Private Fuel Storage, LLC

July 20, 2001

P.O. Box C4010, La Crosse, WI 54602-4010
John D. Parkyn, Chairman of the Board

U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, D.C. 20555-0001

DATA NEEDED FOR NRC REVIEW
DOCKET NO. 72-22/TAC NO. L22462
PRIVATE FUEL STORAGE FACILITY
PRIVATE FUEL STORAGE L.L.C.

References:
1. NRC letter, Delligatti to Parkyn, "Acceptance of License Application Amendment/Schedule for SER Supplement", dated June 20, 2001
2. July 5, 2001 teleconference between PFS, S&W and the NRC
3. July 16, 2001 teleconference between PFS, S&W and the NRC
4. July 18, 2001 teleconference between PFS (Parkyn) and NRC (Brach)

In Reference 1, the NRC identified additional data needed from Private fuel Storage (PFS) in order for the staff to conclude its technical review of the PFS License Application Amendment No. 22. Additional clarification of the information needed was provided in the July 5 and July 16, 2001 teleconferences (Reference 2 and 3) with the NRC. The PFS response to the request for additional data is enclosed. Supporting calculations will be issued to the NRC by July 27, 2001.

PFS believes that with the information and commitments supplied in this enclosure, we have provided all the information that the NRC has requested and is in agreement with commitments between PFS and the NRC (Reference 4) to bring closure to the review process. PFS would now expect that this will allow the NRC to proceed with the issuance of the FEIS and SER well ahead of the proposed schedule dates in the Reference 1 letter.

If you have any questions regarding this submittal, please contact me at 608-787-1236 or Mr. J. L. Donnell, Project Director, at 303-741-7009.

Sincerely,

John D. Parkyn, Chairman
Private Fuel Storage L.L.C.

JDP:JRJ:pjs
Enclosure
F:\GNR\PFS\LETTERS\072001.doc

NMSS01 Public

copy to: (with enclosure)

E. William Brach
Mark Delligatti
Scott Flanders
John Parkyn
Asadul Chowdhury
Jay Silberg
Sherwin Turk
Greg Zimmerman
Scott Northard
Richard E. Condit
John Paul Kennedy
Joro Walker
Denise Chancellor
Duncan Steadman

MISSING INFORMATION IDENTIFIED BY THE NRC STAFF DURING THE ACCEPTANCE REVIEW

6-20-01 NRC Letter (Delligatti to Parkyn)

Item 1

NRC Comment

PFS did not completely or adequately explain in the stand-alone document its reasons for changing the design of the storage pads or the foundation of the Canister Transfer Building (CTB) and their engineered soil environments (including the soil cement), related to the stability of these structures during an earthquake event.

PFS Response

GENERAL—The "stand-alone document" referred to above was formally requested by the NRC in its letter, Brach to Parkyn, "March 30, 2001 License Application Amendment", dated May 7, 2001. The purpose of the document was to assist the NRC in its review of the License Amendment by providing a "roadmap" that would quickly guide the NRC through the various changes and provide an explanation of the reasons, effects, adequacy, and supporting basis for each change. The intent in providing this document was to shorten the required time for the review process and perhaps eliminate the need for an RAI. That "stand-alone document," however, is not a licensing document, but only a summary of the licensing bases for the PFSF, and does not present any new licensing positions for consideration by the NRC. The PFSF licensing basis is contained in the PFSF License Application as submitted through License Amendment #22. Thus, even if the "stand-alone document" were lacking some information, this should not constrain the NRC's ability to complete its review of the PFS License Application – Amendment #22.

STORAGE PADS—The only design change to the storage pads was to change the length of the pad from 64 ft to 67 ft. A detailed explanation of the reason for this increase is provided in Enclosure 2 of the "stand-alone document" pages 7 through 10, submitted with PFS Letter, Parkyn to U.S. NRC, Data Needed for NRC Review of License Amendment #22, dated May 31, 2001. For its stated purpose of providing a roadmap, Enclosure 2 also discusses the effects this change had on other aspects of the facility design, including a listing of affected reports and analyses. Enclosure 2 also addressed the adequacy of the new design and provided a listing of sections in the PFS License Application that were revised to incorporate this design change. Much of this information was also previously provided in PFS Letter, Donnell to U.S. NRC, Summary of Changes for PFSF License Application Amendment #22, dated April 16, 2001.

