222
3/9/78

MASTER

14. 1899

UCRL-52266

SEISMIC ANALYSIS OF THE NUCLEAR FUEL SERVICE REPROCESSING PLANT AT WEST VALLEY, N. Y.

R. C. Murray

T. A. Nelson

A. M. Davito

May 24, 1977

Work performed under the auspices of the U.S. Department of
Energy by the UCLLL under contract number W-7405-ENG-48.

LAWRENCE
LIVERMORE
LABORATORY
University of California / Livermore

NOTICE

"This report was prepared as an account of work sponsored by the United States Government. Neither the United States nor the United States Department of Energy, nor any of their employees, nor any of their contractors, subcontractors, or their employees, makes any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness or usefulness of any information, apparatus, product or process disclosed, or represents that its use would not infringe privately-owned rights."

NOTICE

Reference to a company or product name does not imply approval or recommendation of the product by the University of California or the U.S. Department of Energy to the exclusion of others that may be suitable.

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	\$ 4.00	326-350	\$12.00
026-050	4.50	351-375	12.50
051-075	5.25	376-400	13.00
076-100	6.00	401-425	13.25
101-125	6.50	426-450	14.00
126-150	7.25	451-475	14.50
151-175	8.00	476-500	15.00
176-200	9.00	501-525	15.25
201-225	9.25	526-550	15.50
226-250	9.50	551-575	16.25
251-275	10.75	576-600	16.50
276-300	11.00	601-up	— ¹
301-325	11.75		

¹Add \$2.50 for each additional 100 page increment from 601 pages up.

LAWRENCE LIVERMORE LABORATORY
University of California, Livermore, California, 94550

UCRL-52266

SEISMIC ANALYSIS OF THE NUCLEAR FUEL SERVICE REPROCESSING PLANT AT WEST VALLEY, N. Y.

R. C. Murray
T. A. Nelson
A. M. Davito

MS. date: May 24, 1977

NOTICE

This report was prepared as in account of work sponsored by the United States Government. Neither the United States nor the United States Department of Energy, nor any of their employees, nor any of their contractors, subcontractors, or their employees, makes any warranty, express or implied, or assumes any legal liability or responsibility for the accuracy, completeness or usefulness of any information, apparatus, product or process disclosed, or represents that its use would not infringe privately owned rights.

Contents

Abstract	1
Introduction	1
Summary	2
Nuclear Fuel Services Facility Description	4
Mathematical Model of the Facility	7
Seismic Analysis Methods	18
Structural Loading	23
Analyses Conducted	26
Failure Definition	27
Results	32
Conclusions	51
Areas for Further Investigation	52
Definition of Seismic Related Terms	54
Acknowledgment	59
References	60

Seismic Analysis of the Nuclear Fuel Service Reprocessing Plant At West Valley, N.Y.

Abstract

This report was prepared at the request of the Nuclear Regulatory Commission as a part of a license review of the nuclear fuel reprocessing plant operated by Nuclear Fuel Services, Inc., at West Valley, N.Y. The report discusses the approach used by the Lawrence Livermore

Laboratory in performing an independent seismic analysis of the facility. It includes a description of the facility, our modeling and analysis techniques, failure criteria, results, and conclusions. Two modes of failure are identified and their consequences discussed.

Introduction

Nuclear Fuel Services, Inc., (NFS) began operations in 1966, at its nuclear fuel reprocessing facility located at West Valley, New York. This plant was the first commercial facility to separate reusable uranium and fuel-grade plutonium from reactor spent fuel. As the number of reactors increased around the country, so did the demand for recycling spent fuel. Thus, NFS decided to undertake a modification and expansion program at its West Valley facility which would increase capacity by 150%. This would also involve the approval of the Nuclear Regulatory Commission (NRC) to obtain a license for reprocessing.

As part of the NRC's license review, the Lawrence Livermore Labora-

tory (LLL) was asked to provide a realistic independent seismic analysis of the facility after modification. LLL was to identify the level of ground motion (up to 0.2g) that the plant could withstand before failure begins and also to identify any weak areas in the plant from a structural viewpoint. To accomplish these tasks, we took the following approach:

- Became familiar with structure
- Reviewed SAR
- Selected analysis technique
- Reviewed, selected, and implemented structural modeling techniques
- Selected structural loading

- Performed analyses
- Developed failure criteria
- Interpreted results
- Drew conclusions

The Los Alamos Scientific Laboratory (LASL) was also asked by NRC to conduct an independent seismic analysis. LLL and LASL conducted independent investigations but were allowed to exchange information. The results of these studies will be used by the NRC

to assess the consequences of structural failure.

