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UCRL-52167

SEISMIC ANALYSIS OF LARGE POOLS

Richard G. Dong and Frank J. Tokarz

November 17, 1976

Prepared for U.S. Energy Research & Development
Administration under contract No. W-7405-Eng-48

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Printed in the United States of America

Available from

National Technical Information Service

U.S. Department of Commerce

5285 Port Royal Road

Springfield, VA 22161

Price: Printed Copy \$; Microfiche \$3.00

Page Range	Domestic Price	Page Range	Domestic Price
001-025	\$ 3.50	326-350	10.00
026-050	4.00	351-375	10.50
051-075	4.50	376-400	10.75
076-100	5.00	401-425	11.00
101-125	5.50	426-450	11.75
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LAWRENCE LIVERMORE LABORATORY

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SEISMIC ANALYSIS OF LARGE POOLS

Abstract

Large pools for storing spent, nuclear fuel elements are being proposed to augment present storage capacity. To preserve the ability to isolate portions of these pools, a modularization requirement appears desirable. The purpose of this project was to investigate the effects of modularization on earthquake resistance and to assess the adequacy of current design methods for seismic loads. After determining probable representative pool geometries, we examined three rectangular pool configurations, all 240 x 16 ft and 40 ft deep. One was unmodularized; two were modularized into 80 x 40 ft cells in one case and 80 x 80 ft cells in the other. We studied both embedded and above-ground installations for a hard site and embedded installations for an intermediate hard site.

We found, in general, that modularization was unfavorable in terms of reducing the total structural

load attributable to dynamic effects, principally because one or more cells could be left unfilled. The walls of unfilled cells would be subjected to significantly higher loads than the walls of a filled, unmodularized pool. Generally, embedded installations were preferable to above-ground installations, and the hard site was superior to the intermediate hard site. We determined that Housner's theory was adequate for calculating hydrodynamic effects on spent fuel storage pools.

Current design methods for seismic loads were found to be satisfactory when results from these methods were compared with those from LUSH analyses. As a design method for dynamic soil pressure, we found the Mononobe-Okabe theory, coupled with correction factors as suggested by Seed, to be acceptable. The factors we recommend for spent fuel storage pools are tabulated.

Introduction

Spent fuel elements from nuclear power reactors are stored in pools 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 sometime. Therefore, large pools are being proposed as part of either independent, spent fuel storage installations or large fuel reprocessing plants. To preserve the ability to isolate portions of a pool, required modularization of these pools seems desirable. This would, of course, increase the construction cost and reduce design options. Consequently, the industry has objected to such a requirement when it was suggested in the U.S. Nuclear Regulatory Commission, Regulatory Guide 3.24¹. Although modularization would be helpful for contamination control and for decontamination, its effect on earthquake resistance of the pools was not certain. In particular, the forces transmitted to pool structures with and without modularization, needed investigation. In addition, an evaluation of current design methods for seismic loads was desirable. Therefore, the purpose of this project was to investigate the effects of modularization and to assess the

adequacy of current design methods. The results will provide input for proposed regulations on storage facilities, provide a basis for licensing positions on modularization of storage pools, and indicate suitable design methods for earthquake resistance.

Our effort began with an industry survey to gain a better understanding of design methods in current use and of pool configurations preferred by the industry. The firms contacted are listed in Table 1. Design methods representative of current practice are given in Table 2. A rectangular pool, 40-ft deep with cells in the range of 40 × 80 ft and 80 × 80 ft, appeared to be favored. Embedded installations were generally preferred, but above-ground installations were not ruled out. The expected wall thicknesses ranged from 4 to 7 ft. A preliminary study we did on the relative costs of various modularized configurations also suggested wall thicknesses in this range.²

As a result of these preliminary studies, three pool configurations were selected for investigation (see Fig. 1). The first was not modularized, the second had 40 × 80 ft cells, and the third had 80 × 80 ft cells. All three configurations

Table 1. Industrial firms and organizations contacted.

Organization	Location
Nuclear Fuel Services,	Rockville, Md.
E. R. Johnson Assoc.	Vienna, Va.
Stone and Webster	Boston, Mass.
General Public Utility	Parsippany, N. J.
Consolidated Edison	New York, N.Y.
Exxon Nuclear Co.	Bellevue, Wash.
Bechtel Corp.	San Francisco, Calif.
Tennessee Valley Authority	Knoxville, Tenn.
Gilbert Assoc.	Reading, Pa.
Sargent and Lundy	Chicago, Ill.
United Engineering & Construction	Philadelphia, Pa.
Program and Remote Systems	St. Paul, Minn.
Metropolitan Water District	Los Angeles, Calif.
Southwest Research Institute	San Antonio, Tex.
General Electric Co.	San Jose, Calif.
Southern California Edison Co.	El Monte, Calif.
James Montgomery and Associates	Los Angeles, Calif.
California Institute of Technology	Pasadena, Calif.
University of California	Berkeley, Calif.

Table 2. Design analysis methods representative of current practice.

Parameter measured	Method
Dynamic soil pressure	Mononobe-Okabe theory ^{9,10} plus correction factor recommended by Seed ¹¹
Wall inertia load	Response spectrum method
Impulsive water pressure	Housner's theory ^{5,6}
Rack inertia and entrapped-water inertia	Response spectrum method
Convective water pressure	Housner's theory ^{5,6}
Hydrostatic pressure	
Static soil pressure	Earth pressure at rest

Fig. 1. Pool configurations investigated. The cross sections examined are indicated by letter codes.

had the same volume capacity. The water depth was 40 ft, with the fuel racks occupying the lower 15 ft of the depth.

Two kinds of sites, one hard and the other intermediate hard and both situated well above the water table,

are considered reasonable for a spent fuel storage facility. The hard site was defined as having an average shear wave velocity of 3500 fps and the intermediate hard site, 1500 fps. For the hard site, both embedded and above-ground installations were investigated. For the intermediate hard site, only embedded installations are considered likely to be used. The soil layers comprising the two sites are shown in Figs. 2 and 3; the soil depth to bedrock was 300 ft for the hard site and 140 ft for the intermediate hard site. The material type, density, zero-strain shear modulus, and damping fraction are indicated. Both the density and the modulus increase with depth in a realistic manner.

Various combinations of site and installation types for the three pool configurations were analyzed with the code LUSH³. The worst loading situations were investigated, including ones where the fuel racks were filled to capacity with spent fuel elements and where modularized pools had one empty cell. The effects of modularization were investigated by comparing the LUSH results for the various pool configurations. The adequacy of current design methods was assessed by comparing these results with those obtained using the LUSH code.

