Earthquake Shear$_{N\text{-}S}$	Earthquake F_v	Earthquake Shear$_{E\text{-}W}$
	k-sec² / ft	*k-sec² / ft*	*k-sec² / ft*	*g*	*g*	*g*	*k*	*k*	*k*	*k*
1	1,257.0	1,257.0	1,257.0	0.805	0.720	0.769	40,475	32,583	29,142	31,126
2	490.7	490.7	490.7	0.864	0.764	0.834	15,801	13,652	12,072	13,178
3	299.2	299.2	157.0	0.939	0.829	0.966	9,634	9,047	7,987	4,884
4	219.8	166.9	219.8	0.955	0.839	1.067	5,374	6,759	4,509	7,552
5	0.0	52.9	0.0	0.000	2.013	0.000	1,703	0	3,429	0
6	0.0	0.0	142.2	0.000	0.000	2.366	0	0	0	10,834
CTB Mat Dimensions:			B =	165	ft	***Totals =***	**72,988**	**62,040**	**57,139**	**67,572**
			L =	265	ft					

For ϕ = 0.0 degrees	c = 1.80 ksf					N (*k*)	T (*k*) *Resisting*	V (*k*) *Driving*	FS
Earthquake Vertical Forces Acting Up	IIIA	$F_{v(Static)}$	40% $F_{H(NS)}$	100% $F_{v(Eqk)}$	40% $F_{H(EW)}$				
		72,988	24,816	-57,139	27,029	15,849	79,169	36,693	**2.16**
	IIIB	$F_{v(Static)}$	40% $F_{H(NS)}$	40% $F_{v(Eqk)}$	100% $F_{H(EW)}$				
		72,988	24,816	-22,855	67,572	50,132	79,169	71,985	**1.10**
	IIIC	$F_{v(Static)}$	100% $F_{H(NS)}$	40% $F_{v(Eqk)}$	40% $F_{H(EW)}$				
		72,988	62,040	-22,855	27,029	50,132	79,169	67,672	**1.17**
Earthquake Vertical Forces Acting Down	IVA	$F_{v(Static)}$	40% $F_{H(NS)}$	100% $F_{v(Eqk)}$	40% $F_{H(EW)}$				
		72,988	24,816	57,139	27,029	130,126	79,169	36,693	**2.16**
	IVB	$F_{v(Static)}$	40% $F_{H(NS)}$	40% $F_{v(Eqk)}$	100% $F_{H(EW)}$				
		72,988	24,816	22,855	67,572	95,843	79,169	71,985	**1.10**
	IVC	$F_{v(Static)}$	100% $F_{H(NS)}$	40% $F_{v(Eqk)}$	40% $F_{H(EW)}$				
		72,988	62,040	22,855	27,029	95,843	79,169	67,672	**1.17**

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TABLE 2.6-9
SUMMARY - ALLOWABLE BEARING CAPACITY OF CANISTER TRANSFER BUILDING
Based on Static Loads

Case	F_V	$EQ_{H\,N\text{-}S}$	$EQ_{H\,E\text{-}W}$	$\Sigma M_{@N\text{-}S}$	$\Sigma M_{@E\text{-}W}$	β_B $EQ_{H\,E\text{-}W}$	β_L $EQ_{H\,N\text{-}S}$	GROSS q_{ult}	GROSS q_{all}	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
IA - Static Undrained Strength	72,988	0	0	0	0	0.0	0.0	18.95	**6.31**	0.0	0.0	165.0	265.0	**1.67**	**11.35**
IB - Static Effective Strength	72,988	0	0	0	0	0.0	0.0	135.00	**45.00**	0.0	0.0	165.0	265.0	**1.67**	**80.88**

$c =$	**3,180**	Undrained strength (psf) & $\phi = 0$.
$\phi =$	**30.0**	Effective stress friction angle (deg), c = 0.
$B =$	**165**	Footing width (ft)
$L =$	**265**	Footing length (ft)
$D_f =$	**5.0**	Depth of footing (ft)
$\gamma =$	**90**	Unit weight of soil (pcf)
$\gamma_{surch} =$	**80**	Unit weight of surcharge (pcf)
$FS =$	**3**	Factor of safety for static loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\,E\text{-}W}$ or $EQ_{H\,N\text{-}S}$

$\beta_B = \tan^{-1}[(EQ_{H\,E\text{-}W}) / F_V]$ = Angle of load inclination from vertical (deg) as f(width).

$\beta_L = \tan^{-1}[(EQ_{H\,N\text{-}S}) / F_V]$ = Angle of load inclination from vertical (deg) as f(length).

$e_B = \Sigma M_{@N\text{-}S} / F_V$ $\quad$ $e_L = \Sigma M_{@E\text{-}W} / F_V$

$B' = B - 2\,e_B$ $\quad$ $L' = L - 2\,e_L$

$q_{actual} = F_V / (B' \times L')$

[geotl\05996\calc\brng_cap\can_xfr.xls Table 2.6-9

TABLE 2.6-10

SUMMARY - ALLOWABLE BEARING CAPACITY OF CASK STORAGE PADS

Based on Dynamic Loads Due to Design Basis Ground Motion: PSHA 2,000-yr Return Period

Case	F_V	$EQ_{H\ N-S}$	$EQ_{H\ E-W}$	ΣM_{N-S}	ΣM_{E-W}	β_B $EQ_{H\ E-W}$	β_L $EQ_{H\ N-S}$	GROSS q_{ult}	GROSS q_a	e_B	e_L	EFFECTIVE B'	EFFECTIVE L'	EFFECTIVE q_{actual}	FS_{actual}
	k	k	k	ft-k	ft-k	deg	deg	ksf	ksf	ft	ft	ft	ft	ksf	
II	72,968	62,040	67,572	2,513,041	1,361,325	42.8	40.4	10.98	9.98	34.4	26.9	96.1	211.3	3.59	3.06
IIIA	15,649	24,815	27,029	1,006,218	784,530	59.8	57.4	8.75	7.95	65.4	49.5	38.2	166.0	2.50	3.50
IIIB	50,132	24,815	67,572	2,513,041	784,530	53.4	26.3	9.95	9.04	50.1	15.6	64.7	233.7	3.31	3.00
IIIC	50,132	62,040	27,029	1,006,218	1,361,325	28.3	51.1	14.43	13.12	20.1	39.1	124.9	186.9	2.15	6.72
IVA	130,127	24,815	27,029	1,006,218	784,530	11.7	10.8	17.21	15.64	7.7	6.0	149.6	252.9	3.44	5.00
IVB	95,644	24,815	67,572	2,513,041	784,530	35.2	14.5	13.98	12.70	26.2	8.2	112.6	248.6	3.42	4.08
IVC	95,644	62,040	27,029	1,006,218	1,361,325	15.7	32.9	15.65	14.22	10.5	20.5	144.0	224.1	2.97	5.27

c = 3,180 Undrained strength (psf)

ϕ = 0.0 Friction angle (deg)

B = 165 Footing width (ft)

L = 265 Footing length (ft)

D = 5.0 Depth of footing (ft)

γ = 90 Unit weight of soil (pcf)

$\gamma_{surch.}$ = 80 Unit weight of surcharge (pcf)

FS = 1.1 Factor of safety for dynamic loads.

