UNITED STATES OF AMERICA
NUCLEAR REGULATORY COMMISSION

BEFORE THE ATOMIC SAFETY AND LICENSING BOARD

In the Matter of:	)	Docket No. 72-22-ISFSI
	)	
PRIVATE FUEL STORAGE, LLC	)	ASLBP No. 97-732-02-ISFSI
(Independent Spent Fuel	)	
Storage Installation)	)	January 21, 2000

DECLARATION OF FARHANG OSTADAN, PH. D.

I, FARHANG OSTADAN, hereby declare under penalty of perjury and pursuant to 28 U.S.C. § 1746, that:

1. I hold a Ph.D. in civil engineering from the University of California at Berkeley. My curriculum vitae listing my qualifications, experience, training, and publications has already been filed in this proceeding. *See*, Exhibit No. 2 of the "State's Motion to Compel Applicant to Respond to State's Fifth Set of Discovery Requests" dated December 20, 1999.

2. I have fifteen years experience in dynamic analysis and seismic safety evaluation of above and underground structures and subsurface materials. I co-developed and implemented SASSI, a system for seismic soil-structure interaction analysis currently in use by the industry worldwide. I also developed a method for liquefaction hazard analysis currently in use for critical facilities in the United States.

3. I have participated in seismic studies and review of numerous

nuclear structures, including Diablo Canyon Nuclear Station and the NRC/EPRI large scale seismic experiment in Lotung, Taiwan. I have published numerous papers in the area of soil structure interaction and seismic design.

4. I have read the materials filed by PFS in support of its Motion for Summary Disposition of Contention GG, including the "Safety Analysis Report for the TranStor Storage Cask System," rev. B; the "TranStor Storage Cask Seismic Stability Analysis for PFS Site," July 24, 1997 ("Private Utility Fuel Storage Project Cask Seismic Tipover Analysis," prepared for Sierra Nuclear Corporation by Advent Engineering Services, Inc. (hereinafter "Advent Report")); the "PFSF Site Specific Cask Stability Analysis for the TranStor Storage Cask," September 23, 1999; and the "TranStor Dynamic Response to 2000 Year Return Seismic Event, Holtec Report No. HI-992295." I am familiar with the circumstances and materials in this case as they relate to Contention GG, including PFS's Safety Analysis Report. I am also familiar with and have reviewed the documents that PFS has provided to the State of Utah concerning Utah Contention GG, PFS's responses to Discovery Requests submitted by the State, and PFS's responses to the NRC Staff's Requests for Additional Information.

5. The Applicant has performed a series of simple nonlinear time history analyses in which the interaction between the cask and the foundation pad is modeled by frictional elements. The coefficient of friction was changed in successive analyses from 0.20 to 0.80. Soler Dec., Sum. Disp. at ¶ 9.

6. Dr. Alan Soler states that the "coefficient of friction" is a property

associated with a contact point between two surfaces and the value of the coefficient is dependent on the characteristics of the two materials at the interface contact point. Soler Dec., Sum. Disp. at ¶ 7. In a declaration supporting the Applicant's Response to State of Utah's Motion to Compel Applicant to Respond to State's Fifth Set of Discovery Requests, Dr. Solar claims that the coefficient of friction is independent of the friction forces. Solar Dec., Resp. Mo. Compel at ¶ 10. However, the coefficient of friction is only independent of friction forces under certain circumstances.

7. In justifying that the coefficient of friction is independent of friction forces, Dr. Soler must assume that the contact surface between the bottom of the cask and top of the foundation will remain intact after loading the casks on the pad and during the seismic excitation which effectively implies that the concrete pad is rigid under both static and dynamic loading. This assumption led the Applicant to the simplifying assumption for the dynamic analysis of the cask by using simple frictional elements at the contact points.

