SEISMIC EFFECTS ON MODULARIZED SPENT FUEL STORAGE FACILITIES

by

R.G. Dong[I] and F.J. Tokarz[II]

SYNOPSIS

Large pools are being considered to provide additional storage capacity for spent fuel from nuclear power plants. Because of the large size modularization of the pool into cells would enhance operational safety and convenience in terms of isolating trouble spots and performing localized clean-up. However, the effects of modularization on earthquake resistant was not clear. An investigation of these effects was made and the results are presented. Our findings indicate modularization may or may not be advantageous in terms of structural loads depending on the pool configuration and installation.

INTRODUCTION

Spent fuel from nuclear power reactors are stored in pools located at power plants and fuel reprocessing plants while waiting to be reprocessed. The reprocessing capacity in the nation is expected to fall below the demand for quite some time; therefore, additional storage is being considered in the form of large pools to be part of reprocessing plants or to be operated independently of the plants. Currently considered pool geometries are roughly 240 ft by 160 ft in plan and 40 ft deep. The pools will most likely be embedded deeply in the ground, although above ground installations are not ruled out. Because of the large size, modularization into cells would enhance operational safety and convenience in terms of isolating trouble spots and performing localized clean-up. However, it was not clear what effects modularization has on earthquake resistance. We investigated these effects at the request of the U.S. Nuclear Regulatory Commission, and presented our findings in this paper.

A site with hard soil situated well above the water table is considered reasonable for such facilities. A soil layer composition representative of a hard site is shown in Fig. 1. The density and modulus of the layers increase with depth giving an overall site shear wave velocity of 3500 fps. An intermediate hard site could also be considered suitable for such facilities, and this will be a subject for investigation at a later time.

Two modularized pool configurations were investigated along with an open configuration, Fig. 2. The pools have no roofs as do storage pools now in existance, and all three have the same total volume capacity. Two cell sizes were selected, 40 ft x 80 ft and 80 ft x 80 ft. Many pools now in use are 40 ft x 80 ft in plan size; consequently cells of this geometry are a likely choice for a modularized pool. The cell size 80 ft x 80 ft also appears reasonable. Both embedded and above ground installations were analyzed. The analyses were carried out with the finite element time-history integration code LUSH[1].

I. Mechanical engineer, Lawrence Livermore Laboratory, Livermore, California.

II. Head, Structural Mechanics Group, Lawrence Livermore Laboratory, Livermore, California.

ANALYTICAL MODELS

The two-dimensional finite element models shown in Fig. 1 correspond to the pool cross sections indicated in Fig. 2. Only one half of each cross section was modeled taking advantage of symmetry. Material properties are listed in Table 1. The damping value in concrete was taken as 7% in accordance with the U.S. Nuclear Regulatory Commission, Regulatory Guide 1.61.[2] A preliminary study[3] and existing designs of such pools indicate the walls can be expected to be 4 ft to 7 ft thick. Anticipating a need for extra strength in embedded exterior walls we modeled these to be 7 ft thick, while above ground exterior walls and all interior walls were taken as 5 ft thick. The impulsive hydrodynamic pressure of the water was modeled by an equivalent mass W_o in accordance with Housner's theory.[4,5] The masses W_R and W_w represent the racks fully loaded with fuel and the water immediately surrounding the racks. This water was considered anchored to the rack since it occupies the interstitial spaces within the volume enveloped by the racks. A conservative approximation of the loads transmitted by the racks to the pool walls is to assume that the total lateral support for the racks comes from the walls.

The soil mass was extended 1000 ft beyond the pool outer wall to recover free-field conditions at the boundary. A comparison of response spectrum at the boundary with that of free-field is shown in Fig. 3. An embedment depth of 47 ft was selected to match the ground level with the water level. The finite elements of the model were dimensioned to capture shear wave frequencies up to 15 hz; the highest frequency of significance to the pool structures is roughly 11 hz.

SEISMIC ANALYSIS

The contributions to the total load carried by the pool structure are:

- Dynamic soil pressure
- Pool structure inertia
- Impulsive water pressure
- Rack and surrounding water inertia
- Convective water pressure
- Hydrostatic pressure
- Static soil pressure

The first four are dynamic, and the last three are static by comparison. The convective water pressure is due to water sloshing which is at a very low frequency ranging from 0.07 hz to 0.25 hz.