F_V = Vertical load (Static + EQ_V)

EQ_H = Earthquake: Horizontal force. $F_H = EQ_{H\ E-W}$ or $EQ_{H\ N-S}$

$\beta_B = \tan^{-1}[(EQ_{H\ E-W})/F_V]$ = Angle of load inclination from vertical (deg) as f(width)

$\beta_L = \tan^{-1}[(EQ_{H\ N-S})/F_V]$ = Angle of load inclination from vertical (deg) as f(length)

$e_B = \Sigma M_{N-S}/F_V$ $\quad e_L = \Sigma M_{E-W}/F_V$

$B' = B - 2e_B$ $\quad L' = L - 2e_L$

$q_{actual} = F_V/(B' \times L')$

[ssc(\05996\calcs\brng_cap\cask_pads.xls Table 2.6-10

STONE & WEBSTER ENGINEERING CORPORATION

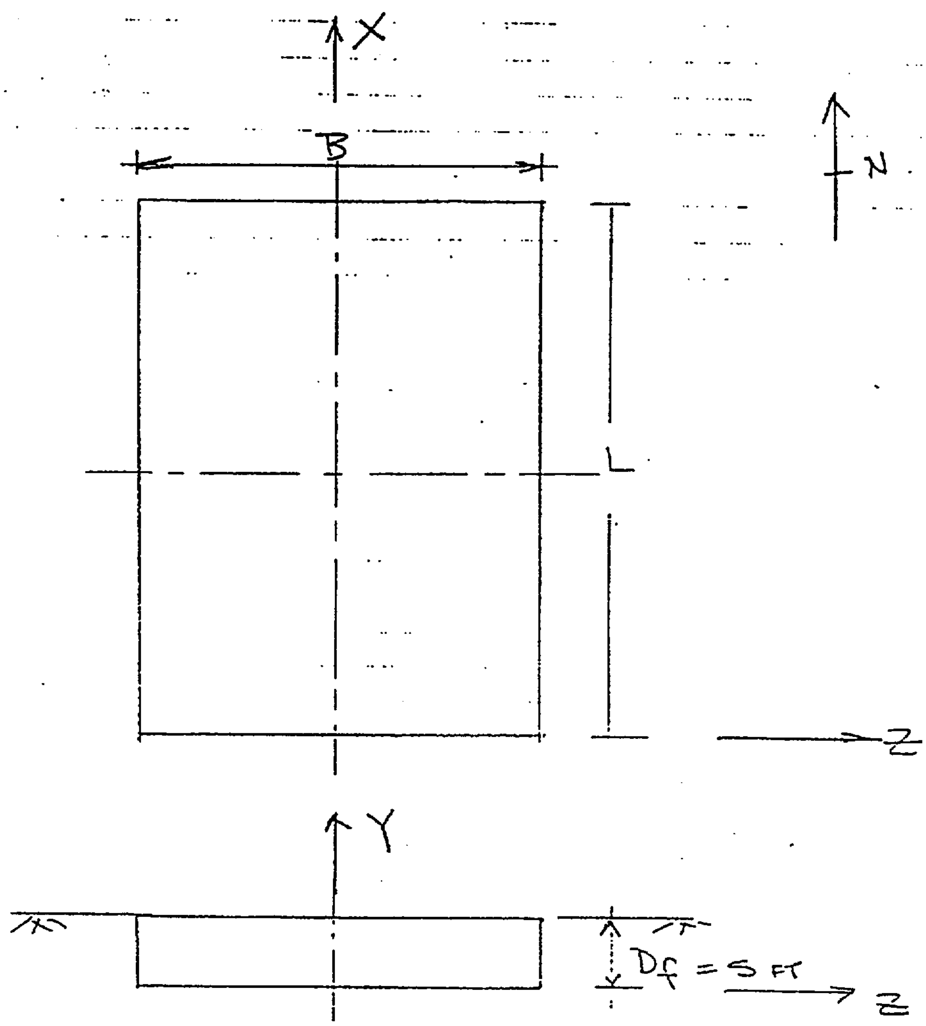
5010.65

CALCULATION SHEET

CALCULATION IDENTIFICATION NUMBER				PAGE 47
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	
05996.02	G(B)	13-3	N/A	

FIGURE 1

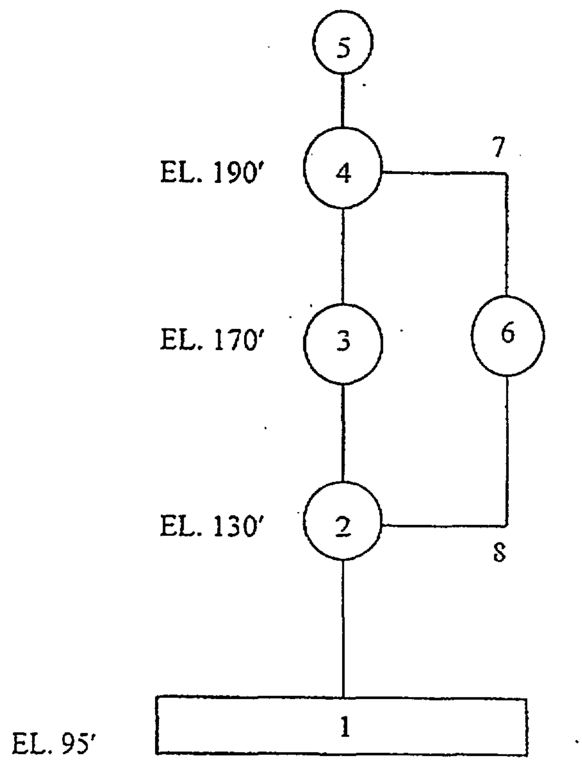
FOUNDATION SCHEMATIC & COORDINATE SYSTEM



Note: The coordinate system is consistent with that used in Calculation 05996.02-SC-5.

FIGURE 2

CANISTER TRANSFER BUILDING STICK MODEL



Note: From Calculation 05996.02-SC-5, Rev 1.

