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STRUCTURAL ANALYSES OF THE FUEL RECEIVING STATION POOL AT THE NUCLEAR FUEL SERVICE REPROCESSING PLANT, WEST VALLEY, NEW YORK

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ABSTRACT

At the request of the Nuclear Regulatory Commission (NRC), a structural assessment was done of the fuel receiving station (FRS) pool at the reprocessing plant operated by Nuclear Fuel Services, Inc., at West Valley, N.Y. The FRS is a pool structure and enclosing building constructed in 1966 for storing spent nuclear fuel. The enclosing building was not analyzed. We determined the pool structure's responses to operating loads, seismic excitation, and an accidentally dropped cask. We identified the locations in the FRS pool where structural strength would be exceeded in the event of an earthquake of 0.2 g maximum ground acceleration or an accident in which a cask dropped from the maximum height of the crane hook used to maneuver it.

INTRODUCTION

At the request of the Nuclear Regulatory Commission (NRC), the Lawrence Livermore Laboratory (LLL) is providing a structural assessment of the fuel reprocessing facility operated by Nuclear Fuel Services, Inc., at West Valley, N.Y. The facility was built in 1966. This report contains our analysis of the fuel receiving station (FRS) portion of the facility and is limited to the FRS pool.

Spent nuclear fuel arriving at the plant is received and stored at the FRS while waiting to be reprocessed. The FRS consists of two parts, a pool and an enclosing building. The FRS pool is an embedded structure consisting of a cask unloading cell, fuel storage cell, and water treatment cell. The enclosing building covers the pool and work areas. Spent fuel arrives in heavy shipping casks, which are decontaminated in an area adjacent to the FRS and then maneuvered by crane into the cask unloading cell. The cask is then opened, and the fuel elements are transferred from the cask into storage canisters, which provide support for handling but allow water to circulate around the elements. The canisters are then moved by crane into the fuel storage cell where they are placed on specially designed racks. The fuel elements remain fully submerged in water for radiation shielding and cooling.

The assessment of the facility consisted of structural familiarization, construction and analysis of a mathematical model, development of limiting criteria,* and formulation of results and conclusions.

We performed analyses to determine the effects of operating loads, seismic excitation, and accidental cask drop loads on the reinforced concrete pool. The techniques for dynamic analysis of embedded structures are still being developed. Therefore, seismic loads were analyzed using equivalent static

* Limiting criteria define the level of stress, deflection, or degradation that initiates structural distress.

methods. A finite element technique was used to do the analyses. Known factors of safety were removed. We compared combinations of our analyses' results to limiting criteria to determine the level of ground acceleration (up to 0.2 g) that the structure could withstand.

Throughout the project we used our judgment extensively to make modeling decisions and to select workable limiting criteria. We relied on past experience, published literature, and discussion with the NRC to assist in making these decisions.

SUMMARY

Results of the seismic and operating load analyses indicated that the only region of structural distress was in the upper east corner of the north wall of the fuel storage cell. This region, which exceeded our limiting criteria at 0.16 g, could crack and leak. Our estimate is that leakage would be above the soil and into the building enclosing the FRS pool.

Analyses of the cask-drop accident indicated that any presently used shipping cask dropped from the maximum crane hook height will puncture the cask unloading cell floor.

FUEL RECEIVING STATION DESCRIPTION

The reprocessing plant operated by Nuclear Fuel Service, Inc., is located south of Buffalo, N.Y., on the Western New York State Nuclear Service Center site near West Valley. The location of the FRS with respect to the NFS process building is shown in Fig. 1.

The FRS consists of two separate structures, the fuel pool and the enclosing building. The fuel pool is a reinforced concrete embedded structure, and the enclosing building is a braced steel frame structure with metal covering. Each structure has an independent foundation. We examined only the fuel pool in this study.

FIG. 1. Plan view of the reprocessing facility showing the location of the fuel receiving station (FRS).

STRUCTURAL CONFIGURATION

The FRS pool consists of three cells as shown in Fig. 2. For convenience of discussion, the walls are designated by the letters shown in Figs. 3 and 4. The fuel storage cell is filled with water to a depth of 28 ft. The cask unloading cell floor has two levels. The upper level is even with the fuel storage cell floor and the lower level is 16 ft lower. This cell is normally filled with water, but occasionally it is drained for clean-up. At such times the gate in wall F/C is closed. The water treatment cell contains treatment equipment and is not filled with water. The exterior walls of the FRS pool are tapered as indicated in Fig. 5, and all interior walls are of constant thickness.