Layer No. from top	Density PCF	Zero-strain shear modulus KSF
1	100	3324
2	"	3073
3	125	7689
4	"	8418
5	"	9361
6	"	9114
7	"	10 912
8	"	10 912
9	"	12 720
10	"	12 720
11	"	15 680
12	150	18 634
13	"	24 643
14	"	33 960
15	"	44 767
16	"	65 000
17	"	94 330
18	"	136 500
19	"	247 400

Fig. 2. Soil layers in the hard site.

Of special interest was the determination of a suitable method for evaluating the hydrodynamic effects. The validity of available methods was

Layer No. from top	Density PCF	Zero-strain shear modulus KSF
1	100	1546
2	"	3531
3	125	4930
4	"	5516
5	"	6183
6	"	7084
7	"	8001
8	"	8916
9	"	9836
10	"	10 525
11	"	11 223
12	"	11 642
13	"	12 058
14	"	12 232
15	"	12 411
16	"	12 606

Fig. 3. Soil layers in the intermediate hard site.

uncertain because of a lack of experimental data on large pools. Consequently, we conducted water-motion experiments on two large swimming pools near the Nevada Test Site during two large underground nuclear events. The data obtained played an important role in our selection of a suitable method for hydrodynamic effects. In

summary, the principal tasks of this project were to:

- Evaluate the effects of modularization on dynamic structural loads,
- Assess the adequacy of design analysis methods in current practice,
- Recommend suitable calculational methods for hydrodynamic effects.

Analyses, Calculations, and Experiments

ANALYSES BY LUSH CODE

Finite Element Models -- The LUSH code handles only two-dimensional geometries. Consequently, pool configurations were analyzed by two-dimensional models corresponding to the cross sections indicated in Fig. 1. In every case, except cross section C2', a two-dimensional model is a close approximation, because a wall 80 ft long, or longer, and 45 ft high responds at midspan essentially as if it is cantilevered at its base. For cross section C2', significant plate action is present in the short, 40-ft long walls, (45 ft high). We approximated the added stiffness provided by this plate action by increasing the stiffness 10-fold as determined by plate theory.

The two-dimensional, finite element models shown in Fig. 4. correspond to the pool cross sections indicated in Fig. 1. Because of symmetry, only one-half of each cross section was modeled. Material properties are

listed in Table 3. The damping value in reinforced concrete was taken as 7% in accordance with the U.S. Nuclear Regulatory Commission, Regulatory Guide 1.61⁴. Anticipating a need for extra strength in embedded exterior walls, we modeled these as 7 ft thick. Above-ground exterior walls and all interior walls were modeled as 5 ft thick.

The impulsive hydrodynamic pressure of the water was modeled by an equivalent mass in accordance with Housner's theory^{5,6}. The adequacy of Housner's theory was addressed as a separate part of our study. Lumped masses represented both the racks fully loaded with fuel and the water immediately surrounding the racks. It is reasonable to consider this water as anchored to the racks, because it occupies the tight interstitial spaces within the volume enveloping the racks. For a conservative approximation of the loads transmitted by the racks to the pool walls, one may assume that the total

Notes: PCS = pool cross section. Wall thickness (cross-hatched regions) of pool is subdivided into four equal divisions (not shown).

Fig. 4. Finite element models for hard site. (Models for intermediate hard site are similar.) Soil layer data is the same as that shown on Fig. 2.

Table 3. Properties of materials.

Material	Unit weight, PCF	Shear modulus, KSF	Poisson's ratio	Damping, %
Reinforced concrete	150	251.8×10^3	0.2	7.0
Water	62.4			~0
Rock	150	a	0.3	5.0 ^b
Clay	a	a	0.3	5.0 ^b

^a For zero-strain shear modulus and zero-strain damping for earth materials, see Figs. 2 and 3.

^b Zero-strain values.

lateral support for the racks comes from the walls. The racks filled to capacity with fuel are taken to have a density of 97 pcf based on information we obtained on recent higher density rack designs.

The soil model extends 1000 ft beyond the pool to recover free-field conditions at the model's boundary. For embedment, a depth of 47 ft was selected to match the ground level with the water level. A comparison of the response spectrum at the boundary with that of the free-field is shown in Fig. 5. A good recovery was obtained for periods of 0.1 sec and higher. 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

structure is roughly 11 Hz. Consequently, the frequencies of significance are less than 11 Hz; therefore, a recovery of the free-field response spectrum for periods greater than 0.1 sec (Fig. 5) was adequate for our study.

Fig. 5. Acceleration-response spectra.

Finite Element Analyses - The following contribute to the total load carried by the pool structure:

- 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 contributions are dynamic, the last three, static. Because convective water pressure is due to water sloshing at very low frequencies, ranging from 0.07 to 0.25 Hz, it is essentially static.

The four dynamic loads are included in analyses carried out with the LUSH code³. 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.⁷ The free-field time history used (Fig. 6) was especially tailored to produce the response spectrum in the U.S. Nuclear Regulatory Commission, Regulatory Guide 1.60.⁸ In Fig. 5, the response spectrum obtained is compared with that in Ref. 8. A maximum horizontal ground acceleration of 0.2 g was taken as representative of earthquakes east of the

Continental Divide. The effect of vertical motion was found to be insignificant and, therefore, was not included in the analyses. Specifically, two cross sections were analyzed using a maximum vertical acceleration of two-thirds the maximum horizontal acceleration; the effect was nil in one case and less than 1% in the other. The static loads were evaluated independently of the LUSH analyses and were summed directly with the dynamic loads. The convective water pressure was calculated using Housner's theory.^{5,6} The earth pressure at rest was taken as the static soil pressure.

ANALYSES BY DESIGN METHODS

Calculational methods representative of current design practice, listed in Table 2, were applied in our investigation as described below.

Dynamic Soil Pressure - The dynamic soil pressure was evaluated by a combination of the Mononobe-Okabe (M-O) method^{9,10} and the recommendation by Seed.¹¹ Seed recommended increasing the M-O soil pressure by a factor ranging from 1 to 3, depending on soil hardness and structural stiffness. For example, for a perfectly rigid structure bearing against a hard soil, the recommended factor is 3. The values we explored depended in each case on the situation

Fig. 6. Acceleration-time history at free-field surface.

involved, as will be discussed later. The pressure profile was taken as triangular, with the maximum at the top. The maximum horizontal acceleration of the pool exterior wall, taken as 0.2 g, was within 10% of the mean of the maximum acceleration profile of the exterior

wall as found by LUSH analysis. For applying the M-0 theory, the densities of the soil layers adjacent to the wall to be examined are shown in Figs. 2 and 3.