Throughout the project we used our judgment extensively to make modeling decisions and to select workable failure criteria. We relied on past experience, published literature, and discussions with the NRC, NFS, LASL, and DRAVO (architect/engineer firm selected by NFS for the plant modification) to assist in making these decisions.

Summary

Summarized below is the approach taken and the results obtained from our analysis of the NFS facility.

STRUCTURAL FAMILIARIZATION

We gained an understanding of the structure by site visits to the West Valley facility, a review of the structural drawings supplied by NFS, and the construction of a plastic scale model. This model allowed us to study the interrelation of various structural components and to see how they were interconnected. The plastic model served as a very useful aid in constructing the computer model of the facility.

REVIEW OF SAFETY ANALYSIS REPORT

A review of the Safety Analysis Report (SAR) for the NFS modification allowed us to become familiar with the West Valley site, soil conditions, and foundation. The structure has a pile foundation consisting of approximately 500 H-Piles from 30- to 60-ft long. We used this document as a source of information for our foundation modeling.

The SAR also provided us information to assist in selecting the structures to be included in our analysis.

SELECTION OF ANALYSIS TECHNIQUE

We reviewed available static and dynamic methods of analysis used to

estimate the seismic response of structures and they are discussed and compared. We elected to conduct a static analysis to estimate the dead load response. Our seismic analyses were conducted by using the response spectrum approach.

REVIEW OF STRUCTURAL MODEL TECHNIQUE

We reviewed the literature and discussed modeling philosophy with NRC and DRAVO before adopting a modeling plan. Our model had to represent the mass and stiffness distribution of the facility as well as account for the pile foundation and embedded structures interacting with the surrounding soil. To account for the expected variations in foundation behavior, four foundation models were analyzed. The scheme we selected was to use uncoupled linear springs to model the foundation interaction and to use a three-dimensional finite element assemblage of plate and membrane elements to model the structure. Additional mass was added to represent heavy equipment loads and parts of the structure not included as finite elements.

SELECTION OF STRUCTURAL LOADING

Since site-specific spectra were not available at the West Valley site, we used the spectra presented in

Regulatory Guide 1.60 to represent the frequency content of the free field input ground motion. We also chose a damping value of 7% for all parts of the model.

This spectrum was used as input to the structure for the dynamic analysis. The spectrum was applied at the foundation level, and all foundation points were subjected to the same input.

PERFORMANCE OF ANALYSIS

We conducted linear elastic analyses by both static and response spectrum techniques to estimate the structural behavior during an earthquake. Static dead-load results were combined with results from the four response spectrum analyses to estimate the onset of failure. The static dead-load analysis was also useful for verifying our model.

DEVELOPMENT OF FAILURE CRITERIA

Failure criteria were needed for both piles and shear walls. We developed these criteria from a literature review and discussions with NRC. These criteria were then used to determine the level of ground motion at which individual components of the model would fail. Our analyses allowed us to estimate the onset of failure. A progressive failure assessment was not possible since a linear analysis technique was selected.

INTERPRET RESULTS

Results from the analyses were assembled and studied. Review of this data allowed us to draw the following conclusions. These values are based on worst case estimates from the foundation cases we studied:

- Reinforced concrete shear walls of the confinement cell structures begin to fail in the range of 0.09 to 0.15g.

- Concrete block walls of less critical structures begin to fail in the range of 0.03 to 0.07g.
- Lateral pile failures began at 0.11g.
- Pile tension failures began at 0.17g.

Specific locations and discussions of the onset of failures can be found in the Results and Conclusions sections.

Nuclear Fuel Services Facility Description

The Nuclear Fuel Service Reprocessing Plant is located south of Buffalo, New York, on the western New York Nuclear Service Center site near West Valley. A site plan of the NFS facility¹ is shown in UCID-17453.* In this study, we concentrated on the process building (Fig. 1) which is constructed of concrete block and reinforced concrete, and is approximately 250-ft wide by 200-ft long by 85-ft high (Fig. 2). The main reprocessing operations are performed inside the process building and include shearing the fuel, separating reusable materials, and handling the waste materials. Although the existing

process building has a stack mounted on it, both NFS and DRAVO indicated that the stack would be relocated during modifications to the plant; therefore, it was not included in our analysis. However, the equipment that would be in the plant after modification was included.