FIG. 2. Plan view of the FRS pool showing rack area and close-up of canister arrangement.

FIG. 3. Three-dimensional view of the FRS pool showing wall designations.

FIG. 4. Detailed view of water treatment cell walls W/C, W/H, and W/V.

FIG. 5. Section view of the FRS pool showing storage rack area and tapered exterior wall.

FUEL RACKS

The FRS pool holds 1092 canisters on 42 aluminum racks, which occupy a region 75 ft long, 36 ft wide, and 17 ft high as indicated in Section IV-2-9 of Ref. 1. Figure 6 shows a typical fuel rack. Each canister weighs approximately 1200 lb when filled with fuel and is supported by two rack beams as shown in Fig. 7.

Our past experience² indicates that the water within, and surrounding, fuel canisters occupies approximately one-half the volume enclosed by the racks. Because the canisters in the FRS pool are arranged closely together, it is reasonable to assume that this water is confined to translate with the canisters. Consequently, considering the weight of the canisters, water, and mounting brackets, we found the density of the rack volume to be close to that used in Ref. 2, i.e., 128 pcf. Therefore, 128 pcf was also used for this analysis for consistency with previous investigations involving spent-fuel storage facilities.

FIG. 6. One of 42 fuel racks spaced 1.75 ft apart (Source: Bechtel Corp. drawing 1A-M-7 received from the NRC).

FIG. 7. Detailed drawings of fuel canister and support system.

SOIL PROPERTIES

The soil at the site is of glacial origin with deposits consisting of mixtures of clay and silt. The general soil profile is shown in Fig. 8. For this analysis, the important soil parameter is in situ density. Borings 16, 20, and 22, shown in Fig. 9, are the closest ones to the FRS for which soil densities are reported.³ More credibility is given to the densities reported from boring 16 since it is the closest to the FRS pool.

FIG. 8. Section view of the FRS showing general soil profile.

FIG. 9. Locations of borings near the PRS.

Table 1 lists the in situ soil density as a function of depth for each boring. As a conservative estimate, the resulting density at each 5-ft increment of depth was taken as either the average for the three borings or that of boring 16, whichever is higher. The resulting density distribution was then idealized as a two-layered system as shown in Table 1, with the top layer having a density of 118 pcf and the bottom layer having a density of 142 pcf.

We believe that liquefaction should not occur at the site because, as stated in the Safety Analysis Report,¹ the fine-grained soil has a large enough clay fraction to be cohesive.

TABLE 1. Soil density vs depth near the FRS.

Depth, ft	Soil densities from borings, pcf			Resulting density, pcf	Idealized density, pcf
	16	20	22		
0	105.3	194.4	111.3	107.0	
5	127.5	120.5	128.7	127.5	118
10	136.6	143.0	137.5	139.0	
15	145.4	134.4	155.4	145.4	
20	147.2	135.4	139.9	147.2	
25	138.6	134.5	132.2	138.6	142
30	139.9	133.1	141.6	139.9	
35	146.7	123.2	126.1	146.7	

STRUCTURAL LOADING

The following loading conditions were considered for our assessment of the FRS pool:

- Operating loads due to
 - static soil pressure
 - hydrostatic pressure
 - thermal gradient;
- Seismically induced loads due to
 - wall inertia
 - fuel rack inertia
 - soil response
 - hydrodynamic action;
- Impactive loads due to
 - cask-drop accident .

The methods used to determine the magnitude and distribution of these loads are explained below.

OPERATING LOADS

Static Soil and Hydrostatic Pressures

The earth pressure at rest is taken as the static soil pressure given by

$$P = \gamma h \left(\frac{1}{1-\nu} \right) = \gamma h K_0$$

where,

γ = soil density

h = soil depth

ν = Poisson's ratio, assumed 0.3

K_0 = coefficient of earth pressure at rest ($K_0 = 0.43$).

For our idealized two-layer soil system the external pressure profile is shown in Fig. 10 (Note: In this and subsequent drawings, external walls are sketched as having uniform thickness although they are actually tapered.).