CANISTER TRANSFER BUILDING (CTB)—The reasons for changes to the foundation of the CTB are explained in detail in Enclosure 2 pages 10 through 13, submitted with PFS Letter, Parkyn to U.S. NRC, Data Needed for NRC Review of License Amendment #22, dated May 31, 2001. Again, Enclosure 2 explains the reasons, discusses the effects, and addresses the adequacy of the new design, and provides a listing of sections in the PFS License Application that were revised to incorporate this design change. The majority of this information was also previously provided in PFS Letter, Donnell to U.S.

NRC, Summary of Changes for PFSF License Application Amendment #22, dated April 16, 2001.

SOIL ENVIRONMENTS—Issues concerning soil cement and its relation to stability of the CTB and the storage pads are addressed in item 2 below.

Item 2

NRC Comment

The Staff also requested that PFS provide revised analyses of the stability of the storage pads to include a clear identification of the potential failure modes and failure surfaces, and the material strengths required to satisfy the regulatory requirement, considering the critical failure modes.

- The calculated seismic acceleration in Amendment 22 increased from approximately 0.53 g to 0.7 g. To accommodate this increase, PFS revised their soil foundation design and used a modified approach to demonstrate foundation stability. PFS previously relied on friction forces between clay and soil-cement layers and passive resistance from soil cement surrounding the pad. The modified approach relies on assumed bonding forces between clay and soil cement layers, and between the pad and the soil cement layer to transfer shear to the underlying soil, and ensure stability during an earthquake event.

PFS Response

The quoted statement from the NRC comment above, "*PFS previously relied on friction forces between clay and soil-cement layers and passive resistance from soil cement surrounding the pad*" is not correct. PFS has not relied on the friction forces between the clay and soil-cement layers to demonstrate foundation stability. PFS relies exclusively on the cohesion forces between these interfaces to demonstrate foundation stability. As indicated on p. 2.6-36 of the SAR:

> *"The results of these tests are presented in Attachment 7 of the Appendix 2A and they are plotted in Figure 7 of Calculation 05996.02-G(B)-05 (SWEC, 2001a). 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 of the cask storage pads constructed directly on the silty clay are performed using the shear strength measured in these direct shear tests for the normal stress that equals 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 Calculation 05996.02-G(B)-05 (SWEC, 2001a), this shear strength is 2.1 ksf and the friction angle is set equal to 0° for the clayey soils underlying the cask storage pads."*

That is, friction forces were assumed to be 0 at this interface, since friction is calculated as the normal force multiplied by the tangent of the friction angle (tan $0° = 0$).

PFS provided an analysis that estimates the horizontal displacement of the pads, assuming they are supported directly on cohesionless soils with a friction angle of 30°. This analysis is presented on pp. 34-39 of Rev. 8 of Calculation 05996.02-G(B)-4 under the heading "Evaluation of Sliding on Deep Slip Surface Beneath Pads." This analysis uses a very conservative value of the friction angle (30°) to calculate the frictional resistance available to resist sliding along the base of the pad and a postulated cohesionless material. Conservatively ignoring any passive resistance acting on the side of the pad, it demonstrates that the maximum estimated horizontal displacement would be 2.2". The analysis shows that the resulting maximum horizontal displacements, if they were to occur due to the earthquake, would be of no safety consequence to the pads or the casks.

NRC Comment (1st bullet, 2nd paragraph)

A fundamental assumption in the PFS approach is that sufficient bonding, and shear transfer between clay and soil cement interfaces could be achieved by unspecified "construction techniques." However, the staff had informed PFS on April 18, 2001, that PFS would have to identify these "construction techniques" and demonstrate that the assumed bonds can be developed and sustained through the period of regulatory concern. PFS did not provide this information.

<u>PFS Response</u>

PFS has previously specified the construction techniques that could be used to assure that sufficient bonding and shear transfer between clay and soil cement interfaces could be achieved. Most of these are identified in SAR Section 2.6.4.11, "Techniques to Improve Subsurface Conditions." As indicated on p. 2.6-112 of the SAR:

> *"DeGroot [1] noted that increasing the time delay between placement of subsequent lifts decreases the bond strength. The nature of construction of soil cement is such that there will be occasions when the time delay will be greater than the time required for the soil cement to set. This will clearly be the case for construction of the concrete storage pads on top of the soil-cement surface, because it will take some period of time to form the pad, build the steel reinforcement, and pour the concrete. He noted that several techniques can be used to enhance the bond between these lifts to overcome this decrease in bond due to time delay. In these cases, more than sufficient bond can be obtained between layers of soil cement and between the set soil-cement surface and the underside of the cask storage pads by simply using a cement surface treatment.*

---

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

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

Further discussion of construction techniques is presented on p. 2.6-114 of the SAR:

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

With respect to the interface between the bottom of the soil cement and the in situ silty clay, p. 2.6-114 of the SAR continues:

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

Further definition of the construction requirements is provided in p. 2.6-67 of the SAR, which indicates:

"The soil cement will be mixed and compacted into the surface of the silty clay, providing a bond at the interface that will exceed the strength of the silty clay."