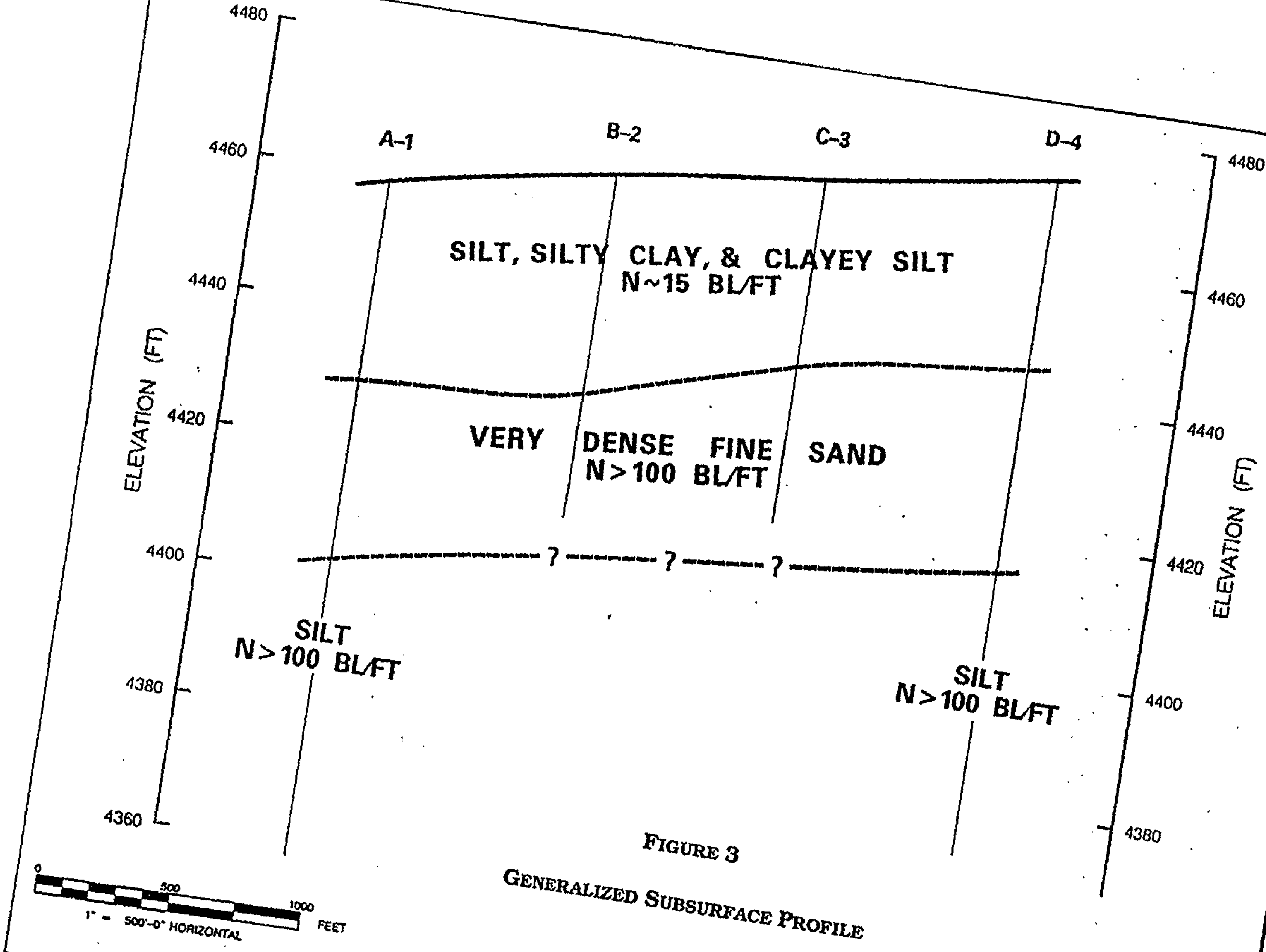


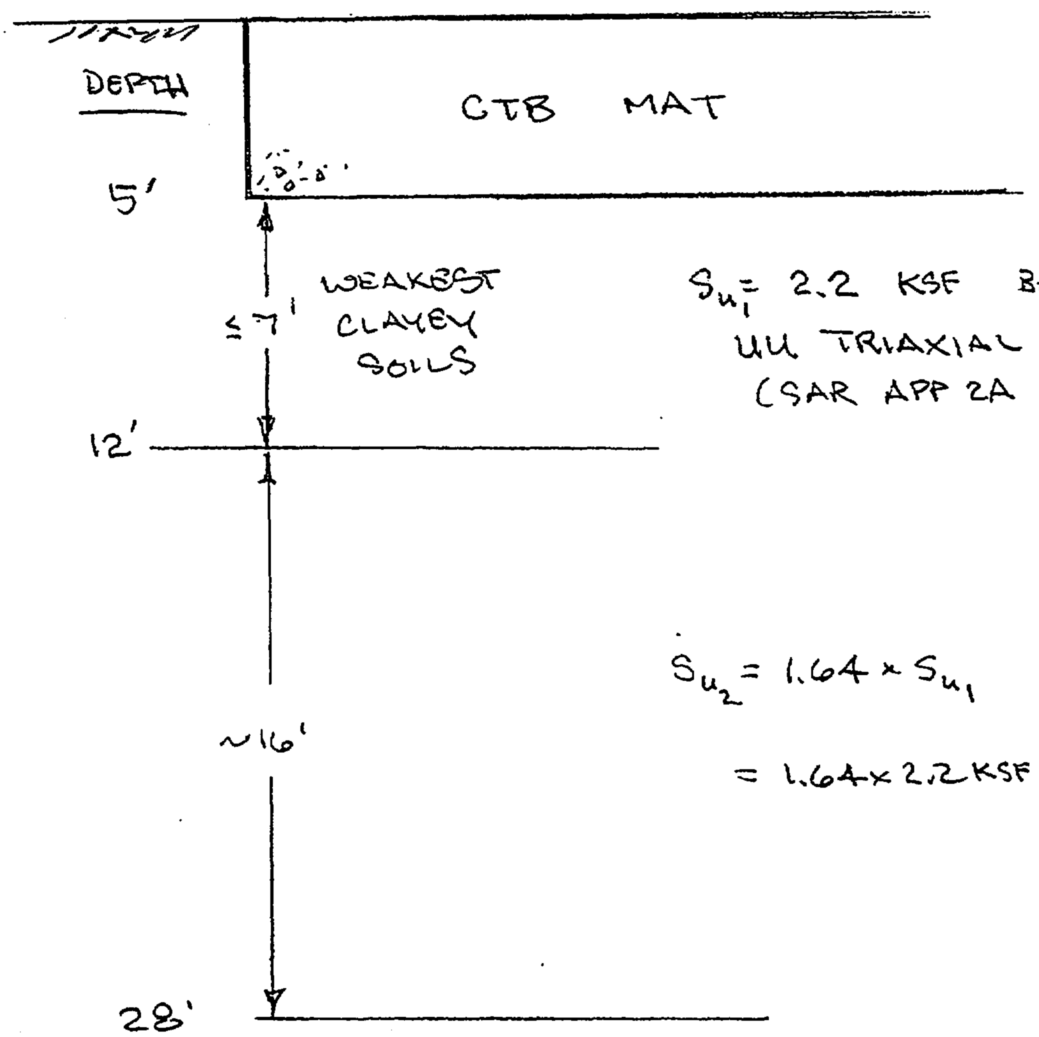
FIGURE 3

GENERALIZED SUBSURFACE PROFILE

Note: From Calculation 05996.01-G(B)-03-1.

FIGURE 4

DETERMINATION OF WEIGHTED AVERAGE VALUE OF S_U BASED ON RELATIVE STRENGTH DIFFERENCE OF DEEPER LYING SOILS MEASURED IN CONE PENETRATION TESTS



$S_{u_1} = 2.2$ KSF BASED ON UU TRIAXIAL TESTS (SAR APP 2A ATT 2)

$S_{u_2} = 1.64 \times S_{u_1}$

$= 1.64 \times 2.2\text{ KSF} = 3.6\text{ KSF}$

$$S_{u\,AVG} = \frac{7' \times 2.2\text{ KSF} + 16' \times 1.64 \times 2.2\text{ KSF}}{23\text{ FT}} = 3.18\text{ KSF}$$