FIG. 10. Static soil pressure.

Walls of cells containing water are subjected to an internal pressure distribution resulting from a fluid density of 62.4 pcf. This pressure distribution is shown in Fig. 11.

FIG. 11. Hydrostatic pressure.

Thermal Gradient

Based on discussions with the NRC, we assumed a pool water temperature of 85°F and an outside soil temperature of 50°F. The resulting thermal gradient is shown in Fig. 12.

FIG. 12. Thermal gradient through a wall.

SEISMIC LOADS

Seismic input needed for analysis of the FRS pool consisted of peak ground acceleration and relative spectral displacements below 0.35 Hz. We used a peak horizontal acceleration of 0.2 g and a peak vertical acceleration of 0.14 g. This is consistent with other analyses for the site.^{4,5} We chose NRC Regulatory Guide 1.60⁶ design response spectrum to represent the spectral shape as shown in Fig. 13.

FIG. 13. Horizontal design response spectra, scaled to 1-g horizontal ground acceleration (from Ref. 6).

Wall Inertia

For a fairly rigid structure completely embedded in the ground, such as the FRS pool, there is insignificant dynamic amplification. Therefore, the inertial load is simply the wall mass times the peak ground acceleration. We determined the wall mass using a reinforced concrete density of 150 pcf. The loading is shown in Fig. 14.

FIG. 14. Inertia loading for a typical exterior wall.

Fuel Rack Inertia

The fuel rack inertia loading was based on the density of the rack and enclosed water, 128 pcf. We assumed that wall F/N carries the entire horizontal load under north-south excitation. This is the only case considered since it is the only direction resulting in a transverse load to any pool wall. The distribution is shown in Fig. 15.

FIG. 15. Rack inertia produced by horizontal acceleration.

Dynamic Soil Pressure

The dynamic soil pressure due to horizontal acceleration is determined with the Mononobe-Okabe (M-O) theory^{7,8,9} increased by a factor of 2.0. The factor of 2.0 was found applicable through an earlier LLL study² as well as through work on embedded structures by H. B. Seed at the University of California at Berkeley and N. Newmark and W. Hall at the University of Illinois at Urbana-Champaign.

The pressure profile is triangular with the maximum at the top given by

$$P_{\max} = \frac{3}{2} \gamma H \frac{a}{g}$$

where

- γ = soil density
- H = distance from bottom of wall
- a = maximum horizontal ground acceleration
- g = gravitational acceleration.

For the idealized two-layer soil system the dynamic soil pressure profile is shown in Fig. 16 for a 0.2 g maximum horizontal ground acceleration.

FIG. 16. Dynamic soil pressure produced by horizontal acceleration.

The soil pressure due to a maximum vertical ground acceleration of 0.14 g is taken as the earth pressure at rest multiplied by 0.14. The pressure distribution is triangular with the maximum at the bottom given by

$$P_{\max} = \gamma h \frac{a}{g} K_0$$

where

γ = soil density

h = wall embedded depth

a = maximum vertical ground acceleration

g = gravitational acceleration

K_0 = coefficient of earth pressure at rest.

The soil pressure due to the vertical earthquake component is shown in Fig. 17.

FIG. 17. Dynamic soil pressure produced by vertical acceleration.

Hydrodynamic Pressure

The hydrodynamic pressure results from two effects, the confined fluid (impulsive pressure) and the sloshing fluid (convective pressure). Both are confirmed in Ref. 2 to be adequately described by Housner's theory.^{10,11} Because the theory is so well documented in the references, the numerous equations and procedures will not be repeated here. The impulsive pressure depends directly on the horizontal acceleration, and the convective pressure results from water sloshing. Because the FRS pool is large, the water sloshes at low frequencies, below 0.35 Hz. For a fairly rigid structure such as the FRS pool, a load of such low frequency constitutes essentially a static load.

For the walls of the fuel storage cell, Housner's theory is applied using the entire water depth if no fuel canisters are present. If the cell is filled to capacity, the theory is applied only to the water above the fuel.

The water between the canisters is assumed constrained to translate with the canisters and, therefore, does not participate in creating the impulsive and convective pressures. Instead, it is included in the fuel rack inertial load (density = 128 pcf). The impulsive and convective water pressure distributions are shown in Fig. 18.