8. However, using the Applicant's parameters, including the coefficient of the subgrade reaction of 2.75 kips/ft3 (SAR, Rev. 8 at 2.6-35) and the pad dimensions (SAR, Rev. 8, at 2.6-87), and the relationship described in "Foundation Analysis and Design," Fourth edition, Joseph E. Bowels, McGraw Hill Company, 1988, Section 9.7, hereto attached as Exhibit A, to distinguish a flexible versus a rigid mat, I have calculated that the pad will not be rigid and, in fact, will deform when subjected to cask loading. Thus, Dr. Soler's assumption that the cask pad is rigid is incorrect.

Moreover, because the pad is flexible, the coefficient of friction is dependent on friction forces and will be affected at the contact points between the cask and the pad.

9. Under dynamic loading, the dynamic properties of a flexible pad are different from those of a rigid pad.[1] The flexible behavior of the foundation pad will amplify under the inertia of the casks on the pad. Thus, the coefficient of friction will not be constant across the pad and the Applicant's analysis of uniform coefficient of friction will not bound the actual behavior of the casks and the pad.

10. It is also possible that the casks on the pad could develop a cold bonding over time. The cold bonding causes a contact condition between the cask and the pad that is not covered by the highest coefficient of friction used by the Applicant. Additionally, the effect of the cold bonding is not necessarily the same as the hinge condition that the Applicant assumed in the previous Advent analysis.[2] The bonding may break during seismic shaking in a nonuniform pattern depending on the contact stresses causing a nonuniform contact condition between the cask and the pad.

11. Within the context of Contention GG and the modeling technique used by the Applicant and considering the realistic and flexible behavior of the pad under

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[1] An excellent comparison of the dynamic properties of rigid versus flexible foundation is presented by Iguchi and Luco in "Dynamic response of Flexible Rectangular Foundations on an Elastic Halfspace," Journal of Earthquake Engineering and Structural dynamics, 1981, Vol. 9.

[2] The Advent Report assumed that the cask was analytically pinned at one edge and did not consider the coefficients of friction. Soler Dec., Sum. Disp. at ¶ 4.

both the static and dynamic loading, it is my opinion that the Holtec 2000 analysis relied upon by the Applicant still fails to consider variation of coefficient of friction over the surface of the pad and the shift from static case to kinetic case.

12. This Declaration has been prepared in support of the State of Utah's Response to Applicant's Motion for Summary Disposition of Contention Utah GG, and the State's accompanying Statement of Material Facts, and is true and correct to the best of my knowledge and belief.

DATED this January 21, 2000.

Farhang Ostadan, Ph.D.

FOUNDATION ANALYSIS AND DESIGN

Fourth Edition

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9-7 CLASSICAL SOLUTION OF BEAM ON ELASTIC FOUNDATION

When flexural rigidity of the footing is taken into account, a solution is used that is based on some form of a beam on an elastic foundation. This may be of the classical Winkler solution of about 1867 in which the foundation is considered as a bed of springs ("Winkler foundation") or a finite-element procedure of the next section.

The classical solutions, being of closed form, are not as general in application as the finite-element method. The basic differential equation is (see Fig. 9-10)

$$EI\frac{d^4y}{dx^4} = q = -k'_s y \qquad (9\text{-}11)$$

TABLE 9-2 Closed-form solutions of infinite beam on elastic foundation (Fig. 9-10*a*)

Concentrated load at end	Moment at end
$y = \frac{2V_1\lambda}{k_s'} D_{\lambda x}$	$y = \frac{-2M_1\lambda^2}{k_s'} C_{\lambda x}$
$\theta = \frac{-2V_1\lambda^2}{k_s'} A_{\lambda x}$	$\theta = \frac{4M_1\lambda^3}{k_s'} D_{\lambda x}$
$M = \frac{-V_1}{\lambda} B_{\lambda x}$	$M = M_1 A_{\lambda x}$
$Q = -V_1 C_{\lambda x}$	$Q = -2M_1\lambda B_{\lambda x}$
Concentrated load at center	**Moment at center**
$y = \frac{P\lambda}{2k_s'} A_{\lambda x}$	$y = \frac{M_0\lambda^2}{k_s'} B_{\lambda x}$ deflection
$\theta = \frac{-P\lambda^2}{k_s'} B_{\lambda x}$	$\theta = \frac{M_0\lambda^3}{k_s'} C_{\lambda x}$ slope
$M = \frac{P}{4\lambda} C_{\lambda x}$	$M = \frac{M_0}{2} D_{\lambda x}$ moment
$Q = \frac{-P}{2} D_{\lambda x}$	$Q = \frac{-M_0}{2} A_{\lambda x}$ shear