Wall Inertia Load - The maximum load due to the inertia of the wall is proportional to the maximum

acceleration of the wall, and we considered the load to be uniformly distributed over the wall. For an exterior wall bearing against soil, the maximum acceleration was taken as 0.2 g; the soil provided a foundation which prevented significant seismic amplification. For all other walls, the seismic amplification found by LUSH analyses was significant and, therefore, must be taken into account while applying the design methods. The response spectrum technique was used to determine the seismic amplification. The natural frequencies of the walls were calculated assuming the inertias of the impulsive water, racks, and water entrapped by the racks, as masses added to the walls. The response spectrum at the base of each wall was obtained from the LUSH results. The LUSH analyses conveniently provided the spectra we needed for our calculation; however, any other suitable method of obtaining the wall base response spectra would also suffice. Some examples obtained from LUSH results of response spectra at the base of walls are shown in Fig. 7. The wall fundamental periods fall within the range of significant seismic amplification.

Impulsive Water Pressure — The impulsive water pressure is the hydrodynamic response to instantaneous

acceleration of the pool structure. The pressure profile and magnitude was calculated using Housner's theory.^{5,6} We established that Housner's theory was adequate for our purposes; a more detailed explanation is given later. The racks filled with fuel occupy the lower 15 ft of the water depth. Therefore, only the upper 25-ft depth of water was taken to respond in the manner described by Housner's theory. The maximum acceleration of an exterior wall which bears against soil was assumed to be 0.2 g. Seismic amplification was considered for all other walls and was determined by the technique described in the preceding paragraph.

Fuel Racks and Entrapped-water Inertia Load — The interstitial spaces within the fuel racks were estimated to comprise 50% of the enveloping volume. For our investigation, the water occupying this volume was considered as anchored to the rack and, therefore, to move as if a part of the rack. The resulting density of the rack, fuel, and entrapped water was 128 pcf. To be conservative, we assumed the pool structure provided the only lateral support for the rack and the inertia load that was generated acted uniformly over the lower 15 ft of the walls. The maximum acceleration was taken as 0.2 g for an exterior wall bearing

Fig. 7. Acceleration-response spectra obtained at the root of the walls.
The free-field spectrum and the RG 1.60 spectrum are included for comparison.

against soil. For all other walls, seismic amplification was taken into account in the manner described for the wall inertia load.

Convective Water Pressure - The convective water pressure is associated with sloshing and is in response to low-frequency displacements. The natural frequency of sloshing for the pool and cell sizes examined ranged from 0.07 Hz to 0.25 Hz. This was significantly below the structural natural frequencies, the lowest of which was estimated to be 11.0 Hz. Therefore, the convective water pressure was essentially static. At frequencies below 0.25 Hz (periods above 4 sec) the pool motion is essentially the same as the free-field motion as indicated in Fig. 7, where the wall base, response spectra for four representative cases are shown. Therefore, the free-field response spectrum was used in conjunction with Housner's theory^{5,6} to calculate the profile and magnitude of the convective water pressure. As for the impulsive water pressure discussed earlier, only the 25-ft depth of water above the fuel racks was assumed to participate in this response.

Hydrostatic Water Pressure - The hydrostatic water pressure is based on a water density of 62.4 pcf and is applied over the entire 40-ft depth of water.

Static Soil Pressure - The earth pressure at rest was taken as the static soil pressure.

Summation of Loads - For simplicity, all load contributions were evaluated without consideration for vertical motion. As mentioned above under "Finite Element Analyses," a maximum vertical acceleration of two-thirds the maximum horizontal acceleration was found to have a negligible effect on the final results. The contributions to the total structural load were summed in the manner diagrammed in Fig. 8.

Fig. 8. Summation of dynamic and static loads.

HYDRODYNAMIC EFFECTS

Evaluation of Computational Methods

A variety of techniques are available for evaluating various types of fluid-structure interaction.^{5,6,12-20} Some of these, particularly the ones involving numerical techniques, are rather sophisticated and may be suitable for irregular-shaped pools. For regular-shaped pools, such as rectangular and flat-bottomed, we preferred to explore the applicability of simpler techniques such as Housner's theory.^{5,6} However, the validity of this theory, or any other technique, for very large pools could not be firmly established due to a lack of experimental data on large pools. Experimental data are available from scaled-down laboratory experiments with simulated seismic excitation,^{21,22} and from the aerospace industry for a wide variety of tank configurations under simulated flight conditions, e.g., see Ref. 23. Data for large pools subjected to actual earthquake-like ground motions were not found. We were able to carry out such experiments by taking advantage of the unique position of the Lawrence Livermore Laboratory as a weapons development laboratory. Water-motion experiments were conducted on two large swimming pools near the Nevada Test Site during two high-yield underground events. The results

together with conclusions concerning the design criteria for hydrodynamic effects in spent fuel storage pools are presented below.

Swimming Pool Experimental Setup

Two experiments were carried out: the first at the pool in Mercury, Nevada, during the KASSERI Event, October 28, 1975; the second at the Maslow Park pool in Las Vegas, Nevada, during the MUENSTER Event, January 3, 1976. The yield for both events was approximately the same. The pool sites, relative to the event locations, are shown in Fig. 9. Mercury is roughly 50 miles from the event site, Las Vegas about

Fig. 9. Location of swimming pools relative to the KASSERI and MUENSTER underground nuclear events.

100 miles. The instrumentation layouts at the pools are shown in Fig. 10.

The ground acceleration-time history was monitored with Q-Flex^{*} Model No. QA-116-15 accelerometers (Fig. 11), chosen for their ability

^{*} Reference to a company or product name does not imply approval or recommendation of the product by the University of California, the U.S. Nuclear Regulatory Commission, or the U.S. Energy Research & Development Administration to the exclusion of others that may be suitable.

Fig. 10. Instrument placement at Maslow Park pool, Las Vegas, Nevada (a), and at pool at Mercury, Nevada (b).

Fig. 11. Location of typical accelerometer installation on pool coping.

to accurately capture low frequencies, and was recorded on magnetic tape. For the pool sizes investigated, the sloshing frequency was as low as 0.07 Hz. Ground motions in the longitudinal, lateral, and vertical directions were recorded at each accelerometer

installation. The sloshing motion was recorded on motion picture film, 32 frames per second, at six locations for each pool. The slosh-height was indicated by aluminum targets (Fig. 12) painted with one-inch gradations.

The data acquisition system, diesel power generator, and communication system were all housed in a transportable van (operated by the U.S. Energy Research and Development Administration) parked near the pool. Contact with the control center for the nuclear events was maintained by communication radio. Figure 13 shows the cameras on tripods in position at the side of the pool and the van in the background.

Analytical Technique For Hydrodynamic Effects - Theories and methods available for evaluating hydrodynamic

Fig. 12. Indicator target for water response (Maslow Park pool, Las Vegas).