The four sources of dynamic load are included in the analyses with the LUSH code.[1] This is a finite element code containing a nonlinear description of soil constitutive behavior and using a time-history integration approach. The bedrock acceleration time-history we used was deconvoluted from a prescribed free-field time-history using the code SHAKE.[6] The free-field time-history used was especially tailored to produce the response spectrum in the U.S. Nuclear Regulatory Commission Regulatory Guide 1.60.[7] The response spectrum obtained is compared with that in ref. 7 in Fig. 3. The static loads were evaluated independent of the LUSH analyses, and are summed with the dynamic loads directly. The convective water pressure was evaluated with Housner's theory.[4,5] This theory was shown to be adequate for our purposes in a separate study.[8] The earth pressure at rest was taken as the static soil pressure.

The bending moment in the wall is the most prominent measure of structural load in the pools. The bending moment varied from a maximum at the wall root, to between 10% and 30% of maximum at mid-height, and to zero at the top. The profile of the variation was thus very much the same for all walls. The lower half of the walls carried the significant loads, and in particular the wall root bending moment is a good indicator of load intensity. Therefore, comparisons of the loads carried by the various pool configurations are made in the basis of the wall root bending moment.

EFFECTS OF MODULARIZATION

Wall root bending moments are tabulated in Table 2. The effects of plate action are included as necessary. The effects of modularization are summarized in Fig. 4 in terms of wall root bending moments normalized to the open pool. Values in single parenthesis pertain to interior walls filled on one side only. These were estimated by taking the results obtained for above ground exterior walls. The response spectrum at the root of interior and exterior walls of the same pool cross section were virtually identical in every case, and the loads carried by corresponding embedded and above ground interior walls filled in both sides were much the same. Extrapolating these observations to the case of interior walls filled only on one side led us to conclude that this approximation is essentially exact for above ground interior walls and conservative up to 26% for embedded interior walls. Values in double parenthesis are estimates based on the assumptions that the ratio of the load carried by a 80 ft interior wall to that carried by a 40 ft interior wall in a 12 celled pool remains roughly the same under all situations. This is somewhat arbitrary, but in the absence of analytical results this should provide reasonable estimates.

Modularizing an embedded pool could result in an increase or decrease in the exterior wall loads depending on the configuration. The interior walls of an embedded pool carry higher loads than do the exterior walls. Dynamic amplification was measureable for the interior walls whereas due to the soil foundation exterior walls had little, or no, dynamic amplification. In addition, the hydrostatic load was a significant contribution in the case of interior walls filled only on one side.

In retrospect because the interior walls carry high loads than exterior walls in an embedded pool, exterior walls should not have to be modeled thicker than interior walls. However, we do not expect drastic changes in the results if the wall thickness were altered but yet kept within the range of 4 ft to 7 ft. The trends established are expected to remain.

The modularized above ground pools, on the other hand, exhibited greater earthquake resistant than did the open pool. The loads carried by the modularized pools are all lower than those carried by the open pool as shown in Fig. 4.

ABOVE GROUND VS EMBEDDED INSTALLATIONS

Figure 5 shows the ratio of above ground to embedded wall loads for corresponding walls. In virtually every case the above ground walls carry higher loads than embedded walls. Exterior walls in particular exhibited the greatest difference. This was due to an appreciable amount of dynamic amplification in the above ground exterior walls and almost none in the embedded exterior walls. The loads carried by the interior walls were not drastically different between embedded and above ground installations.

CONCLUSIONS

In the case of an embedded pool the effects of modularization on earthquake resistance must be evaluated on a case by case basis. The effects cannot be relied upon to be beneficial as it is dependent on the pool configuration. For the above ground pool on the other hand modularization consistently resulted in increased earthquake resistance for the configurations examined. However, structural loads are higher in above ground pools than in embedded pools, and significantly so in the exterior walls. Dynamic amplification had a significant effect on all walls except the embedded exterior walls.