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05996.02	G(B)	13-3	N/A	

FIGURE 5

ESTIMATE STRESSES UNDER THE CANISTER TRANSFER BUILDING AT COMPLETION OF CONSTRUCTION

$$\sigma_v = \frac{F_v}{A}$$

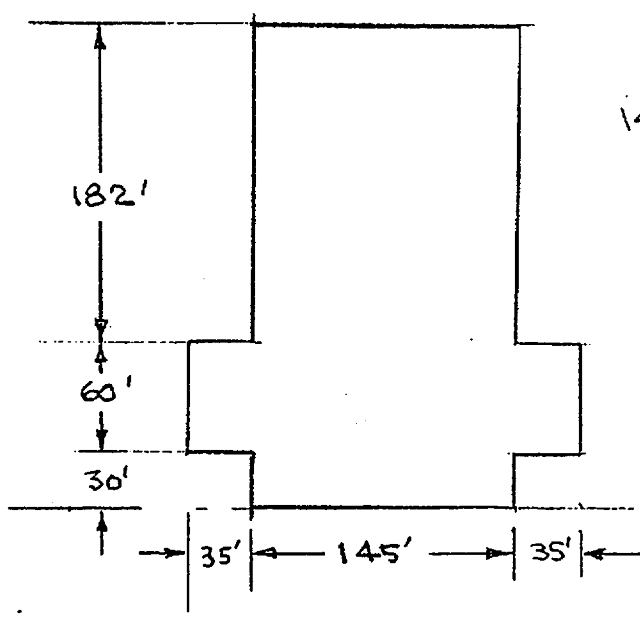
F_v = TOTAL DEAD WT

A = AREA OF MAT

SOURCES OF DATA:

F_v = 72,988 K STATIC D.L. p37 OF CALC 05996.02-SC-5-1

MAT AREA: DRAWING 0599602-EC-2-A



AREA (FT²)

145' × 182' = 26,390

60' × (145+70) = 12,900
(215')

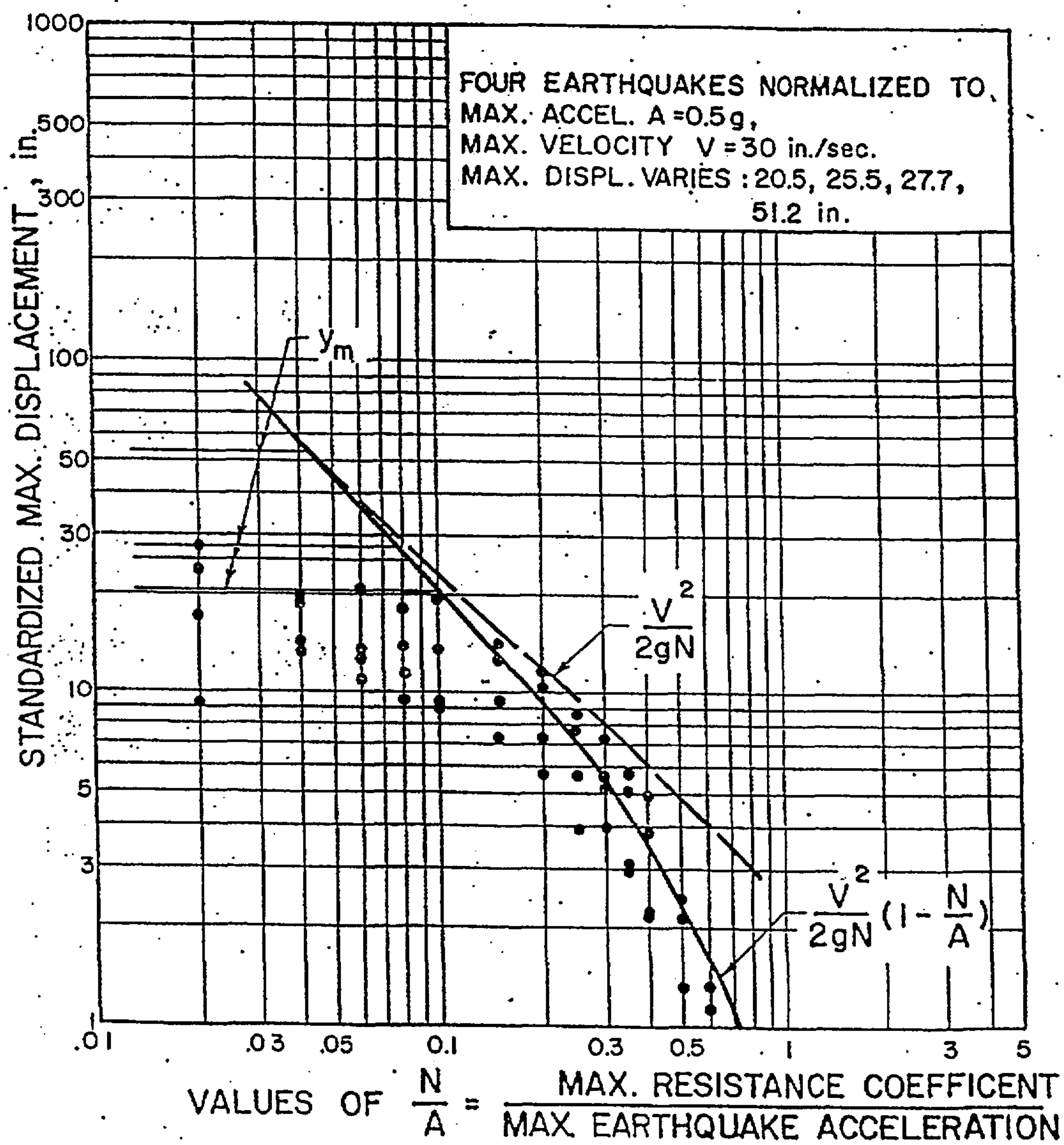
30' × 145' = 4,350

43,640 FT²

⇒ σ_v = 72,988^K / 43,640 FT² = 1.67 KSF

FIGURE 6

STANDARDIZED DISPLACEMENT FOR NORMALIZED EARTHQUAKES (SYMMETRICAL RESISTANCE)



Note: From Newmark (1965)