The buildings adjacent to the process building were not included in this analysis directly. A review of the Fuel Receiving Station, Utility Building, Manipulator Service Module Repair Shop, and Iodine and Acid Recovery Cell suggested that these structures could be separated from the process building. The process building offices were included in our analysis only indirectly; we included the weight of the concrete block construction material.

*A companion report containing supporting documentation.

Fig. 1. NFS process building and adjacent structures.

Fig. 2. Existing NFS facility before modification (looking east).

The process building is a very complex structure composed of many individual cells. These cells are of reinforced concrete construction with their floors, walls, and ceilings ranging from 1- to 6-ft. thick. The location of these major cells relative to each other was determined from reprocessing requirements, rather than structural behavior. This cell arrangement is one of the main reasons for the complexity of the facility. The names of the major cells included in our analysis are listed below:

Process Mechanical Cell	(PMC)
General Purpose Cell	(GPC)
Chemical Process Cell	(CPC)
Equipment Decontamination Room	(EDR)

Liquid Waste Cell	(LWC)
Ventilation Wash Room	(VWR)
Analytical Cells	(AC)
Extraction & Purification Cells	(EPC)
Warm Aisle	(WA)
Uranium Product Cell	(UPC)
Acid Recovery Off-Gas Cell	(AOC)

These cells are interconnected to each other to form an integral structure. Some of the interconnections are monolithic while others provide vertical supports and allow relative lateral motion on steel bearing plates.

Surrounding the heavily reinforced concrete cell structures is lighter construction consisting of masonry walls and metal deck flooring supported on steel beams or masonry walls. This construction houses the control room and other less critical parts of the structure.

The massive cell structure is supported by a pile foundation. This was required due to the inadequate bearing capacity of the surface layers of the soil at the site. The foundation consists of approximately 500 H-bearing piles located in staggered rows under the cell walls. A drawing showing the location and orientation of the piles is given in UCID-17453.

Mathematical Model of the Facility

STRUCTURAL MODELING TECHNIQUES

Several modeling techniques can be used for the seismic analyses of complex facilities such as NFS. Key aspects of these techniques are summarized below, and the modeling approach we selected is indicated.

After becoming familiar with the geometry and construction of a structure, a mathematical model may be constructed. The analyst should consider the anticipated loading description and analysis technique. The model must incorporate all the important response characteristics of the actual structural system being analyzed (e.g., mass and stiffness distributions, damping, and pertinent

effects of soil-structure interaction). The mathematical model constructed is usually too complex to utilize a closed-form solution so approximate methods must be employed. Either of two methods can be used to "discretize" the structure: the concentrated mass-spring (lumped-mass) approach or the finite element approach.

The concentrated mass-spring approach divides the structural system into discrete mass points connected by massless springs that represent the stiffness characteristics of the structure. These springs may represent truss members, beams, or shear panels. An example of a simple lumped-mass model is shown in Fig. 3. In this model, the actual mass is

Fig. 3. Concentrated mass-spring representation of a structural system.

concentrated at the appropriate level and the stiffness of the structure is modeled with beam elements that reflect both the bending and shear stiffness of the real structure.

The finite element approach is a technique for "discretizing" a structure into an assemblage of elements. Each element models the mass and stiffness distribution of part of the structure. The mathematical model constructed may be composed of different types of elements to represent differences in structural behavior. Elements are currently available to model truss, beam, shear wall, plate, shell, and pipe behavior. The use of the finite element procedure eliminates the need to approximate continuous structures by concentrated mass-spring systems. Figure 4 shows a finite element representation of a building which uses truss, beam, membrane, and plate elements to model a structure.

STRUCTURAL IDEALIZATION OF THE NFS FACILITY

We selected the finite element method to idealize the NFS facility. This allowed us to preserve the actual geometry and mass and stiffness distribution of the structure. Thus, it eliminated the need to approximate the mass and stiffness properties by concentrating them in individual

Fig. 4. Finite element model of a structural system.

members and attempting to interconnect these parts properly to form an equivalent system. The resulting finite element mesh for the NFS process building is shown in Fig. 5. Detailed modeling information is presented in UCID-17453.