The dynamic water pressure due to a vertical acceleration of 0.14 g is the hydrostatic pressure multiplied by 0.14, as shown in Fig. 19.

IMPACTIVE LOADS: CASK-DROP ACCIDENT

As part of this analysis, we examined the response of the cask unloading cell floor to the impact of an accidentally dropped cask. Such an accident can take place while the cask is being maneuvered into, or out of, the cell. LLL and NRC mutually decided that cask impact on other locations, such as the upper edge of the cell and the edge of the step of the floor, were not to be included in this investigation.

Two casks are considered, truck cask NLI 1/2 and rail cask NLI 10/24. The dimensions, weights, and assumed drop heights are listed in Fig. 20. We and NRC attempted to determine the impact area sizes and the maximum drop heights; however, the information was difficult to obtain. Therefore, we conservatively assumed that the cask had a flat end with an impact area the same size and shape as the cask end. The drop height was assumed to be from the maximum elevation that the crane could hoist the cask.

a) No fuel in cell

Impulsive water

Convective water

b) Fuel in cell

Impulsive water

Convective water

FIG. 18. Impulsive and convective water pressure produced by horizontal acceleration.

FIG. 19. Dynamic water pressure produced by vertical acceleration.

	Casks examined	
	NLI 10/24	NLI 1/2
Weight, lb	194,000	49,100
Diam., ft	7.5	6.25
Length, ft	17.0	20.0
h_L , ft	53.5	50.5
h_U , ft	37.5	34.5

FIG. 20. Schematic and applicable data for analysis of possible cask-drop accidents.

ANALYSES FOR SEISMIC AND OPERATING LOADS

STRUCTURAL MODELS

A preliminary analysis was performed at the request of the NRC to determine if the FRS pool had a reasonable chance of resisting the seismic loading specified for the site. For this assessment, simple models were used to study the walls deemed most critical. The walls are discussed below.

- Wall F/N. This wall resists the fuel rack inertial load. Because the wall has a large length-to-height ratio, its middle can be modeled as a cantilever beam.
- Wall W/N. Soil pressure bears on the exterior face of this wall, while no counterbalancing pressure exists on the interior face. Wall W/N was modeled as a plate with fixed bottom and side edges. The thin plate theory bending moment coefficients tabulated by Moody¹² were used to determine the bending moments produced by the loads.
- Wall W/C. Water pressure bears on the south face of this wall, while no counterbalancing pressure exists on the north face. Wall W/C is structurally complex and was analyzed using the SAP4 computer code.¹³ The analytical model consists of plate finite elements.

These walls and the models used are shown in Fig. 21. The load cases considered were the seismic loads and static pressures only, as defined in the next section, "Detailed Analyses."

A refined three-dimensional finite element model using SAP4 plate elements was developed for the detailed analysis as shown in Fig. 22. The tapered exterior walls are modeled in steps by changing the plate thicknesses. The gate in wall F/C was modeled as shown in Fig. 22. These elements were assigned a negligible stiffness for an open gate, and the normal reinforced concrete stiffness for a closed gate. The floor at 29 ft depth was assumed to be rigid and fixed, and was represented by fixed boundary conditions.

FIG. 21. Simple models of three critical walls used in a preliminary analysis to assess the pool's ability to resist the seismic loading specified for the site.

FIG. 22. A three-dimensional finite element model of the FRS pool used for the SAP4 detailed analysis.

DETAILED ANALYSES

Sixteen load cases were assessed using the SAP4 computer code. These cases are shown in Fig. 23 a-e and described in Table 2.

The thermal analysis which was done to qualitatively determine the effect of thermal gradient loading, assumed a constant gradient of 35°F over the entire wall. It was based on an elastic analysis using the gross concrete section. The thermal analysis was unrefined and based on very conservative assumptions.

FIG. 23a. Load cases for static soil and hydrostatic pressures.

FIG. 23b. Thermal gradient load cases.

FIG. 23c. Dynamic soil pressure load cases.

P_{DR} = dynamic rack
plus wall inertia

P_{DW} = dynamic water pressure
plus wall inertia

FIG. 23d. Load cases for dynamic water pressure and dynamic rack plus wall inertia (north-south motions).

FIG. 23e. Load cases for dynamic water pressure, wall and rack inertia (east-west motions).

TABLE 2. Sixteen load cases.