A review of the technical literature shows that the assumed bonds can be achieved. As indicated above, DeGroot has demonstrated that techniques are available that will enhance the bond between lifts of soil cement. These techniques should be equally effective when applied to the soils at the PFSF site. Further, as stated on p. 2.6-113 of the SAR, PFS will demonstrate by testing site-specific samples that the required interface strengths are available to resist sliding forces due to the earthquake event:

"Soil-Cement Mix and Procedure Development – The sliding forces due to the design basis ground motion will be resisted by bond between the base and sides of the foundation and the soil cement and by passive resistance of the soil cement acting against the vertical side of the foundation. The soil-cement mix will be designed and constructed to exceed the minimum shear resistance requirements. During the soil-cement design phase, direct shear testing will be conducted along manufactured soil-cement lift contacts and concrete contacts that represent anticipated field conditions. The direct shear testing, along with other standard soil-cement testing, will be used to confirm that adequate shear resistance and other strength requirements will be provided by the final soil-cement mix design."

and on p. 2.6-114 of the SAR:

"In addition to conventional quality control testing performed for soil-cement projects, direct shear testing will be performed on representative samples obtained from placed lift contacts to confirm design requirements are obtained. Sacrificial soil-cement lifts may be used to protect the soil-cement subgrade in the pad foundation areas."

PFS is developing the soil-cement mix design using standard industry practice. This effort includes performing laboratory testing of soils obtained from the site. This on-going laboratory testing is being performed in accordance with the requirements of Engineering Services Scope of Work (ESSOW) for Laboratory Testing of Soil-Cement Mixes, ESSOW 05996.02-G010, Rev. 0. This program includes measuring gradations and Atterberg limits of samples of the near-surface soils obtained from the site. It includes testing of mixtures of these soils with varying amounts of cement and the testing of compacted specimens of soil-cement to determine moisture-density relationships, freeze/thaw and wet/dry characteristics, compressive and tensile strengths, and permeability of compacted soil-cement specimens. The entire laboratory testing program is being conducted in full compliance with the Quality Assurance (QA) Category I requirements of the ESSOW.

As part of this effort, PFS is performing so-called durability testing. These tests are performed in accordance with ASTM D559 and D560 to measure the durability of soil

cement specimens exposed to 12 cycles of wet/dry and freeze/thaw conditions. As indicated on p. 16 of PFS Calculation 05996.02-G(B)-04-8:

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

PFS is performing these tests to determine the amounts of cement and water that must be added to the site soils and to determine the compaction requirements to ensure that the soil cement will be durable and will withstand exposure to the elements. As indicated on p. 8 of PCA[2]:

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

And on p. 32:

> *"The principle requirement of a hardened soil-cement mixture is that it withstand exposure to the elements. Thus the primary basis of comparison of soil-cement mixtures is the cement content required to produce a mixture that will withstand the stresses induced by the wet-dry and freeze-thaw tests. The service record of projects in use proves the reliability both of the results based on these tests and of the criteria given below.*
>
> *The following criteria are based on considerable laboratory test data, on the performance of many projects in service, and on information obtained from the outdoor exposure of several thousand specimens. The use of these criteria will provide the minimum cement content required to produce hard, durable soil-cement, suitable for base-course construction of the highest quality.*
>
> 1. *Soil-cement losses during 12 cycles of either the wet-dry test or freeze-thaw test shall conform to the following limits:*
>   *Soil Groups A-1, A-2-4, A-2-5, and A-3, not over 14 percent;*
>   *Soil Groups A-2-6, A-2-7, A-4, and A-5, not over 10 percent;*
>   *Soil Groups A-6 and A-7, not over 7 percent.*
> 2. *Compressive strengths should increase both with age and with increases in cement content in the ranges of cement content producing results that meet requirement 1."*

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

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

Based on the above, it is PFS's position that it has adequately defined the measures that will be followed in the design and construction of the soil cement to assure that the assumed bonds can be sustained through the period of interest. PFS has committed to performing site-specific testing to confirm that the required interface strengths are available to resist sliding forces due to an earthquake. As indicated above, this testing will include direct shear tests to be performed in the laboratory in the near-term (pre-construction) during the soil-cement mix development to demonstrate that the required interface strengths can be achieved (p. 2.6-113 of SAR) and during construction to demonstrate that the required interface strengths are achieved (p. 2.6-114 of SAR). In addition, PFS has committed to augmenting this field testing program by performing additional site-specific testing of the strengths achieved at the interface between the bottom of the soil cement and the underlying soils.