Fig. 13. Typical placement of motion picture cameras and tripods at pool. Transportable van shown in background housed data acquisition system.

effects include the classical linear theory,¹¹ Housner's theory,^{5,6} finite difference methods,^{13,14} and finite element techniques.¹⁵⁻¹⁹

As stated earlier, the simplicity of spent fuel storage pools (rectangular and flat-bottomed) suggested that relatively simple analytical techniques should be explored first. These would include Housner's theory and the classical linear theory. In the present work, pool walls are of 4- to 7-ft-thick reinforced concrete so that overall rigidity will be high. Because Housner formulated his theory especially for rigid structures of regular geometry, his

theory is simpler than the classical one. Nevertheless, Housner⁵ showed that results given by his theory were within 2.5% of classical theory. Therefore, in terms of accuracy and ease of application, Housner's theory appears quite reasonable, a view apparently shared by industry where the theory is widely used. Thus, Housner's theory was selected for comparison with our experimental results.

Linear hydrodynamic response consists of two contributions: impulsive and convective. The impulsive pressure is the response to instantaneous acceleration. A

portion of the water mass behaves as if it were a solid firmly attached to the pool. The mechanism, which appears straightforward, is affected only by possible nonlinearities. The convective response is associated with sloshing and results from low-frequency motions. This response is complicated by damping in the water as well as by possible nonlinearities and, therefore, is in greater need of experimental confirmation. If Housner's theory is shown to be applicable with respect to its convective portion, thereby establishing that linearity holds, then the applicability of the impulsive portion could be assumed to follow. Consequently, our experiments focused on confirming the convective portion of the theory.

For the pool size of interest, the frequency range for the first-sloshing mode is from 0.07 to 0.25 Hz. Unfortunately, in the case of earthquakes, the response-spectrum values at these low frequencies often are not accurately determined.^{21,24,25} Special care was taken in these experiments to assure that accurate values were obtained for the displacement-response spectrum at frequencies down to 0.07 Hz. Hydrodynamic pressures were not recorded, because the expected values would have been too low for accurate determination at

the magnitude of ground motion anticipated. The convective pressure is directly related to recorded slosh height, so for our purposes the lack of pressure data is not critical.

The swimming pools used in our experiments were not flat-bottomed; depth from one end to the other varied from about 3.5 to 11.0 ft (Fig. 14). The profile of the pool at Mercury is similar to the pool at Las Vegas. The mean depths were 6.6 and 7.6 ft for the Las Vegas and Mercury pools, respectively. The pools were analytically treated as flat-bottomed with a depth equal to the mean depth. The flat-bottomed equivalence has been successfully applied to conical containers for evaluating the overall structural load and the slosh response up through the second slosh mode.^{23,26} According to Ref. 26, for the equivalence to be valid, the container should not be extremely shallow; i.e., the mean depth to diameter ratio, h/D , should not be $\ll 0.25$. The effective h/D ratio for the swimming pools was 0.11 which is $\ll 0.25$. However, because the pools investigated are closer to being flat-bottomed than are the conical containers, we feel the flat-bottomed equivalence is valid. The depth of the pools varied gradually and only

All dimensions given in feet.

Fig. 14. Profile of water depth for Maslow Park pool, Las Vegas.

in one direction, never becoming zero as is the case along the entire circumference of a conical container. For the test pools, good agreement between data and theory lend support to the validity of this equivalence.

Results and Conclusions.

Experimental Observations Compared with Theory -- At the pool sites, the ground motion generated by the nuclear events was very similar to that of a distant earthquake. The recorded acceleration-time histories for the Las Vegas and Mercury pools are shown in Figs. 15 and 16, respectively. Figure 17 shows an example of displacement-time history obtained by integration. The response spectra (Figs. 18 and 19) are comparable to those of an earthquake. For example, in Fig. 18a the displacement-response spectrum obtained for the Las Vegas

pool is compared to the spectrum for seismic design as specified in the U.S. Nuclear Regulatory Commission Guide 1.60.⁸ The latter spectrum was scaled down to a maximum ground acceleration of 0.0075 g to match the Las Vegas record.

Special care was taken to extract accurate values for the displacement-response spectrum from the acceleration records. To obtain spectral displacements, the effect of drift, inherent in all such instrumentation systems, must be accounted for in the manipulations. A refinement of Trifunac's procedure,²⁵ as explained by Jackson,²⁷ was followed in making the necessary drift corrections. However, in our case, an additional improvement was available, because one-half minute of quiescent data were taken before the ground waves from the nuclear

Fig. 15. Acceleration-time history for Maslow Park pool, Las Vegas.

Fig. 16. Acceleration-time history for Mercury, Nevada, pool.

Fig. 17. Example of displacement-time history (Maslow Park pool, Las Vegas).

event arrived. This provided information on drift alone. The displacement-response spectra for the longitudinal and lateral motions at each recording station are shown in Figs. 18 and 19. The mean curve that is drawn through each spectrum plot is later used in theoretical evaluations for slosh height. The vertical component of motion was not used since it does not contribute to the slosh response.

The water response data recorded on motion picture film revealed high-frequency, short-length water waves generated by the first arrival of ground motion. These died down rather quickly, and the first sloshing-mode eventually dominated. This is in general agreement with laboratory observations by Kana and Dodge.²¹

The maximum amplitude of sloshing occurred about 60 sec after arrival time. The total motion can be considered as a summation of the longitudinal and lateral slosh motions, with the slosh frequencies differing in the two directions. At a given location in the pool, the most probable maximum slosh amplitude is the root mean square sum of the contributions from the two directions. A representative amplitude was extracted from each data record by randomly selecting several large excursions between a minimum and a maximum and averaging the readings (Table 4). Notice that the slosh amplitudes are higher in the Las Vegas than in the Mercury pool even though the maximum ground acceleration is greater at Mercury. Low-frequency

Fig. 18. Displacement-response spectra, 0.1% damping (Maslow Park pool, Las Vegas).

Fig. 19. Displacement-response spectra, 0.1% damping (Mercury, Nevada, pool).

Table 4. Experimental vs theoretical water slosh amplitudes.

Target station	Mercury Experiment			Las Vegas Experiment		
	Housner's theory, in.	Test data, in.	T/E ratio ^a	Housner's theory, in.	Test data, in.	T/E ratio ^a
2	0.23	0.26	0.88	0.49	0.49	1.00
3	0.32	0.26	1.23	0.61	0.60	1.02
4	0.29	0.34	0.85	0.51	0.56	0.91
5	0.28	0.29	0.96	0.50	0.60	0.83
6	0.32	0.41	0.78	0.60	0.70	0.86
7	0.23	0.23	1.00	0.49	0.52	0.94
Average T/E ratio:			0.95			0.93

^a Ratio of theoretical to experimental slosh amplitudes.

motions generated by underground nuclear events are generally greater in Las Vegas than in Mercury; therefore it was not surprising to see greater slosh amplitudes in the Las Vegas pool.