Load case	Description	<u>Cask unloading cell</u>		<u>Fuel storage cell</u>	
		Water	No water	Fuel	No fuel
1	Static soil and water pressure		X		X
2	Static soil and water pressure	X			X
3	Thermal gradient		X		X
4	Thermal gradient	X			X
5	Dynamic soil on east and west walls	X			X
6	Dynamic soil on south wall		X		X
7	Dynamic soil on north walls	X			X
8	Dynamic soil on south wall	X			X
9	Dynamic water and wall. N-S motions		X		X
10	Dynamic water and wall. N-S motions		X	X	
11	Dynamic water and wall. N-S motions	X			X
12	Dynamic water and wall. N-S motions	X		X	
13	Dynamic water and wall. E-W motions		X		X
14	Dynamic water and wall. E-W motions		X	X	
15	Dynamic water and wall. E-W motions	X			X
16	Dynamic water and wall. E-W motions	X		X	

Seismic loads can be applied in either direction on the walls with the restriction that resultant soil and water pressures can only be compressive. These cases were then combined either directly or by the square root of the sum-of-the-squares (SRSS) method depending on possible phase differences. The load combination procedure is shown in Fig. 24. Convective water pressure can be considered a static load because the period associated with it is so long. However, the convective water pressure magnitude is so small that the combination method used for it is not significant.

The responses to north-south and east-west excitations were considered to be independent except at the corners of the pool where the two components were combined by the SRSS method. The directions of the seismic loads were varied to determine the worst case conditions, which were those resulting in the largest lateral loading on the walls. Since lateral loading governed, wall dead load and vertical wall inertia were not considered. This was a reasonable assumption because small axial compressive loads increase the ultimate moment capacity of a reinforced concrete section.

The worst case loading conditions for the critical elements were used to calculate the resulting moments. The applied moments were then compared to the capacity of the concrete sections.

FIG. 24. Combination procedure for seismic and operating loads.

The moment capacities of the concrete sections are determined by the American Concrete Institute formula.¹⁴

$$M_u = \phi \rho t_y b d^2 \left(1 - 0.59 \rho \frac{t_y}{t'_c} \right),$$

where

- $\phi = 1.0$, we assumed no strength reduction
- ρ = area of steel divided by area of concrete
- t_y = yield strength of steel, 60,000 psi
- $b = 1$ ft, for a unit foot of wall length
- d = wall depth
- t'_c = concrete strength, 3000 psi.

The value of ρ was checked for all walls and was found in all cases to be significantly below the balanced reinforcement condition.

RESULTS

The thermally induced bending moments calculated by the crude analysis exceeded the moment capacities throughout by a factor ranging from 0.8 to 1.5. The most severely stressed exterior walls were F/N and C/E. Because of wall-to-wall interactive effects, interior walls F/C, W/F, and W/C were also highly stressed. We believe these results are overly conservative for the following reasons:

- The actual thermal gradient will most likely not be constant over the wall height and will be less than that assumed because the top of the wall is inside the building and the bottom of the wall is insulated by virtue of its depth of burial.
- The water temperature increased gradually as fuel was added over a period of years, so that the concrete had time to stress relax. Our analysis assumed a time independent response and did not include stress relaxation.

- Thermal stresses are self limiting. As the thermal strains increase, the concrete section cracks and stress is relieved. Our analysis did not account for this since the transformed section must be used, rather than the gross section. These transformed section properties would have temporal and spatial variations. This level of analysis is not warranted due to the many uncertainties involved.

Based on our judgment and experience, these stresses will not approach the levels predicted. Refined analysis appears unwarranted until there is better understanding of the actual behavior of concrete structures under thermal stress. Therefore, we did not include these stresses in the load combinations.

Results from the preliminary and detailed analyses are shown in Table 3. These results are for critical wall locations and represent the worst case load combination as shown. The threshold maximum ground acceleration is 0.16 g occurring in the upper east corner of wall F/N. Both the preliminary and detailed analyses revealed a critical location in this region.

Although some differences are apparent between preliminary and detailed analyses, the overall trends are in general agreement. The most notable difference is at the top corners of wall W/N. We believe that the difference results primarily from the wall-to-wall interaction that was considered in the detailed analysis but not in the preliminary assessment.