The NFS Process Building is composed of cells for chemical processes constructed of reinforced concrete as well as office spaces utilizing concrete block construction. The primary consideration for this analysis is the integrity of the portions of the building confining radioactive material. For this structure, these portions also comprise the primary lateral-force resisting system where

Fig. 5. Views of the mathematical model of the NFS process building showing substructures.

Fig. 5. (continued). Views of the mathematical model of the NFS process building showing substructures.

the thick shielding and confinement walls act as shear walls. The roof and floor slabs also provide confinement and are subjected to both vertical (out-of-plane) and lateral (in-plane) loads during seismic activity. Thus, the shear walls are subjected to in-plane forces while the roof and floor slabs experience both in- and out-of-plane forces.

As discussed earlier, the process building is a complex structure which could result in a model so complex that it would tax the capacity of the computer hardware and software used for a seismic analysis. Therefore, it was decided to use the most efficient analytical model possible to idealize the important components of the structure while still capturing the overall response characteristics of the total structure. Many assumptions and slight modifications to the actual geometry were made to develop our finite element structural model. We excluded parapets and penetrations through walls and floors, and the locations of some walls and floors were moved slightly to simplify the model. This approach resulted in a three-dimensional assemblage of two-dimensional elements. Since the structure is unsymmetrical, a 3-dimensional model was necessary for capturing such effects as torsional response and coupled modes. Because of the complexity of this structure,

a plastic model was constructed to help visualize the relative position and interconnections of the various components. This model proved to be extremely valuable during the mathematical modeling phase of the analysis. It is illustrated in Fig. 6.

The first step in formulating the structural idealization was to model the substructures (delineated by bold lines in Fig. 5). Plane stress membrane elements were used to model the shear walls. These are more efficient than thin plate elements and adequately provide for in-plane shear resistance. The roof and floor slabs were modeled with thin plate elements which contribute in- and out-of-plane mass and stiffness properties. After constructing the models, a free-vibration dynamic analysis was performed on each discrete substructure, which allowed verification of the model in manageable size units. In addition,

Fig. 6. Plastic model of Process Building (with FRS).

the modeling of discrete substructures permitted the units to be interconnected in different ways if require . In our model, we assumed the various cells were rigidly interconnected while in reality some flexibility between cells does exist. The model constructed by LASL attempted to idealize flexible connections. Because of the reportedly small relative deflections between substructures in the LASL model, it was decided² that LLL would conduct its entire series of seismic analyses with the rigid connections. As a result of the free-vibration analyses, it was discovered that the vertical vibration of the roof and floor slabs were dominating the lower modes. These were not considered significant to the overall response and, therefore, were eliminated by artificially increasing the modulus of these elements. This made them respond essentially as rigid floor diaphragms which is a common assumption in conventional building analysis.

The next step was to include the effects of portions of the structure other than the primary lateral-force resisting system. The office portions of the structure were constructed of materials considerably lighter and weaker than the process cells and, thus, were deemed to have no effect on the structural integrity of the confinement structure. Additional

mass was added to the locations (nodal points) where significant portions of the building were omitted and where heavy equipment masses were located. Much of the supported equipment mass was modeled as increased mass density of the floors and walls. We also included the effects of adjacent structures in our model by adding concentrated masses at appropriate nodal points. Additional structural loading caused by liquid sloshing in tanks was not included.

In summary, the structural idealization of the process building attempted to preserve:

- Mass distribution.
- Major lateral-force resisting system.
- Three-dimensional response.

A linear description of the structural elements and connectivity was selected to allow a response spectrum seismic analysis. This had the following benefits:

- It provided an estimate of response within a reasonable time span.
- It allowed for interpretation and verification of results based on past experience.
- It provided the only practical analysis available for a structure this complex.

We believe our representation of the actual structure uses the most

realistic approaches currently available and should do well in estimating the overall motion of the structure.

FOUNDATION MODELING FOR SEISMIC ANALYSIS

All structures derive support from a foundation through which any earthquake motions will be transmitted. In the past, seismic analyses were performed by using a noninteractive system that assumed the foundation was rigid. Since that time, many authors have indicated that the compliance of a foundation can have a significant effect on the seismic response of a structure (see Ref. 3).