Theoretical slosh heights at each data-recording station were evaluated using Housner's theory.^{5,6} Only the first sloshing-mode was considered. Spectral displacements used in the slosh amplitude determinations were those at the center of gravity of the total water mass. These values at the center of gravity were calculated from values determined at the opposing corners of the pool using a linear variation of motion across the pool. The mean spectral displacement curves

(Figs. 18 and 19) were used in this determination. This is in keeping with treating the pools as if flat-bottomed; a flat-bottomed configuration is a representation of the mean of the actual configuration.

Damping in the water depends on pool geometry and size. Expressions for circular vessels were represented in Ref. 23 and by Kana and Dodge.²¹ Applying the expression from either source to the swimming pools and using an equivalent circular geometry indicates the damping in the water should be nearly zero. The displacement-response spectra for a typical earthquake are practically the same for all values of damping equal to, and below, 0.1% of critical. Therefore,

a damping value of 0.1% was used for evaluating theoretical slosh amplitudes (see Table 4). Between 0.1 and 0% damping, the slosh amplitude was estimated to vary by only 2.5%.

The ratio of the theoretical to the experimental slosh amplitude value (T/E ratio) at each target station indicates generally good agreement (Table 4). The fact that the pools were not really flat-bottomed would partially account for the discrepancies. The average value of the ratio is a good indication of the overall agreement, which is more significant than station by station agreement, and the comparison with theory is excellent on this basis.

Applicability to Spent Fuel

Storage Pools - This experimental study covered hydrodynamic characteristics of pools in the size range of interest for spent fuel storage. The first sloshing-mode frequencies of the swimming pools were comparable to those projected for spent-fuel storage pools (see Table 5). Damping in the water decreases with increasing pool size. Therefore, nearly zero damping established for the swimming pools would hold also for larger spent-fuel storage pools.

The validity of applying a linear theory such as Housner's^{5,6} to spent

fuel storage pools subjected to actual earthquake was addressed by evaluating the sloshing response to the response spectrum specified in Regulatory Guide 1.60.⁸ A maximum ground acceleration of 0.2 g was taken as representative of earthquakes east of the Continental Divide. In Table 6, the slosh amplitudes evaluated for the pool sizes of concern are compared with the linear range as prescribed by a criterion suggested by Kana and Dodge.²¹ The criterion specifies that linearity applies if the slosh amplitude is no greater than one-tenth of the pool length in the direction of concern. Therefore, the calculated slosh amplitudes for the pool sizes shown in Table 6 are within the linear range except for two cases which are slightly beyond the linear range. Nonlinearity tends to give the same effect as increased damping in that the slosh height will be less than predicted by linear theory.²⁸ This fact was experimentally observed also by Clough²⁹ in a series of laboratory experiments on 6-ft- and 12-ft-diameter tanks. Therefore, in terms of the slosh height, and thus the convective hydrodynamic pressure as well, the results obtained from the linear theory will be conservative for the two cases lying slightly beyond the linear range of response.

Table 5. Comparison of first-mode sloshing frequencies of swimming pools and spent-fuel storage pools.

Type of pool	Side dimension ^a , ft	Depth, ft	Fundamental frequency, Hz
Swimming pool:	40	7 ^b	0.18
	80	7 ^b	0.09
Spent-fuel storage pool:			
Unmodularized:			
Without racks	160	40	0.10
	240	40	0.072
Filled with racks	160	25 ^c	0.086
	240	25 ^c	0.058
Modularized: ^d			
Without racks	40	40	0.25
	80	40	0.17
Filled with racks	40	25 ^c	0.23
	80	25 ^c	0.16

^a Each side of the two dimensions of a pool is considered separately.

^b Mean depth.

^c Effective depth.

^d Consisting of individual modules, 40 × 80 ft.

Conclusions Concerning Hydrodynamic

Calculational Methods - Our results

indicate that Housner's theory^{5,6} is adequate for evaluating hydrodynamic effects in spent-fuel storage pools subjected to seismic loading. The question of applicability of linearity must be addressed for pool

dimensions and seismic inputs other than those examined. The pool dimensions examined ranged in length from 40 to 240 ft and in depth from 25 to 40 ft. The seismic input was 0.2 g maximum ground acceleration with a response spectrum corresponding to Regulatory Guide 1.60.⁸

Table 6. Range of applicability of linear hydrodynamic theory.

Side dimension of pool, ft	Linear slosh amplitude ^a , ft	Theoretical slosh amplitude ^b	
		Without racks, ft	Filled with racks, ft
40	4	4.6 ^c	4.5 ^c
80	8	3.5	2.7
160	16	2.2	1.5
240	24	1.6	1.1

^a Criterion is 1/10 of pool dimension²¹.

^b Based on Housner's theory^{23,24} and the response spectrum in Regulatory Guide 1.60⁸ at 0.2 g maximum ground acceleration and 0.1% critical damping.

^c Both values are slightly beyond the linear range but are similar because the effective water depths for slosh response are almost the same (see Refs. 23, 24).

We anticipate no problems in applying a linear theory for pools deeper than 40 ft and longer than 240 ft. However, a pool length less than 40 ft will fall outside the linear range for a 0.2 g maximum ground acceleration, and a linear theory will then tend to be conservative. The damping in the water for the pool sizes of concern is very low. A critical

damping value of 0.2% was found to be sufficiently small for the pool sizes examined. The response spectrum remains virtually unchanged for lower damping values. The displacement-response spectrum at frequencies in the range of 0.07 to 0.25 Hz governs the convective response. Unfortunately, values reported at these frequencies are not always accurate.^{21,24,25}

Effects of Modularization

EVALUATION OF EFFECTS

The effects of modularization were examined under three conditions: embedded installations on a hard site, above-ground installations on a hard site, and embedded installations on an intermediate hard site. An above-ground installation on an intermediate hard site or any installation on a soft site were considered less likely. Our evaluation was based on results from finite element analyses using the LUSH code. The analytical procedures were explained earlier in this report. The effects of modularization are presented as the ratio of structural loads in a modularized as opposed to the unmodularized configuration.

The three pool configurations examined are shown in Fig. 1. 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 modeled as 5 ft thick. To briefly explore the effect of exterior wall thickness, configuration (b) in Fig. 1 also was modeled with 5-ft-thick exterior walls in the embedded installation. The structures and cross sections analyzed are shown in Fig. 20. Cross section A1 for the above-ground

installation was not analyzed with LUSH, because the finite element model for this case exceeded the capacity of our computer system. However, the lack of this particular analysis was not serious, because we were able to use two approximations to generate results which adequately covered this case. One approximation was that the above-ground exterior walls of cross sections A1 and C2 carried roughly the same load. The other approximation was that for all 80×40 ft cells, the ratio of the load carried by the 80 ft interior wall to that carried by the 40 ft interior wall is roughly the same. Thus, these two approximations allowed us to deduce, from the cases which were modeled and analyzed, the loads for an above-ground installation of cross section A1.