CONCLUSIONS

The only region of structural distress found was the upper east corner of wall F/N. This region exceeded our limiting criteria at 0.16 g. This could cause cracking resulting in a leak in this region. Our estimate is that leakage would be above the soil and into the building enclosing the FRS pool.

TABLE 3. Comparison of results from preliminary and detailed analyses.

Wall locations (looking north)	Preliminary analysis				Detailed analysis			
	Moment capacity M_{ult} kip-ft/ft	Total applied moment M^* kip-ft/ft	M_{ult}/M	Threshold g level **	Total applied moment M^* kip-ft/ft	M_{ult}/M	Threshold g level **	Worst case load combinations
315.0	241.3	1.4	0.26	188.0	1.67	0.35	2,7,10
9.8	----- not considered -----				11.3	0.87	2,6,10,14
315.0	57.3	5.9	3.5	39.9	7.9	2.64	2,5,7,12,16
35.0 *** 9.8	70.3	0.50	0.074	8.77	1.11	0.23
64.0	27.9	2.3	1.54	27.9	2.30	1.54	2,5,12,16
35.0	23.4	1.5	0.68	20.2	1.74	0.94	

$$^*M = M_{static} + M_{seismic}$$

$$^{**} \text{Threshold g level} = \frac{M_{ult} - M_{static}}{M_{seismic}} \times 0.2 \text{ g.}$$

*** 35.0 kip-ft/ft used for preliminary analysis only (based on constant wall thickness).

ANALYSIS OF THE CASK-DROP ACCIDENT

ANALYTICAL PROCEDURE

Two analytical approaches were investigated. One used empirical formulas and the other used an energy approach.

Various experimentally derived empirical formulas for impact response are available.¹⁵ Unfortunately, such formulas are based on high velocity impact data, in the range of 500-2000 ft/sec, whereas the velocity of the cask impact is of the order of 50 ft/sec. None are available for this velocity range. Therefore, the applicability of these formulas to our case is questionable. Results from these formulas are inconsistent. Some of the predictions reveal a large margin of safety for our case, whereas others indicate that the floor thickness is significantly less than that required to prevent perforation. Consequently, we decided that the empirical approach was inconclusive for the cask drop problem and did not pursue it further.

Instead, we developed an energy approach to estimate the impact of the cask. In this approach the energy of the dropped cask is assumed to be completely absorbed by the cell floor and underlying soil. The underlying soil was modeled as an elastic foundation as shown in Fig. 25.

FIG. 25. Soil underlying the cell floor was modeled as an elastic foundation with spring constant K for analysis of the cask-drop accident.

The water drag force was expected to be small for the low velocity of the cask, and was, therefore, ignored as suggested by NRC. The cell floor consists of an upper level and a lower level, each approximately 12 x 26 ft in area (Fig. 20). Both levels were examined.

As an approximation, the soil spring constant is assumed to be given by the expression for a rectangular foundation displaced vertically,¹⁶

$$K_Z = \frac{G}{1-\nu} \beta_Z \sqrt{BL} ,$$

where

G = soil shear modulus (4000 kip/ft²)

ν = soil Poisson's ratio (0.3)

B = cell floor width (12 ft)

L = cell floor length (26 ft)

$\beta_Z = 2.2$ for $L/B = 2.2$.

The soil shear modulus and Poisson's ratio for the NFS site were provided by NRC. The soil foundation modulus, k , for our analysis is K_Z divided by the area 12 x 26 ft, i.e.,

$$k = \frac{K_Z}{BL}$$

$$k = 711.8 \text{ kip/ft}^3.$$

The impact contact area is between 30 ft² and 44 ft² for the two casks. This is small compared with 312 ft² for the areas of the upper and lower floor levels. Thus, as an approximation we determined the local spring rate at the impact area with an expression for a large plate on an elastic foundation subjected to a load over a small area,¹⁸

$$K_S = 8 k \left[\frac{E t^3}{12(1-\nu^2)k} \right]^{1/2} ,$$

where

E = elastic modulus for concrete (449,600 kip/ft²)

ν = Poisson's ratio for concrete (0.2)

t = slab thickness

k = soil foundation spring constant (711.8 kip/ft³).