Two approaches may be used to model soil-structure interaction. A fully interactive approach attempts to account for feedback of the structural response to the soil medium as well as accounting for foundation compliance. This technique usually employs a nonlinear finite element characterization of the complete soil-structure system. Between the two extremes of noninteractive and fully interactive approaches is an approach which accounts for foundation compliance but utilizes a given free-field surface motion as an input forcing function. Using this approach, the fixity of the foundation to the soil medium can be determined independently from the loading conditions. A linear

foundation stiffness becomes a structural attribute and, as such, will be reflected in the natural frequencies and mode shapes of the structure. The methods employing this approach are frequently referred to as half-space methods and are applied to nonpile foundations as in Ref. 4, and to pile foundations as in Ref. 3. Some limitations of each method are summarized in Table 1.

It was concluded³ that, in comparison with finite element methods, half-space methods offer two major advantages. First, they capture the 3-dimensional response of the structure and, second, they are economical. Therefore, the approach employing half-space methods was utilized for the NFS facility. The resulting linear system is consistent with the response spectrum method of analysis chosen for the structure.

SOIL CONDITIONS AT THE NFS SITE

The process building is located on soil of glacial origin with the depth to bedrock varying from 70 to 110 ft. The pertinent soil properties for pile stiffness determination are those of the surface region for lateral stiffness and the regions near the pile tip for vertical stiffness. The soil data were supplied by Dames & Moore via Supplement 17 to the S.A.R.¹ which consisted of standard penetration

Table 1. Limitations of the half-space and finite element methods.

Half-Space method:	Finite element method:
<ul style="list-style-type: none"> ● Does not include material and radiation^a damping. ● Cannot model multilayer soil deposits easily. ● Neglects structure - structure interaction. ● Guidelines are presently scarce for determining dynamic spring stiffnesses. 	<ul style="list-style-type: none"> ● Appropriate three-dimensional codes are not available. Presently requires a plane strain solution. ● High-frequency information may be lost because of mesh size or the use of Rayleigh damping. ● The extent of the model may not be sufficient to eliminate wave reflections from boundaries. ● Analysis is more complex and expensive than for the half-space method.

^aRadiation damping accounts for the energy lost by radiation of waves from the base of the structure.

resistance from field tests correlated with relative density, using the Gibbs and Holtz relationship.

The surface material consists of a 15- to 25-ft. layer of medium dense to dense predominantly cohesionless soil. Therefore, the modulus is assumed to increase linearly with depth. The relative density varied from 55 to 95%, from which the coefficient of modulus variation, "f", was determined to be 25-55 tons/ft.³ (Ref. 5). These comprise the limiting soil values for which the lateral-pile stiffnesses were computed.

The piles terminate approximately 60 ft. below grade and rest either in stiff silty clay or very dense fine-to-coarse gravel and sand. Two

vertical load tests were performed on piles located under the process building, one at the north end and one at the south end. These showed that one pile behaved as an end-bearing pile while the other emulated friction-pile behavior. These, then, formed the bounding conditions for vertical pile stiffness.

The soil data necessary for calculating stiffnesses for embedded portions of the structure are shear modulus or shear wave velocity, and Poisson's ratio. As indicated in Ref. 6, the stiffnesses were calculated based on shear wave velocities determined from in situ measurement. Since these were determined at very low strains, they were used for upper

bound stiffnesses. To determine stiffnesses conforming to a higher strain, the shear modulus presented in the S.A.R.¹ (Page II-5-63) was used. This modulus was obtained from a torsionally excited, resonant column test. The lower bound stiffnesses were calculated by multiplying the previously calculated upper bound values by the ratio of the two moduli.

Determination of actual in-place soil properties applicable to the dynamic analysis of foundations is extremely difficult. The actual properties of soils are nonlinear (strain-dependent), frequency-dependent and load-history dependent. Therefore, all that can be obtained for an elastic analysis is a range of values. For the NFS facility, this range is incorporated into the calculated range of stiffness coefficients.

PILE STIFFNESS COEFFICIENTS FOR PROCESS BUILDING

Half-space methods require the evaluation of stiffness coefficients to approximate the fixity of the structure attached to the surface of a semi-infinite half-space. The most appropriate method to use for a structure like the process building utilizes stiffness coefficients directly to describe uncoupled linear springs. These are then applied to the foundation boundary nodes of the structural model.