NRC Comment (Item 2, Bullet 2)

- PFS identified shear failure through the "clayey" soils underneath the storage pads as the critical failure mode for stability against sliding during an earthquake event. The PFS analysis concluded that this was the critical failure mode based on the assumption that the shear strength at the other interfaces (pad/soil cement and soil cement/"clayey" soils) would be greater than the shear strength of the "clayey" soils. However, PFS did not adequately demonstrate the validity of this assumption.

<u>PFS Response</u>

As summarized above, PFS has provided detailed discussions and references to relevant published reports in the technical literature, all of which demonstrate that the shear strength at the other interfaces (pad/soil cement and soil cement/"clayey" soils) would be greater than the shear strength of the "clayey" soils. Further, as stated on pp. 2.6-113 and 2.6-114 of the SAR and in the previous response, PFS is committed to demonstrating by testing site-specific samples that the required interface strengths are available to resist sliding forces due to the earthquake event.

NRC Comment (Item 2, Bullet 3)

- For the CTB, PFS relied on the passive resistance of soil cement without adequately considering processes that may reduce the amount of applicable passive resistance. Specifically, the analysis did not consider: (1) the impact of freeze-thaw cycles on the top layer of the soil cement; (2) the potential for tensile or shear failure along lift interfaces within the soil cement layer; and (3) the need for a large pad displacement to induce the assumed passive resistance.

PFS Response

(1) The stability analysis of the Canister Transfer Building does not expressly address the impact of freeze-thaw cycles on the layer of soil cement within the frost zone or the potential for tensile failure along lift interfaces; however, those effects are being addressed by PFS in the on-going testing to design the soil-cement mix (see ESSOW 05996.02-G010, Rev. 0). As discussed above, PFS is currently performing industry-standard soil cement testing, including wet/dry and freeze/thaw tests, specifically to ensure that the soil-cement mix to be placed within the frost zone (30" below grade at the site) will be durable enough to withstand the rigors of wetting, drying, freezing, and thawing. These tests are being performed in accordance with ASTM D559 and D560 and will measure the durability of soil cement specimens exposed to 12 cycles of wet/dry and freeze/thaw conditions. PFS is designing the soil-cement in accordance with the criteria specified by the Portland Cement Association. Following these procedures will ensure that the resulting soil-cement will be able to withstand exposure to the elements, as indicated in the paragraphs from the PCA "Soil-Cement Laboratory Handbook," cited above.

As indicated on p. 9 of PFS Calculation 05996.02-G(B)-13-5, the unconfined compressive strength of the soil cement is assumed to be 250 psi. This is consistent with the soil cement mix proposed for use within the frost zone adjacent to the cask storage pads and is based on the assumption that the strength will be at least this value to obtain a soil cement mix design that will satisfy the durability requirements of the ASTM wet/dry and freeze/thaw tests.

Further, PFS will construct the soil cement using techniques that will ensure that the bond between the layers is stronger than the shear strength of the underlying clayey soils. These standard practices in design and construction of soil cement have resulted in successful application of soil cement to more extreme environmental considerations than are applicable for this project and, thus, they will ensure that tensile cracking or shear failure does not occur along these interfaces. Therefore, PFS is considering the effects of freeze/thaw cycles on the top layers of soil cement, for both the pad emplacement area and the soil cement surrounding the CTB.

(2) Although the sliding stability analysis of the CTB does not specifically refer to the potential for tensile or shear failure along lift interfaces within the soil cement layer, the analysis does indicate that the soil cement will be designed in accordance with the ASTM requirements for durability. There are also a number of references in the SAR (pp. 2.6-112 to 114) documenting PFS's commitment to construct the lift interfaces using techniques to ensure that the bond between soil cement layers is adequate to resist the sliding forces due to an earthquake.