REFERENCES

1. Lysmer, J., Udaka, T., Seed, H.B., and Hwang, R., "LUSH, A Computer Program for Complex Response Analysis of Soil-Structure Systems," Report No. EERC 74-4, College of Engineering, University of California, Berkeley, California, 1974.

2. "Damping Values for Seismic Design of Nuclear Power Plants," Regulatory Guide 1.61, U.S. AEC, October, 1973.

3. Dong, R.G., and Tokarz, F.J., "Preliminary Seismic Investigation of Five Configurations for Spent Fuel Storage Pools," UCID-16831, Lawrence Livermore Laboratory, May 16, 1975.

4. Housner, G.W., "Dynamic Pressures on Accelerated Fluid Containers," Bulletin of the Seismological Society of America, Vol. 47, No. 1, 1957, pp. 15-36.

5. "Nuclear Reactors and Earthquakes," U.S. Atomic Energy Commission, TID-4500, August 1963.

6. Schnabel, P.B., Lysmer, J., and Seed, H.B., "SHAKE, A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites," Report No. EERC 72-12, College of Engineering, University of California, Berkeley, California, December, 1972.

7. "Design Response Spectra for Seismic Design of Nuclear Power Plants," Regulatory Guide 1.60, U.S. AEC, October, 1973.

8. Dong, R.G., To be written.

TABLE 1

Material Properties

	Reinforced Concrete	Soil
Density, PCF	150	See Fig. 1
Zero strain shear modulus, KSF	251800	See Fig. 1
Poisson's Ratio	0.2	0.3
Zero strain damping, %	7%	5%

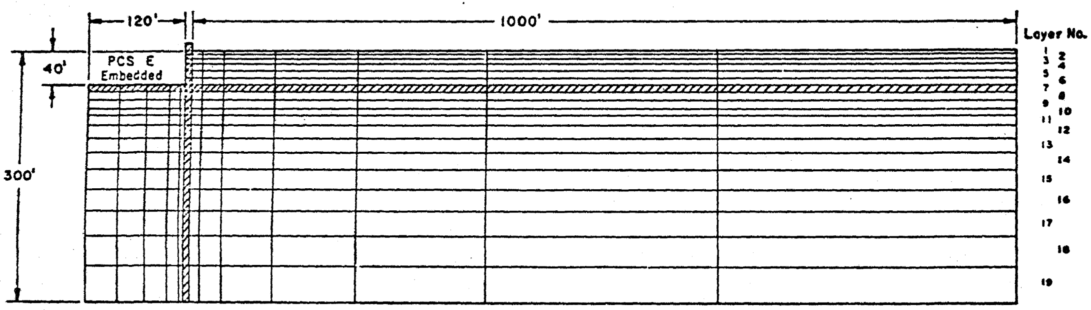
TABLE 2

Wall Root Bending Moments

E - Emedded
AG - Above Ground
EW - Exterior Wall
IW - Interior Wall
FBS - Filled on Both Sides
FOS - Filled on One Side Only

Pool Cross Section	E or AG	EW or IR	FBS or FOS	Wall Root Bending Moment, Ft-Lb
E	E	EW	—	230900
C2	E	EW	—	246000
C2'	E	EW	—	158400
A1	E	EW	—	153000
E	AG	EW	—	132200
C2	AG	EW	—	112800
C2'	AG	EW	—	287500
C2	E	IW	FBS	545200
C2	E	IW	FOS	112800
C2'	E	IW	FBS	118800
C2'	E	IW	FOS	287500
A1	E	IW	FBS	337100
C2	AG	IW	FBS	701500
C2	AG	IW	FOS	112800
C2'	AG	IW	FBS	161400
C2'	AG	IW	FOS	287500

FIG. 1 — FINITE ELEMENT MODELS OF POOL CROSS SECTIONS (PCS)



Layer No.	Density PCF	Zero Strain Shear Modulus KSF
1	100	3324
2	"	3073
3	125	7689
4	"	8418
5	"	9361
6	"	9114
7	"	10912
8	"	10912
9	"	12720
10	"	12720
11	"	15680
12	150	18634
13	"	24643
14	"	33960
15	"	44767
16	"	65800
17	"	94330
18	"	136580
19	"	247480

Note: Pool structure thickness are subdivided in 4 equal divisions. These are not shown to enhance clarity. Shaded areas in PCS E indicate regions discussed.