The worst load situations were analyzed for each pool configuration. The unmodularized pool was analyzed while it was filled to capacity. For modularized pools, the two possible worst load cases were considered: One, a pool filled to capacity, and the other, a pool having one empty cell and the rest filled to capacity. It turns out that modularizing created a situation in which one or more cells could be

Embedded installations:

Above-ground installations:

Fig. 20. Pool configurations, wall thicknesses, and types of installations studied. The cross sections examined are indicated by letter codes.

left empty, and, as we will see, this resulted in the highest structural loads. Because LUSH is only two-dimensional, we could not model a complete pool having one empty cell and the others filled. Therefore, in these cases, we again resorted to approximations to deduce the loads from those cases which were modeled and analyzed.

For the hard site, an interior wall filled on one side and empty on the other was approximated by an above-ground exterior wall. For the intermediate hard site, above-ground installations were not

analyzed, but we achieved an approximation in an embedded installation by eliminating the soil elements immediately adjacent to the exterior wall. These approximations, which were exact for above-ground interior walls and for exterior walls embedded in the intermediate hard site, ranged from exact to 26% conservative for exterior walls embedded in the hard site. These estimates were based on the fact that the pool floor was found to move as a rigid unit in every case and on an assumption that the pool floor motion was virtually the same for both a

completely filled pool and for a pool with one empty cell.

We deduced the structural load on an embedded exterior wall of an empty cell by using calculational design methods in conjunction with LUSH results. The ratio of dynamic loads on the exterior wall of the empty cell to those on the same wall when the cell is full was calculated by design methods. The LUSH result for the same wall when the cell is full was then multiplied by this ratio. Support for this procedure was obtained by LUSH analysis of the unmodularized pool, taking the ratio of the load in the empty pool to that in the filled pool. The LUSH results gave a ratio of 0.72 as compared to the ratio of 0.66 obtained by design methods, a difference of 8.3%. Therefore, we concluded that the procedure was adequate for our purposes. To the dynamic loads deduced in this manner, we added the static loads. The total load was significantly higher than in a totally filled pool, because the hydrostatic pressure was no longer present to help resist the static soil pressure.

Structural Load Indicator - The bending moment in the wall was the most prominent measure of structural load in the pools. In all cases, the bending moment varied from a maximum at the base of the wall to

between 10% and 30% of maximum at mid-height and to zero at the top. Thus, the profile of the variation was very much the same for all walls. The lower half of the walls carried the significant loads, and the wall root bending moment in particular, was a good indicator of load intensity. Therefore, comparisons of the loads carried by the various pool configurations were made on the basis of the wall root bending moment.

RESULTS

Embedded Installation on the Hard Site - For pools embedded in the hard site, the effects of modularization are given in terms of the wall root bending moments normalized to that of the unmodularized pool (Fig. 21). For a totally filled pool, modularization could result in an increase or a decrease in the exterior wall loads, depending on the configuration. However, modularization creates the option of leaving one or more cells empty. An exterior wall of an empty cell would be loaded significantly higher than the wall in an unmodularized filled pool. The difference between a 5- and a 7-ft-thick exterior wall was noticeable but not drastic. The trends established would not be expected to change had we modeled all exterior walls as 5 ft thick.

Exterior wall, filled cell:

Exterior wall, empty cell:

Interior wall, both sides filled:

Interior wall, one side filled:

Fig. 21. Effects of modularization: Embedded installations on the hard site. The values shown are the ratios for the loads normalized to that of a filled unmodularized pool having a wall-root bending moment = 230,900 ft-lb.

The structural load on virtually every interior wall of the modularized configurations was higher than on the filled unmodularized pool. The only exceptions were the 40 ft long walls of cross sections C2' and C2" (see Fig. 20) when cell sides were filled. The walls of cells filled on one side carried noticeably higher loads than walls of cells

filled on both sides. Again, the option, created by modularization, of leaving one or more cells empty resulted in a measurable increase in the maximum possible structural load.

Above-ground Installation on the Hard Site — In contrast to the above embedded installation, modularization of an above-ground pool on a hard site reduces the structural

Exterior wall, filled cell:

Interior wall, both sides filled:

Interior wall, one side filled:

LW = long walls

SW = short walls

IW = all interior walls

— 7-ft thick wall

--- 5-ft thick wall

Fig. 22. Effects of modularization: Above-ground installation on the hard site. The values shown are the ratios for the loads normalized to the load of a filled unmodularized pool having a wall-root bending moment = 1,322,500 ft-lb.

load in every situation examined (see Fig. 22). For the above-ground installation, modularization was

beneficial in terms of increasing the resistance to earthquake damage.

Embedded Installation on the

Intermediate Hard Site — The effects of modularizing a pool embedded in the intermediate hard site are shown in Fig 23. For pools totally filled, the loads carried by exterior walls of modularized pools were lower than those of the unmodularized pool, while the loads on interior walls were somewhat higher. If one cell were left empty, the loads on both exterior and interior walls of modularized pools were significantly higher than in the filled unmodularized pool except for the 40-ft long walls of cross sections C2' and C2". In terms of structural loads, therefore, modularization was not beneficial for pools embedded in the intermediate hard site. Again, the option of leaving one or more cells empty played a major role in increasing the maximum possible load. As with the hard site, the difference between a 5- and 7-ft-thick exterior wall was noticeable but not drastic. Again, the trends established would not be expected to change even if all exterior walls had been modeled as 5 ft thick.

Exterior wall, filled cell:

Exterior wall, empty cell:

Interior wall, both sides filled:

LW = long walls
SW = short walls
IW = all interior walls
— 7-ft thick wall
--- 5-ft thick wall

Interior wall, one side filled:

Fig. 23. Effects of modularization: Embedded installation on intermediate hard site. The values shown are the ratios for the loads normalized to that of a filled unmodularized pool having a wall-root bending moment = 310,700 ft-lb.

Comparisons Between Embedded and Above-Ground Installations on the Hard Site

Exterior wall, filled cell:

Exterior wall, empty cell:

Interior wall, both sides filled:

Interior wall, one side filled:

LW = long walls

SW = short walls

IW = all interior walls

— 7-ft thick walls

--- 5-ft thick walls

Figure 24 shows the ratios of the above-ground loads to the embedded loads. The above-ground loads on every wall are either higher or essentially the same as the embedded loads. A major cause of the higher loads was the higher seismic amplification in an above-ground installation. Additionally, for an exterior wall of a filled cell, the higher load in an above-ground installation is caused by the absence of static soil pressure to counteract the internal loads.