The modulus for the concrete is determined by its compressive strength of 3000 psi.¹⁴ The Poisson's ratio ν is taken as 0.2. The slab thickness t is 2.5 ft for the lower level, and 3.25 ft for the upper level. The resulting values of K_s are

$K_{SL} = 166,700$ kip/ft, lower level

$K_{SU} = 247,000$ kip/ft, upper level.

These are the effective stiffnesses a cask would encounter if it impacts the floor of the cell.

The energy of impact is the weight of the cask W times the drop height h . For the values of W and h given in Fig. 20, the energies are as follows:

Cask	<u>Impact energy, kip-ft</u>	
	Lower floor	Upper floor
NLI 10/24	10,380	7,280
NLI 1/2	2,480	1,690

Assuming that the impact energy is completely absorbed by the floor and soil, $Wh = 1/2 K_s \delta^2$, where δ is the floor deflection. The following impact forces, $F = K_s \delta$, were determined using the K_{SL} and K_{SU} values presented above:

Cask	<u>Impact force, kip</u>	
	Lower floor	Upper floor

NLI 10/24	58,820	59,950
NLI 1/2	28,740	28,930

A rough estimate of the impact stress induced in the cask, shown below, was obtained by dividing the impact forces by the impact area, assuming the cask is flat ended.

Cask	<u>Rough estimate of impact stress in the cask, psi</u>	
	Lower floor	Upper floor

NLI 10/24	9,250	9,220
NLI 1/2	6,500	6,550

Based on these estimated stresses we assumed that none of the impact energy is absorbed by plastic deformation of the cask.

The expressions for the maximum shear and bending moment induced in the floor by the impact force is given by the expression for a large plate on an elastic foundation subjected to a load over a small area.¹⁷

$$V = F \left(1 - \frac{\pi r_o^2}{8 L_e^2} \right)$$

$$M = \frac{F (1+\nu)}{4\pi} \left[\ln \frac{L_e}{r_o} + 0.6159 \right]$$

$$L_e = \left[\frac{E t^3}{12 (1-\nu^2) k} \right]^{1/4},$$

where

V = maximum shear force

M = maximum bending moment

F = impact force

E = modulus of concrete (449,600 kip/ft²)

ν = Poisson's ratio of concrete (0.2)

L_e = effective length

r_o = radius of impact contact area

k = soil foundation spring constant (711.8 kip/ft³)

t = slab thickness.

The contact area radius r_o is 3.75 and 3.125 ft for casks NLI 10/24 and NLI 1/2 respectively. The slab thickness t is 2.5 and 3.25 ft for the lower and upper levels respectively. The results are shown in Table 4.

TABLE 4. Dropped-cask analysis results.

Cask	Lower level					
	V, kip	V _{ult} [*] kip	V/V _{ult}	M, kip-ft	M _{ult} ^{**} kip-ft	M/M _{ult}
NLI 10/24	47,654	4,646	10.3	5,518	62	89
NLI 1/2	28,716	3,872	7.4	3,196	62	52

	Upper level					
	V, kip	V _{ult} [*] kip	V/V _{ult}	M, kip-ft	M _{ult} [*] kip-ft	M/M _{ult}
NLI 10/24	59,959	6,040	9.9	6,754	162	42
NLI 1/2	28,908	5,033	5.7	3,763	162	23

* Shear capacity was determined, as an upper bound for comparison, from

$$V_{ult} = 10 \sqrt{f_c} \times \text{shear area.}$$

** Moment capacity was determined using the ACI formula¹⁴ for ultimate moment capacity.

CONCLUSIONS

The loading induced by a cask-drop accident exceeds the shear capacity of the floor by at least 5.7 times and exceeds the moment capacity of the floor by at least 23 times as shown in Table 4. Thus, our conclusion is that the cask unloading cell floor will be punctured by any presently used cask that is accidentally dropped.

AREAS FOR FURTHER INVESTIGATION

Two topics have been identified for possible further investigation. One is the seismic analyses of the fuel racks under various canister placement configurations. Areas of concern are the connections of the racks to the FRS pool, impact of the rack into walls F/W and F/C, sliding of canisters, and rack collapse.

The second topic is to determine the actual thermal gradient in the pool walls. This could be accomplished by field measurements and analytical simulations. This would provide the NRC with a documented basis to verify the intuition and judgment used in our analysis.

Both of these topics were beyond the scope of the present investigation.

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