TABLE 6

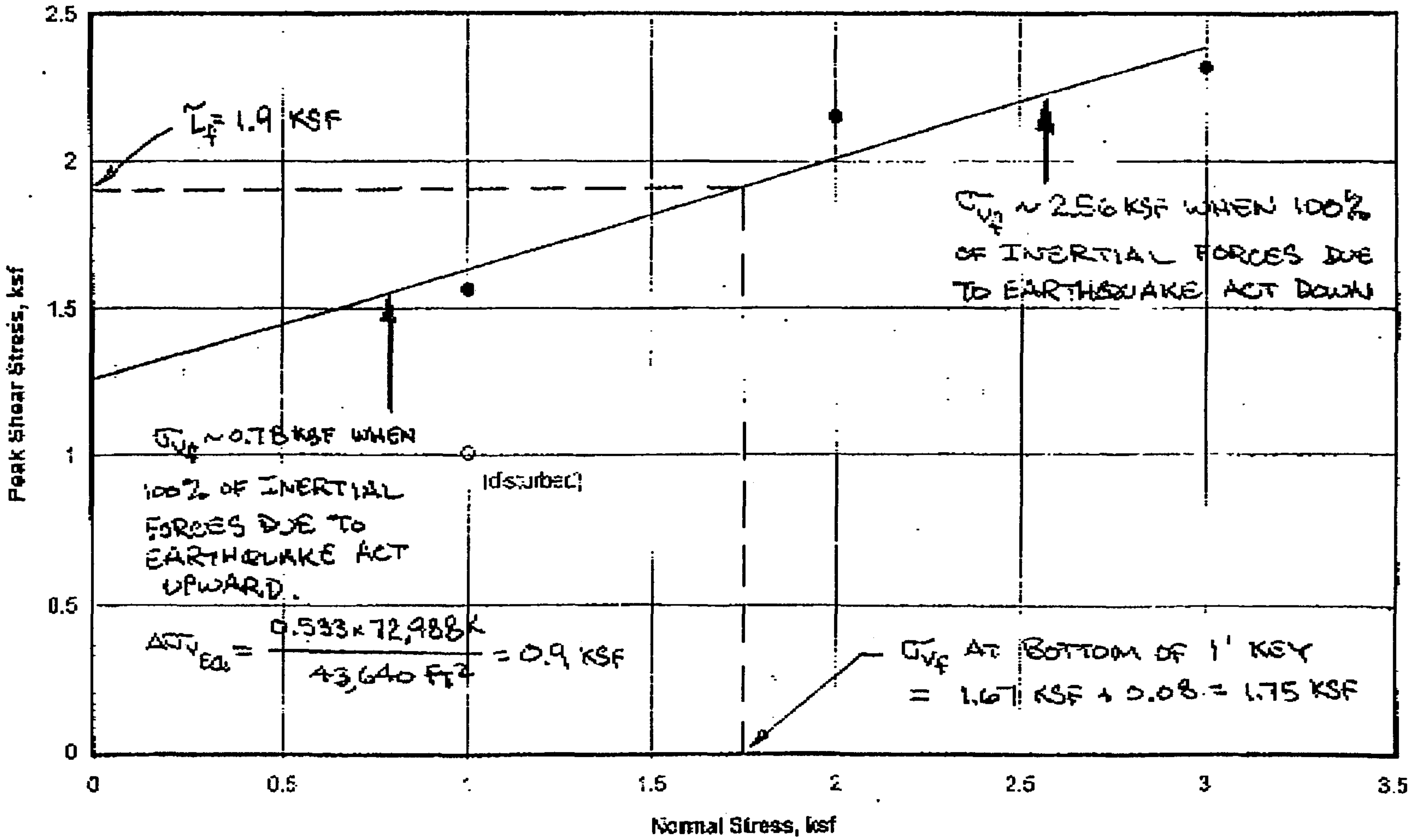
SUMMARY OF TRIAXIAL TEST RESULTS FOR SOILS WITHIN ~10 FT OF GROUND SURFACE AT THE SITE

Boring	Sample	Depth ft	Elev ft	w %	ATTERBERG LIMITS LL	PL	PI	USC Code	γ_m pcf	γ_d pcf	e_o	c_c ksf	s_u ksf	ε_f %	Type	Date
B-1	U-2C	5.9	4453.9	47.1	66.1	33.4	32.7	MH	79.3	53.9	2.15	0.0	2.03	1.7	CU	Nov '99
B-1	U-2B	5.3	4454.5	52.9	80.6	40.9	39.7	MH	70.6	46.3	2.67	1.0	2.21	6.0	CU	Nov '99
B-4	U-3D	10.4	4462.1	27.4	42.5	24.7	17.8	CL	85.5	67.1	1.53	1.3	2.16	4.0	CU	Jan. '97
C-2	U-2D	11.1	4453.4	35.6	See U-2C & E[1]			CL	78.5	57.9	1.93	1.3	2.39	11.0	CU	Jan. '97
CTB-1	U-3D	6.7	4463.7	47.9	See U-3C[2]			CH	91.6	62.1	1.73	1.7	2.84	5.0	CU	June '99
CTB-4	U-2D	9.5	4465.5	45.2	See U-2E[2]			CH	87.7	60.4	1.81	1.7	3.11	6.0	CU	June '99
CTB-6	U-3D	6.3	4467.9	52.7				CH	85.7	56.2	2.02	1.7	2.70	7.0	CU	June '99
CTB-N	U-1B	5.7	4468.4	30.1	41.3	22.5	18.8	CL	100.6	77.3	1.20	1.7	3.00	8.0	CU	Nov '98
CTB-N	U-2B	7.7	4466.4	65.4	See U-2A[2]			MH	74.6	45.1	2.76	1.7	2.43	13.0	CU	June '99
CTB-N	U-3D	10.5	4463.6	52.2	61.1	30.8	30.3	CH	86.5	56.7	1.98	1.7	2.73	7.0	CU	June '99
CTB-S	U-1B	5.9	4465.7	73.6	66.2	40.9	25.3	MH	78.0	44.9	2.79	1.7	2.05	12.0	CU	Nov '98
CTB-S	U-2D	6.4	4466.1	54.6	57.9	28.9	29.0	CH	90.0	58.2	1.92	1.7	2.40	5.0	CU	June '99
B-1	U-2D	6.5	4453.3	45.2	59.8	34.7	25.1	MH	75.7	52.8	2.22	2.1	3.26	15.0	CU	Mar '99
B-3	U-1B	5.2	4463.0	33.5	52.4	25.2	27.2	MH	90.6	67.9	1.50	2.1	3.55	8.0	CU	Mar '99
C-2	U-1D	6.3	4458.2	50.5	70.3	41.3	29.0	MH	74.5	49.5	2.43	2.1	3.03	12.0	CU	Mar '99

NOTES 1 Attachment 2 of SAR Appendix 2A.
2 Attachment 6 of SAR Appendix 2A.