In general, an embedded installation is preferred in terms of minimizing the structural loads.

Fig. 24. Comparison of embedded and above-ground installations at hard site. For each individual wall, loads were normalized to the embedded installation load.

Comparisons Between Hard and Intermediate Hard Sites

In Fig. 25, a comparison is made, for different pool configurations, in terms of the ratio of structural loads for the intermediate hard site to those for the hard site. The loads on corresponding exterior walls

are quite similar for the two sites. On the other hand, the loads on corresponding interior walls are higher for the softer site. In general, the hard site is preferred in terms of structural loads.

Exterior wall, filled cell:

Exterior wall, empty cell:

Interior wall, both sides filled:

LW = long wall
 SW = short wall
 IW = all interior walls
 — 7-ft thick wall
 --- 5-ft thick wall

Interior wall, one side filled:

Fig. 25. Comparison of hard and intermediate hard sites at embedded installations. For each individual wall, loads were normalized to the loads for the hard site.

Ratio of Dynamic Load to Total Load

The individual contributions to the total load for each wall are shown in Fig. 26 in terms of the worst combinations. We believe the profiles and relative magnitudes are good estimates. Our choice for the worst load combination for an embedded exterior wall of a filled cell (Fig. 26a) was based on the profiles of the dynamic and static soil pressures. Had the dynamic soil pressure acted toward the right (tensile), the static soil pressure near the top of the wall would not

have supported it. Our analyses indicate a lower total structural load would have resulted. Therefore, for dynamic loads determined by both the LUSH and design methods, we took the direction of action to be toward the left as shown in Fig. 26a. Figures 26b, c, and d are all obviously worst combinations.

For each wall, the ratio of the dynamic to the total load is shown in Fig. 27 for the hard site and in Fig. 28 for the intermediate hard site. It is immediately apparent

Fig. 26. Worst load combinations for each situation. Pressure profiles and relative magnitudes are indicated.

Embedded, exterior wall, filled cell:

Embedded, exterior wall, empty cell:

Above-ground, exterior wall, filled cell:

LW = long wall
 SW = short wall
 IW = all interior walls
 — 7-ft thick wall
 --- 5-ft thick wall

Embedded, interior wall:

Above-ground, interior wall:

Fig. 27. Hard site: Ratio of dynamic load to total load.

that the values are greater than unity for all exterior walls of embedded, totally filled pools. For these walls, the sum of the three static loads (Fig. 26a) is generally much less than the sum of the dynamic loads, and the two sums are in the opposite direction. As a result, the total load was lower than the dynamic load.

Figures 27 and 28 revealed that the dynamic load constituted the major

portion of the total load for those walls which had material (water plus racks or earth) bearing on both sides. On the other hand, the static load constituted the major portion of the total load for those walls which had material bearing on only one side. This is an interesting observation which may be helpful in deciding which loads are most important in a given situation.

Embedded, exterior wall, filled cell:

Embedded, exterior wall, empty cell:

Embedded, interior wall:

Fig. 28. Intermediate hard site: Ratio of dynamic load to total load.

Comparison Between Design Methods and LUSH Code

Using the design methods presented earlier in this report, we compared the structural loads obtained by these methods with those obtained using the LUSH code.³ For the dynamic soil pressure, we used as the design method, the Mononobe-Okabe (M-O) theory^{9,10} in conjunction with a correction factor as recommended by Seed.¹¹ The values for this factor have not been documented by Seed and, in our opinion, depend on structural type as well as structural stiffness and site hardness. Because the proper values to use for a spent fuel storage pool were not immediately apparent, we examined these factors as part of our investigation. In the case of a perfectly rigid structure embedded in a hard soil, a factor of 3 was suggested by Seed,¹¹ thus providing a check point for us. Consequently, we explored the following values for both the hard and intermediate hard sites:

- 1, 2, and 3 for a perfectly rigid pool,
- 1 and 2 for a pool slightly less rigid, and
- 1 for a pool only somewhat rigid.

To create a "perfectly rigid" pool for the check point, we analyzed cross section C2 (Fig. 20) with 50 times the normal shear modulus for

reinforced concrete. Cross sections C2' and C2'' with normal shear modulus were considered to be slightly less rigid. Cross sections A1, C2, E, and F with normal shear modulus were considered as only somewhat rigid. To the dynamic soil pressure load calculated in this manner, we added the other load contributions in the summation procedure outlined in Fig. 8.

A direct comparison of dynamic soil pressure determined by the M-O theory and by the LUSH code was of interest but could not be carried out for the following reasons:

- Dynamic loads are treated statically by the M-O theory but dynamically by the LUSH code. A direct comparison of the two methods would imply that dynamic soil pressure obtained by LUSH analysis can be applied statically as is done in the M-O theory. This would be incorrect.
- The LUSH code gives maximum stresses in terms of absolute values rather than as a time history. The moment at which maximum stress occurs differs for each element. Consequently, a true profile of dynamic soil pressure is not available from

the code in its present form.

- The finite element mesh was as fine as our computer system would accept and was adequate for capturing the essence of the response. However, it was too coarse to capture accurately the pressure at the interface of the pool structure and the soil. That is, pressures in soil elements adjacent to the wall differ in varying degrees from those in the wall elements adjacent to the soil.

Of the seven contributions to the total load listed in Table 2, dynamic soil pressure is the most nebulous, the rest being much less questionable. Therefore, an overall comparison of total loads can be considered to focus principally on the validity of the M-O theory.

In comparing design methods with the LUSH code, as was also done in determining the effects of modularization, the structural load indicator used was the wall-root bending moment. The comparison was made in terms of the ratio of the structural loads obtained by design methods to those obtained by LUSH code.

EMBEDDED EXTERIOR WALLS

The comparison between design methods and the LUSH code for embedded exterior walls is shown in Table 7.

The various correction factors used to modify the dynamic soil pressure are indicated. Two situations were considered: One, a pool totally filled, and the other, a pool with one empty cell. As noted above, the second situation produced the highest loads. Table 7 indicates that in the second situation the agreement between design methods and the LUSH code was quite good for both the hard and intermediate hard sites. The agreement was not nearly as good for totally filled pools.

The design methods gave conservative results relative to LUSH analyses with the few exceptions noted in Table 7. Consequently, for the exceptions, we concluded that the correction factors we applied to the M-O method were beyond the appropriate range for the structure rigidity and site hardness involved. Therefore, our recommended factors are given in Table 8 together with the degree of conservatism we observed.