The A, B, C, and D coefficients are:

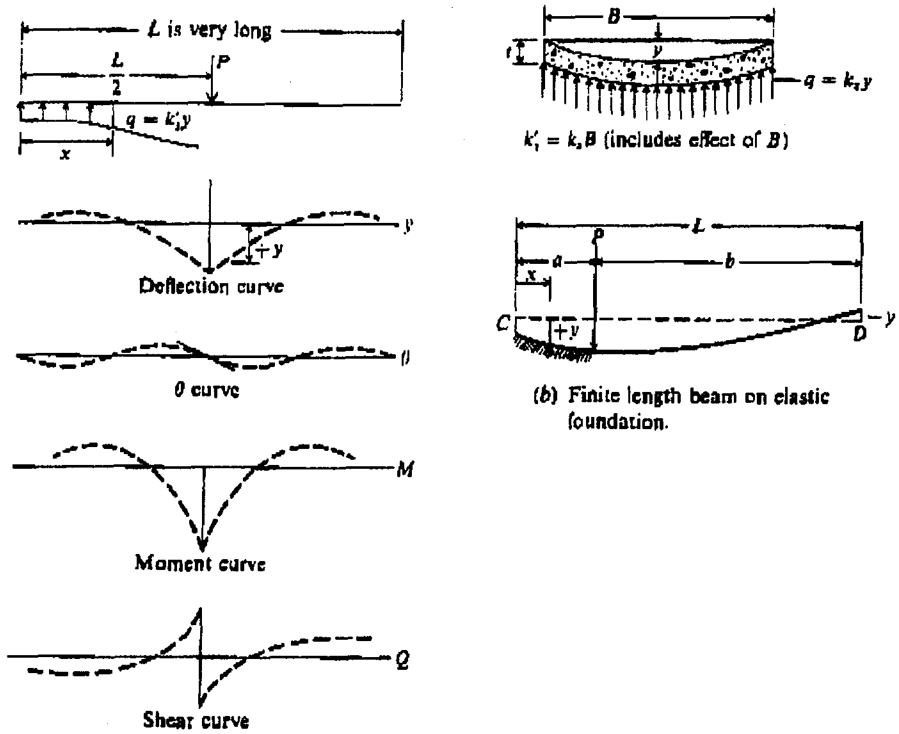
$$A_{\lambda x} = e^{-\lambda x}(\cos \lambda x + \sin \lambda x)$$
$$B_{\lambda x} = e^{-\lambda x} \sin \lambda x$$
$$C_{\lambda x} = e^{-\lambda x}(\cos \lambda x - \sin \lambda x)$$
$$D_{\lambda x} = e^{-\lambda x} \cos \lambda x$$

where $k_s' = k_s B$. In solving the equations, a variable is introduced:

$$\lambda = \sqrt[4]{\frac{k_s'}{4EI}} \qquad \text{or} \qquad \lambda L = \sqrt[4]{\frac{k_s' L^4}{4EI}}$$

Table 9-2 gives the closed-form solution of the basic differential equations for several loadings shown in Fig. 9-10 utilizing the Winkler concept. It is convenient to express the trigonometric portion of the solutions separately as in the bottom of Table 9-2.

Hetenyi (1946) developed equations for a load at any point along a beam (see Fig. 9-10*b*) measured from the left end as follows:



(a) Infinite length beam on an elastic foundation.