ATTACHMENT A p A1/4
TO CALC 05996.02-G(B)-13-3

FIGURE 9
DIRECT SHEAR TEST
Boring CTB-6, Sample U-3B&C
CANISTER TRANSFER BUILDING AREA



CALC 05996.02-G(B)-05-2 - p 34

PFSF SKULL VALLEY, UT
SAR APP 2A ATT 7

ATTACHMENT A p A2
CALC 05996.02-G(B)-13-3

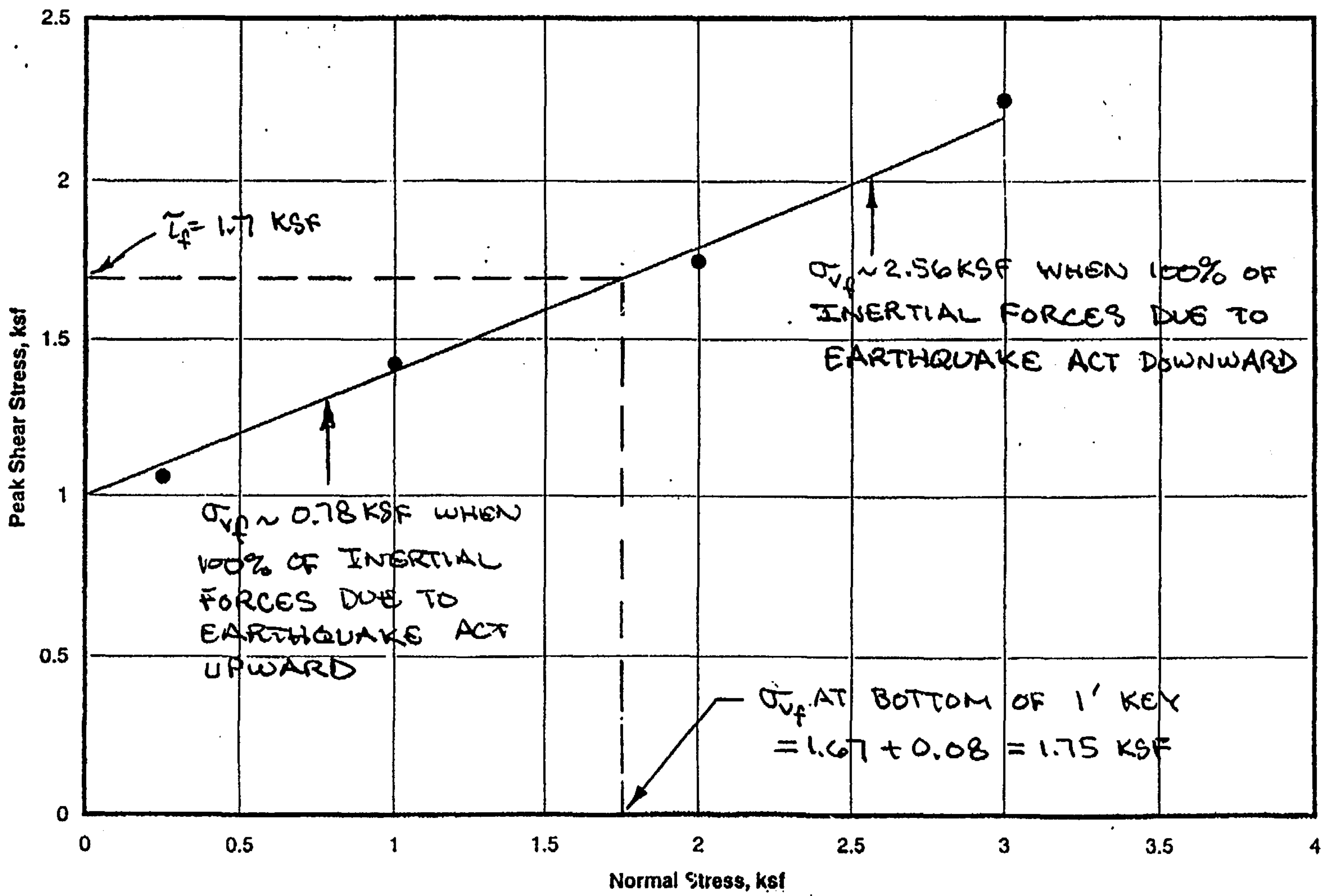
STONE & WEBSTER ENGINEERING CORPORATION

FIGURE 10

DIRECT SHEAR TEST

Boring CTB-S, Sample U-1AA&C

CANISTER TRANSFER BUILDING AREA



CALC 05996.02-G(B)-05-2 P 35

STONE & WEBSTER ENGINEERING CORPORATION

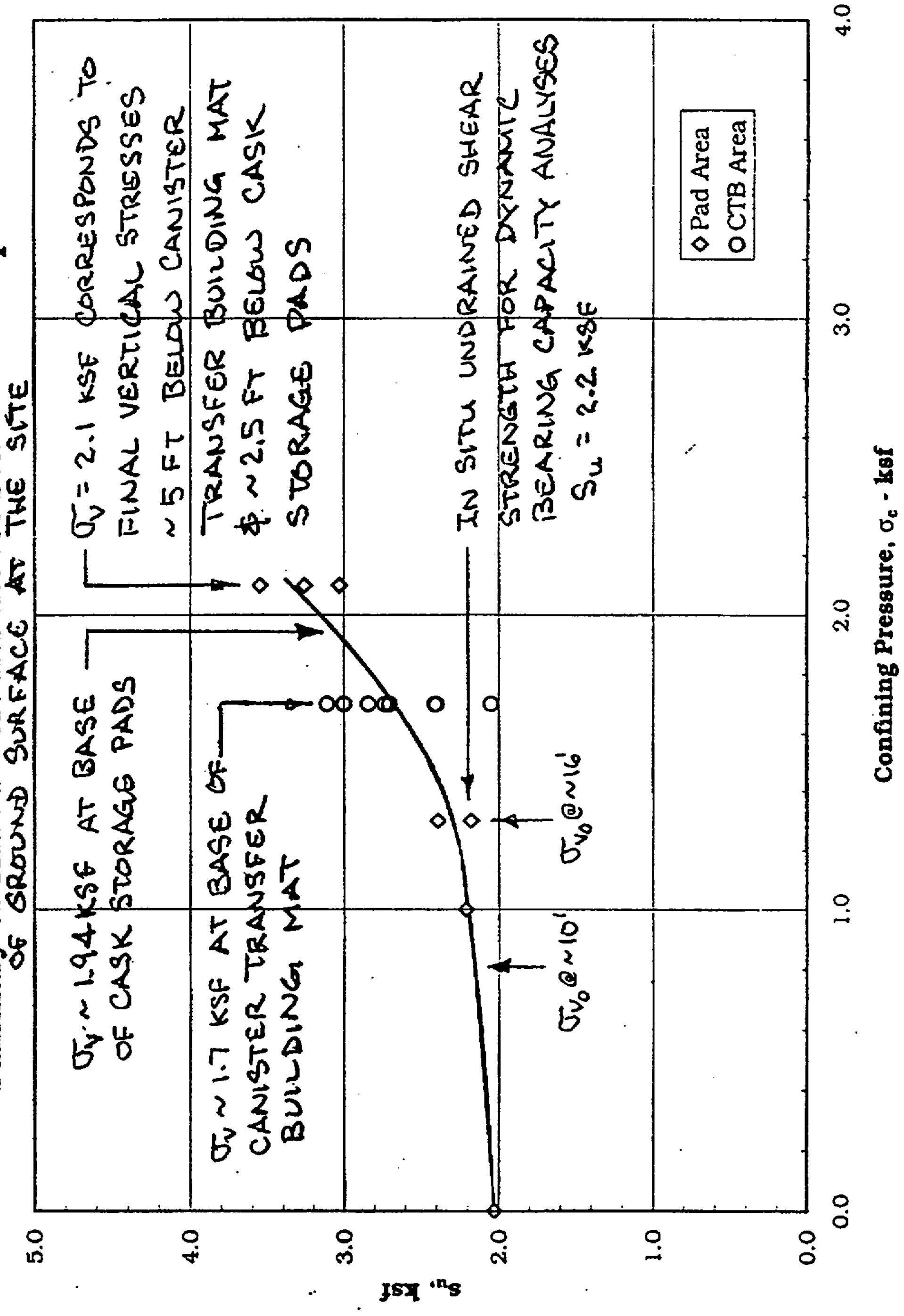
CALCULATION SHEET

5010.65

CALCULATION IDENTIFICATION NUMBER				
J.O. OR W.O. NO.	DIVISION & GROUP	CALCULATION NO.	OPTIONAL TASK CODE	PAGE 36
05996.02	G(B)	05-2		

Figure 11

Summary of Triaxial Test Results for Soils Within Depth of ~10 ft OF GROUND SURFACE AT THE SITE