(3) The stability analysis of the Canister Transfer Building does consider the horizontal displacements required to develop passive resistance from the soil cement. Pages 15, 16, & 44 of PFS Calculation 05996.02-G(B)-13-5 present the results of the sliding stability analysis of the Canister Transfer Building assuming that only half of the passive resistance due to the soil cement is available to resist sliding. This is standard practice in

geotechnical engineering analyses of sliding of structures where there is assurance that the material that is being relied upon to provide passive resistance will not be excavated. In this case, one-half of the passive resistance is used, specifically, to incorporate the effects of strain-compatibility between the horizontal strains required to develop shear strength along the base of the structure and the those required to develop a portion of the passive resistance along the side of the mat. This is stated on p. 16 of this calculation as follows:

> *"Lambe & Whitman (1969, p 165) indicates that little horizontal compression, ~0.5%, is required to reach half of full passive resistance for dense sands. The soil cement will be compacted to a dense state; therefore, assume that half of the total passive resistance is available to resist sliding of the building. Note, 0.5% of the 5 ft height of the mat + 1.5-ft deep key = 0.005 x 6.5 ft x 12 in./ft = 0.39 in. Since there are no safety-related systems that would be severed or otherwise impacted by movements of this small magnitude, it is reasonable to use this passive thrust to resist sliding along with the resistance provided by the peak shear strength of the clayey soils enclosed within the perimeter key at the base of the mat."*

The effects of wall movement on wall pressure are defined in DM-7[3] (p. 7.2-60) as the ratio of horizontal displacement to the height of the wall to emulate the horizontal strain as defined in Lambe and Whitman. For stiff cohesive soils, the wall rotation or yield ratio, y/H, required to fully mobilize passive resistance is 0.02, or 2%.

Based on the shear modulus estimated from the shear wave velocity of the surficial silty clay/clayey silt, the horizontal displacement of the CTB subjected to the full horizontal earthquake load is calculated to be about 1.25 inches using the elastic solution of a buried horizontal rectangle subjected to shear in an elastic half-space. This horizontal displacement corresponds to a yield ratio, defined as horizontal displacement 4 height of wall, of 1.6% from translation of the 6.5 ft height of the CTB foundation mat adjacent to the soil cement. This yield ratio is larger than the yield ratio required to mobilize one half of full passive resistance for dense sand or stiff cohesive soils. However, this analysis ignores the build-up of passive resistance as the mat displaces on the elastic half-space. Recognizing that the resistance does gradually build as the foundation displaces horizontally, it is reasonable to expect that the resulting horizontal displacement will be somewhat reduced from the 1.25 inches calculated by the elastic solution. Even if the maximum horizontal displacement were to occur from an earthquake, there would be no safety consequence to the CTB since the building does not rely on any external "Important to Safety" connections.

Page 17 of PFS Calculation 05996.02-G(B)-13-5 discusses further the horizontal displacement to induce the assumed passive resistance. It states:

---

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

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

The results of the direct shear tests, presented as plots of shear stress vs horizontal displacement in Attachment 7 of Appendix 2A of the SAR (annotated copies are included in Attachment C of this calculation), illustrate that the residual strength of these soils is nearly equal to the peak strength for those specimens that were tested at confining stresses of 2 ksf. For example, for Sample U-1C from Boring C-2, at horizontal displacements of ~0.025" past the peak strength, there is ~1.5% reduction in the shear strength indicated. Note, the horizontal displacement of ~0.025" past the peak strength corresponds to a horizontal strain of ~1%, since the diameter of these specimens was 2.5". The results for Sample U-1AA from Boring CTB-S showed no decrease in shear strength following the peak at ~0.025" horizontal displacement, and Samples U-3B&C from Boring CTB-6 showed a decrease of ~5%. The specimens that were tested at confining stresses of 1 ksf all show reductions of ~20% at horizontal displacements of ~0.025" past the peak.

The final effective vertical stresses at the base of the Canister Transfer Building, σ'_v, are ~1.5 ksf, now that the mat has been changed to 240 ft x 279.5 ft. This value is approximately half-way between the confining stresses of 1 and 2 ksf used for several of the direct shear tests. The residual strength of the clayey soils beneath the building are expected to show reductions from the peak strength of ~12.5%, since the final effective stresses under the building are ~1.5 ksf. Therefore, based of these results, conservatively assume that the peak strength of the clayey soils beneath the soil cement layer underlying the pads is reduced by 20% to reach residual strength, to account for horizontal straining required to reach a strain applicable to the full passive resistance of the soil cement adjacent to the pad."

Therefore, the sliding stability analysis of the Canister Transfer Building properly considers, in the manner discussed above, the horizontal displacement required to induce the assumed passive resistance.