Interior Walls and Above-ground Exterior Walls - For interior walls and above-ground exterior walls, Table 9 compares the results obtained by design methods and by the LUSH code. The interior walls were examined under two situations: One, cells on both sides of the wall were filled (BSF); and the other, a cell on only

Table 7. Design analysis methods compared with the LUSH code: Embedded exterior walls.

Pool cross sections	Dynamic soil pressure, correction factor ^a	Ratio: $\frac{\text{Total load by design method}}{\text{Total load by LUSH code}}$			
		Hard site:		Intermediate hard site:	
		Totally filled pool	One cell empty	Totally filled pool	One cell empty
E	1	3.06	1.42	2.28	1.33
F	1	3.00	1.42	2.29	1.33
A1	1	3.16	1.44	2.69	1.38
C2	1	2.34	1.35	2.02	1.29
C2'	1	0.99 ^b	0.93 ^b	1.33	1.07
	2	1.73	1.36	2.55	1.55
C2''	1	1.29	1.14	1.78	1.19
	2	2.53	1.67	3.24	1.74
C2 (Rigid) ^c	1	0.87 ^b	0.86 ^b	1.12	0.98 ^b
	2	1.49	1.25	1.97	1.42
	3	2.25	1.65	2.90	1.87

^a The dynamic soil pressure is evaluated by the Mononobe-Okabe theory^{9,10} and increased by a factor as recommended by Seed.¹¹ The factor values explored are shown.

^b In this case, design methods gave nonconservative results relative to LUSH analysis.

^c Pool cross section C2 was made "rigid" by increasing the shear modulus for reinforced concrete 50-fold.

Table 8. Recommended correction factors for dynamic soil pressure base compared with the LUSH code: Hard and intermediate hard sites.

Wall dimensions (ft) Length Height Thickness			Recommended factors	Total load conservatism ^a relative to LUSH code
≥80	45	7	1	1.29 to 1.44
40	45	7	2	1.36 to 1.55
40	45	5	2	1.67 to 1.74

^a Conservatism values are those observed in the pool configurations and site characteristics examined and are for the worst situations.

Table 9. Design analysis methods compared with the LUSH code: Above-ground, exterior walls and all interior walls.

Pool cross sections	Ratio: $\frac{\text{Total load by design method}}{\text{Total load by LUSH code}}$			
	Hard site:		Intermediate hard site:	
	Totally filled pool	One empty cell	Totally filled pool	One empty cell
Above-ground exterior walls:				
E & F	2.13			
C2	2.18			
C2'	1.62			
Embedded interior walls:				
A1	2.71		2.40	0.95
C2	2.29	1.72 to 2.41	2.45	1.32
C2' & C2''	3.67	1.11 to 1.55	1.89	1.00
Above-ground interior walls:				
C2	2.23	1.52		
C2'	1.63	1.62		

one side of the wall was filled (OSF). Again, as noted earlier, the OSF situation produced the higher loads. As indicated in Table 9, the agreement between design methods and the LUSH code was good in the OSF situation but was only fair in the BSF situation.

In all but one case, the design methods gave conservative results relative to the LUSH analyses. The exception was the A1 cross section

embedded in the intermediate hard site and with an interior wall filled on only one side. The observed ratio of 0.95, for all practical purposes, says that the design methods agreed with the LUSH code. Our overall conclusion, therefore, is that the design methods give reasonable results relative to the LUSH code. Table 8 tabulates the degree of conservatism we observed.

Conclusions and Recommendations

Our investigation of suitable methods for calculating hydrodynamic effects resulted in the selection of Housner's theory,^{5,6} which is a linear theory formulated for regular-shaped, rigid pool structures. Spent fuel storage pools are virtually rigid and are expected to be regular in shape; i.e., rectangular or circular with a flat bottom. A linear theory was found to be adequate for pool lengths of 40 ft or greater when subjected to an earthquake which is of magnitude 0.2 g maximum ground acceleration and which conforms to the response spectrum specified in Regulatory Guide 1.60.⁸

In terms of the increasing resistance to earthquakes, the overall effect of modularization was found to be unfavorable except for above-ground installations on the hard

site. On the other hand, above-ground installations were less favorable than embedded installations in terms of structural load magnitudes. The principal undesirable effect of modularization was that it creates the possibility of having one or more empty cells. The walls of an empty cell are subject to load magnitudes that are measurably higher than those in a filled, unmodularized pool.

The dynamic load was the major contributor to the total load for walls whose two sides bear on material (water plus racks on one side and earth on the other, or water plus racks on both sides). On the other hand, the static load was the major contributor for those walls which bear on material (water plus racks or earth) on only one side. This information should be helpful in

determining the major loads of concern. In addition, Fig. 26 provides in graphic form the estimated load profiles and the relative magnitudes of the various components contributing to the total load. The worst load combinations for each situation are also shown.

The loads exerted on the exterior walls of the intermediate hard site were either the same or slightly lower than those of the hard site. However, the loads on the interior walls were higher for the intermediate hard site. Overall, we would judge the softer site as being somewhat less favorable with respect to earthquake resistance.

The results obtained by design analysis methods, listed in Table 2, compared favorably with those obtained with the LUSH code.³ An overall comparison of all cases examined indicated that the total structural loads calculated with the design methods will be conservative relative to LUSH results by a factor of from 1 to roughly 3. For the situations where

a wall has material (water plus racks or earth) bearing on only one side, the factor ranged from 1 to roughly 2. For embedded exterior walls, a major influence on the conservatism was the calculation of dynamic soil pressure. The design method used was the Mononobe-Okabe theory^{9,10} increased by a factor of 1 to 3 as recommended by Seed.¹¹ The choice of this factor significantly affects the load on the wall. The values we recommend are listed in Table 8.

These results of our investigation are being further documented by four technical papers.³⁰⁻³³ Three are written: Ref. 30 will be presented at the Sixth World Conference on Earthquake Engineering to be held in New Delhi, India in January, 1977; another³¹ has been submitted to the ASCE Journal of the Power Division for possible publication; a third³² has been submitted to the 4th Structural Mechanics in Reactor Technology Conference to be held in San Francisco, California, in August, 1977.

Acknowledgments

The authors express their appreciation to V. N. Karpenko, Leader during this study; C. E. Walter, Deputy Leader, Nuclear Test Engineering Division, Mechanical Engineering Department, for their encouragement and support of this project. The attention and encouragement given by J. F. Costello of the Office of Standards Development have been most helpful and appreciated. Thanks are given to L. H. Wright for his help with the LUSH code. The assistance of J. E. Mellor and O. R. Mowrey, Jr., was much appreciated. Project funding from the U. S. Nuclear Regulatory Commission is acknowledged.

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