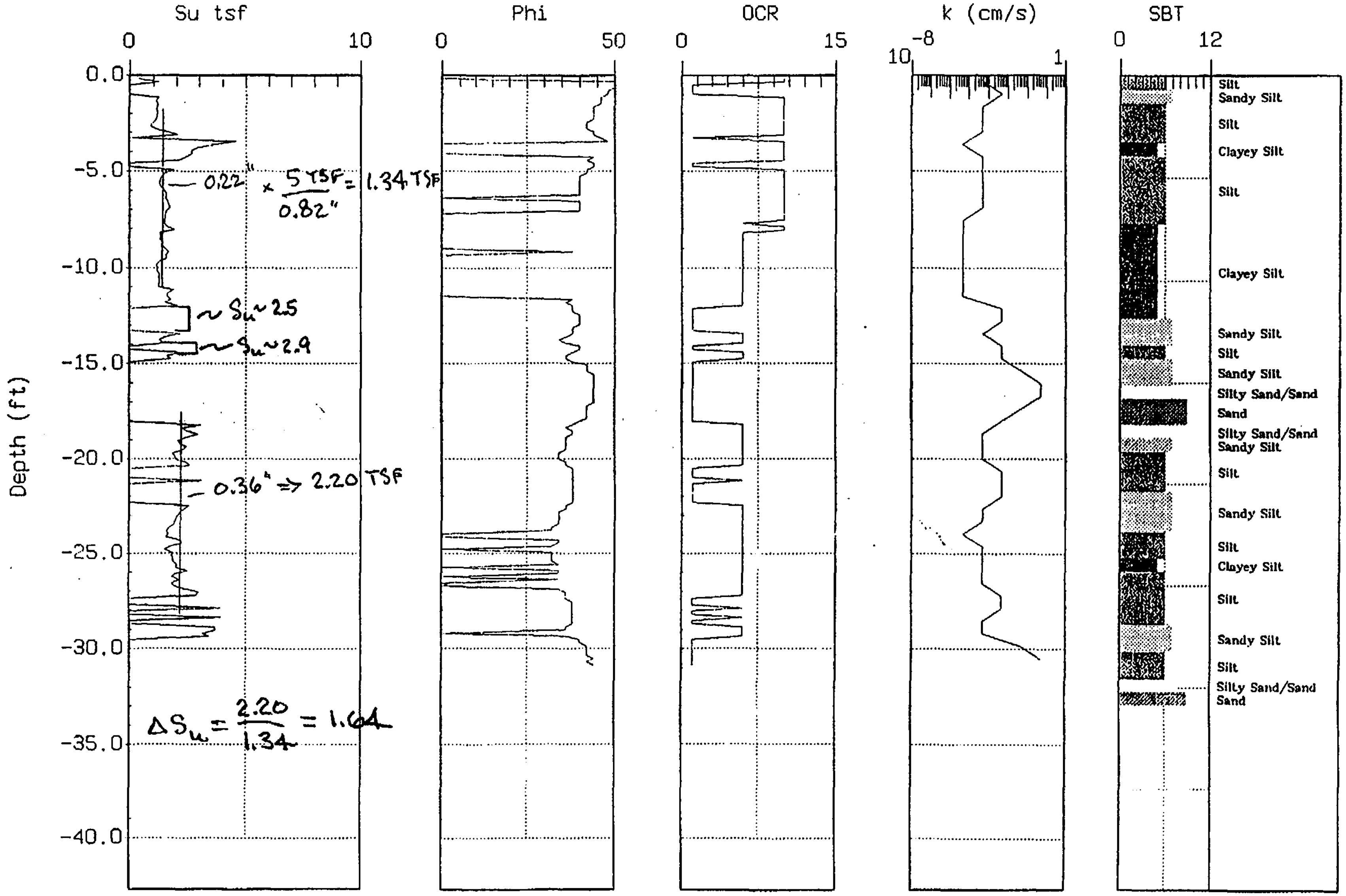
ATTACHMENT A p A4
CALC 05996.02-G(B)-13-3

[geol]\05996\lab\triax-1.xls on 6/15/2000

Stone & Webster

Site: CPT-37
Location: PFSF (05996.02)

Cone: 20 TON A 041
Date: 04:23:99 11:36



Max. Depth: 30.84 (ft)
Depth Inc.: 0.164 (ft)

Permeability k: estimated from soil type

SBT: Soil Behavior Type (Robertson and Campanella 1988)

App. D-37

(5) WED JUN 2 2000 P.J. Index

ATTACHMENT B

CALC 05996.02-G(B)-13-3

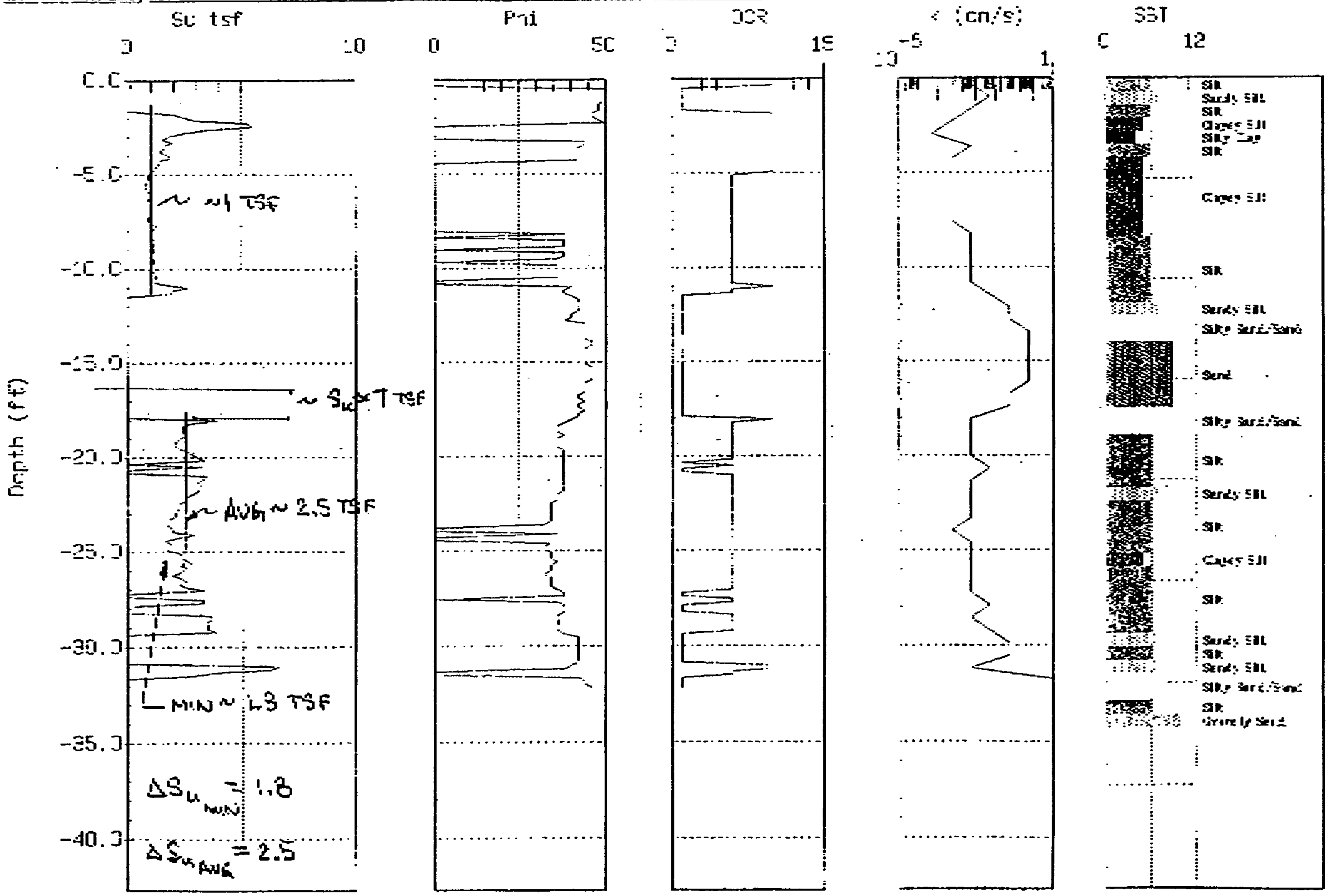
P B1/2

WED JUN 2 2000 P. I. Tridem

ATTACHMENT B
B2
CALC 05996.02-G(B)-13-3

CONETEC

Stone & Webster



Max Depth: 35.16 (ft)