(b) Finite length beam on elastic foundation.

FIGURE 9-10 Beam on elastic foundation.

$$y = \frac{P\lambda}{k_s'(\sinh^2 \lambda L - \sin^2 \lambda L)} \{2 \cosh \lambda x \cos \lambda x (\sinh \lambda L \cos \lambda a \cosh \lambda b - \sin \lambda L \cosh \lambda a \cos \lambda b) + (\cosh \lambda x \sin \lambda x + \sinh \lambda x \cos \lambda x) [\sinh \lambda L (\sin \lambda a \cosh \lambda b - \cos \lambda a \sinh \lambda b) + \sin \lambda L (\sinh \lambda a \cos \lambda b - \cosh \lambda a \sin \lambda b)]\} \quad (9\text{-}12)$$

$$M = \frac{P}{2\lambda(\sinh^2 \lambda L - \sin^2 \lambda L)} \{2 \sin \lambda x \sin \lambda x (\sinh \lambda L \cos \lambda a \cosh \lambda b - \sin \lambda L \cosh \lambda a \cos \lambda b) + (\cosh \lambda x \sin \lambda x - \sinh \lambda x \cos \lambda x) \times [\sinh \lambda L (\sin \lambda a \cosh \lambda b - \cos \lambda a \sinh \lambda b) + \sin \lambda L (\sinh \lambda a \cos \lambda b - \cosh \lambda a \sin \lambda b)]\} \quad (9\text{-}13)$$

$$Q = \frac{P}{\sinh^2 \lambda L - \sin^2 \lambda L} \{(\cosh \lambda x \sin \lambda x + \sinh \lambda x \cos \lambda x)$$
$$\times (\sinh \lambda L \cos \lambda a \cosh \lambda b - \sin \lambda L \cosh \lambda a \cos \lambda b)$$
$$+ \sinh \lambda x \sin \lambda x [\sinh \lambda L (\sin \lambda a \cosh \lambda b - \cos \lambda a \sinh \lambda b)$$
$$+ \sin \lambda L (\sinh \lambda a \cos \lambda b - \cosh \lambda a \sin \lambda b)]\}$$

The equation for the slope θ of the beam at any point is not presented is of little value in the design of a footing. The value of x to use in the equa from the end of the beam to the point for which the deflection, moment, or desired. If x is less than the distance a, use the equations as given, and measure x from C. If x is larger than a, replace a with b in the equations, and measure x from D (Fig. 9-10b). These equations may be rewritten as

$$y = \frac{P\lambda}{k_s'} A' \qquad M = \frac{P}{2\lambda} B' \qquad \text{and} \quad Q = PC'$$

where the coefficients A', B', and C' are the values for the hyperbolic and trigonometric remainder of Eqs. (9-12) to (9-14).

It has been proposed that one could use λL previously defined to determine if a foundation should be analyzed on the basis of the conventional rigid procedure or as a beam on an elastic foundation.

Rigid members:	$\lambda L < \frac{\pi}{4}$	(bending not influenced much by k_s)
Flexible members:	$\lambda L > \pi$	(bending heavily localized)

The author has found the above criteria of limited application because of the influence of number of loads and their locations on the member.

The classical solution presented here has several distinct disadvantages over the finite-element solution presented in the next section, such as:

1. Assumes weightless beam (but weight will be a factor when footing tends to separate from the soil).
2. Difficult to remove soil effect when footing tends to separate from soil.
3. Difficult to account for boundary conditions of known rotation or deflection at selected points.
4. Difficult to apply multiple types of loads to a footing.
5. Difficult to change footing properties of I, D, and B.
6. Difficult to allow for change in subgrade reaction along footing.

Although the disadvantages are substantial some engineers prefer the classical beam-on-elastic-foundation approach over discrete element analyses. Rarely, the classical approach may be a better model than a discrete element analysis so it is worthwhile to have access to this method of solution.