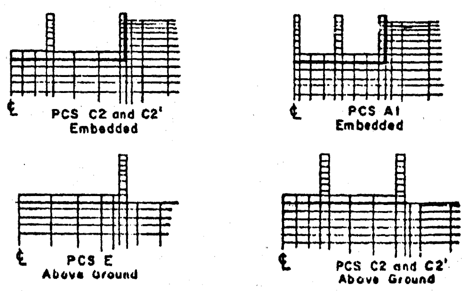


FIG. 2 POOL CONFIGURATIONS INVESTIGATED

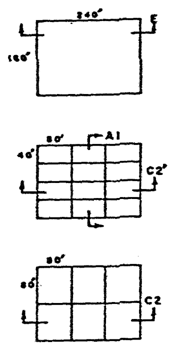


FIG. 3 ACCELERATION RESPONSE SPECTRA

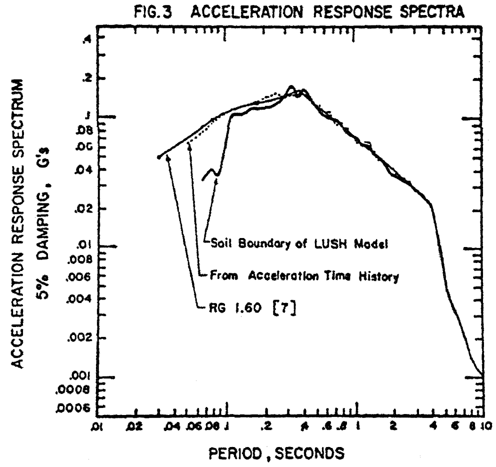


FIG. 4 EFFECTS OF MODULARIZATION ON WALL ROOT BENDING MOMENT

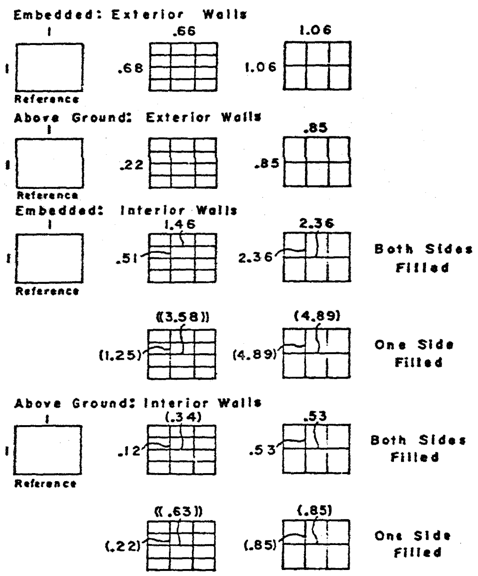
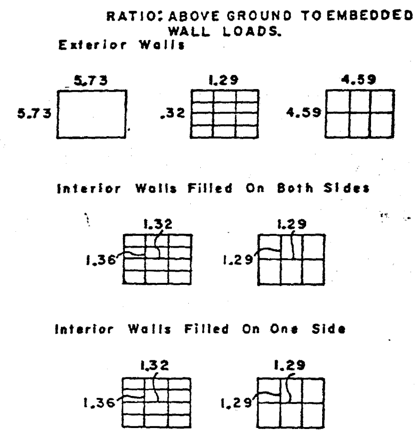


FIG. 5 EMBEDDED VS ABOVE GROUND INSTALLATIONS



DISCUSSION

B. Sarkar (U.S.A.)

It appears that the author did not model the contained water in his structural model. Has the author determined the equivalent hydrostatic forces at the nodal points based on the method suggested by Housner ?

Author's Closure

With regard to the question of Mr. Sarkar, we wish to state that the water in the structure was not modeled by finite elements; the method by Housner was used to describe the hydrodynamic forces. This was stated in the sections entitled "Analytical Models" and "Seismic Analysis" in the paper.