Permeability is estimated from soil type

SBT: Soil Behavior Type (Robertson and Campanella 1983)

App. D-38

QA CATEGORY I
CALCULATION CHECKLIST

Calculation No. 05996.02-G(B)-13
Revision No. 3

Project No. 05996.02
Job Book File Location Q2.9

	Yes	No	N/A
Method			
Identify the method used to verify the "Method" of the calculation			
• By design review	✓		
• Compare the Method with another calculation			✓
• Alternate calculation			✓
If the compare method was used, is the statement identifying the other calculation identified in this calculation?			✓
If an alternate calculation was used for a QA Category I calculation, is it included with the calculation?			✓
Is the calculation method acceptable?	✓		
Assumptions			
Affirmative answers to the following questions are required:			
• Are all assumptions uniquely identified as assumptions and adequately described?	✓		
• Are all assumptions reasonable?	✓		
• Are all assumptions that require confirmation at a later date specifically identified as assumptions that must be confirmed?	✓		
For Revisions to the Calculation			
• Are changes clearly identified?	✓		
• For QA Category I calculations, is a reason for the revision given?	✓		
• Does the calculation identify the calculation, including revision, when applicable, which is superseded?	✓		

Private Fuel Storage Facility

PP 5-21-1
Attachment 2
Page 2 of 2

QA CATEGORY I
CALCULATION CHECKLIST

Calculation No. 05996.02-G(B)-13
Revision No. 3

Project No. 05996.02
Job Book File Location Q2.9

	Yes	No	N/A
• Are affected pages identified with the new calculation number or revision number?	✓		
• When applicable, is an alternate calculation included as part of the calculation?			✓
• When applicable, is a statement identifying the calculation to which the method was compared included as part of the revision?			✓

Thomas Y. Chang	Thomas Y. Chang	6-19-2000
Printed Name	Signature	Date

Exhibit A

Attachment 3

HOLTEC INTERNATIONAL

Holtec Center, 555 Lincoln Drive West, Marlton, NJ 08053
Telephone (856) 797- 0900
Fax (856) 797 - 0909

MULTI CASK RESPONSE AT PFS ISFSI FROM 2000-YR SEISMIC EVENT (REV. 2)

FOR

PFS

Holtec Report No: HI-2012640

Holtec Project No: 70651

Report Class : SAFETY RELATED

HOLTEC INTERNATIONAL

DOCUMENT ISSUANCE AND REVISION STATUS[1]

DOCUMENT NAME: MULTI-CASK RESPONSE AT PSF ISFSI FROM 2000-YR SEISMIC EVENT (REV. 2)

DOCUMENT NO.:	HI-2012640	CATEGORY: ☐ GENERIC
PROJECT NO.:	70651	☒ PROJECT SPECIFIC

Rev. No. [2]	Date Approved	Author's Initials	VIR #	Rev. No.	Date Approved	Author's Initials	VIR #
0	3/29/01	CWB	308110				

4.0 ASSUMPTIONS AND MODELING OF THE CASK / PAD SIMULATION

The assumptions employed in the modeling of the cask/pad dynamic system and the details of the equation development are identical to those employed in the previous analyses and are not repeated here. A recent technical paper contains similar information regarding methodology, acceptance criteria, and modeling of the HI-STAR 100 (metal cask) Storage System [5].

Exhibit B

EXHIBIT B

ACI 230.1R-90

(Reapproved 1997)

State-of-the-Art Report on Soil Cement

reported by ACI Committee 230

Wayne S. Adaska, Chairman

Ara Arman
Robert T. Barclay
Theresa J. Casias
David A. Crocker
Richard L. De Graffenreid
John R. Hess
Robert H. Kuhlman
Paul E. Mueller
Harry C. Roof
Dennis W. Super
James M. Winford
Anwar E. Z. Wissa

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

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

CONTENTS

1—INTRODUCTION

1.1—Scope

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

ACI Materials Journal, V. 87, No. 4, July-August 1990.
Pertinent discussion of the full report will be published in the March-April 1991 *ACI Materials Journal* if received by Nov. 1, 1990.

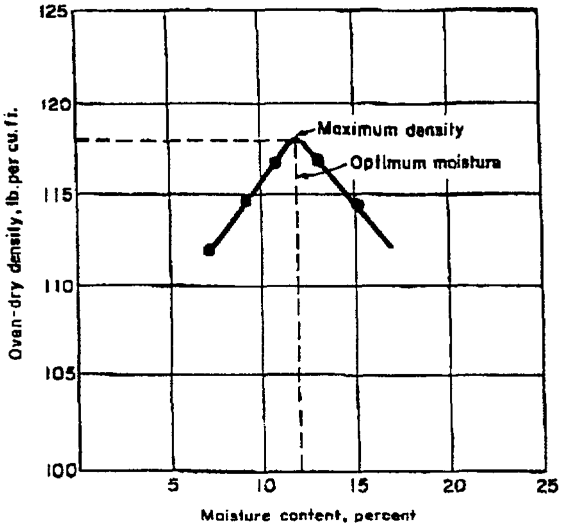


Fig. 4.1—Typical moisture-density curve

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

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

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

4.3—Compressive strength

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

Typical ranges of 7- and 28-day unconfined compressive strengths for soaked, soil-cement specimens are given in Table 4.1. Soaking specimens prior to testing is recommended since most soil-cement structures may become permanently or intermittently saturated during their service life and exhibit lower strength under saturated conditions. These data are grouped under broad textural soil groups and include the range of soil types normally used in soil-cement construction. The range of values given are representative for a majority of soils normally used in the United States in soil-cement construction. Fig. 4.2 shows that a linear relationship can be used to approximate the relationship between compressive strength and cement content, for cement contents up to 15 percent and a curing period of 28 days.

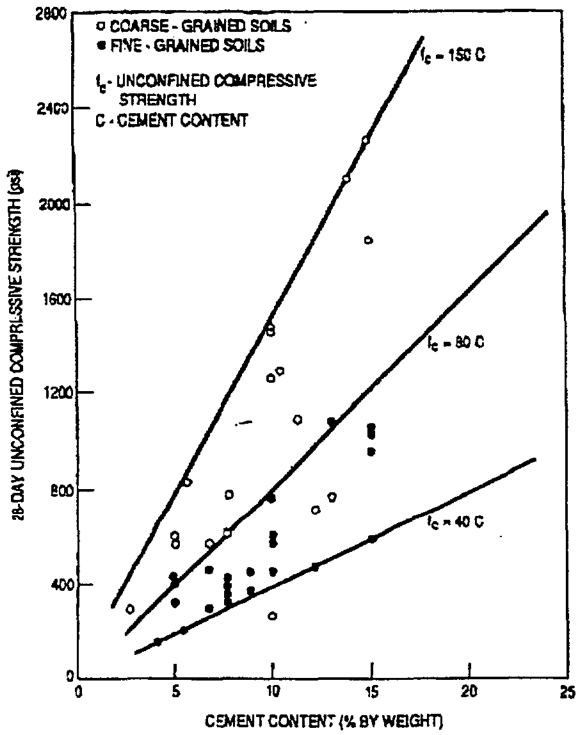


Fig. 4.2—Relationship between cement content and unconfined compressive strength for soil-cement mixtures

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

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

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

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

where

R = flexural strength, psi
f_c' = unconfined compressive strength, psi