For the determination of vertical pile stiffness, one must determine the form of axial pile-soil load transfer. For the process building, this varies from an end-bearing case to a constant friction case. Therefore, the stiffness varies from AE/L to $2AE/L$. These form the limiting vertical stiffness values used in the analysis for both the above and below grade portions of the structure. The stiffnesses are effective in both tension and compression in the mathematical model even though the stiffness of an actual pile subjected to uplift is not well known. In addition, the connection of the pile to the structure has a very low capacity in tension. Therefore, tension, or uplift, for a pile or pile group was considered a failure mechanism.

The lateral stiffness coefficients of piles can be calculated by two general methods: considering the pile embedded in an elastic half-space or the beam-on-elastic-foundation method. The most suitable application of the two methods are presented by Poulos for the first method, and by Reese and Matlock for the second method. The solutions incorporating a linearly increasing modulus with depth were utilized. These methods were described and evaluated in Ref. 3. The calculations are included in UCID-17453, and the results are tabulated in Table 2.

Table 2. Summary of lateral-pile stiffness.

		Min. value, K/ft.	Max. value, K/ft.
K_{HX}	Major axis	400	680
K_{HY}	Minor axis	250	490

The ranges of values indicated incorporate both the range of soil parameters and the use of two different methods of computation. These ranges are deemed to incorporate the effects of group action and cyclic load reversal and, thus, should bracket the actual behavior of the foundation system.

The next step in modeling the pile foundation was to incorporate the uncoupled springs into the structural model. This was accomplished by determining the tributary area of each structural node at the foundation, assuming a homogeneous stiffness distribution, and lumping the appropriate number of pile springs at that node. The resulting system resembles that shown in Fig. 7.

EMBEDDED STRUCTURE STIFFNESS COEFFICIENTS FOR LWC AND GPC

The embedded structures consist of portions of the Liquid Waste Cell (LWC) and the General Purpose Cell (GPC) which lie below grade. The LWC

Fig. 7. Groups of piles modeled as uncoupled linear springs.

is embedded 9.25 ft., and the GPC is embedded 24.5 ft. The LWC was idealized essentially as an axisymmetric problem, and the GPC was idealized as both a plane-strain and axisymmetric problem for the two axes of motion to determine embedded lateral stiffness coefficients. The 1/2-in. insulation board surrounding the GPC was assumed to have no effect on its stiffness.

Two solution techniques were employed:

- Utilization of stiffness coefficients for embedded circular and strip footings determined by static finite element techniques and presented by Johnson, Christiano, and Epstein.⁷
- Calculation of stiffness coefficients based on formulations by Parmelee et al.,^{8,9} for footings resting on a dynamically loaded half-plane, then modified for embedment effects.⁸

As stated by the authors,⁶⁻⁹ an average constant stiffness coefficient, independent of frequency, may be used to determine spring constants for discrete parameter soil-structure systems subjected to dynamic excitation.

The axisymmetric idealizations were formulated in two ways. For one, the equivalent radius was calculated from $\sqrt{\text{Area}/\pi}$, resulting in equal stiffness in the two horizontal directions. In an attempt to account for the rectangular shape of the embedded structures, an equivalent radius was calculated for each direction by dividing the area by the maximum length parallel to the direction of motion. This resulted in a set of values for each direction.

The resulting lateral soil springs were determined by dividing the total resistance among the nodes of each

embedded structure. First, the resistance was divided between the upper and lower portions as shown in Fig. 8. This was accomplished by considering the upper portion as a separate embedded foundation to determine its contribution in compression. This value was then subtracted from the total resistance, calculated earlier, to get the lower portion's contribution. The springs were applied to only one side of the footing because the modeled springs act both in tension and compression, while it was assumed that the soil acts in compression only. The upper and lower values were then distributed horizontally to each node based on its tributary area.

For the embedded portions, as for the piles, a range of stiffness values was calculated. This range incorporated two soil moduli, two solution techniques, and two or three geometric idealizations. The highest and lowest values were used in the seismic analysis. The calculations are included in UCID-17453, and the results are summarized in Table 3.

Table 3. Summary of embedded structure lateral stiffnesses.

Cell	Direction	Min. total resistance, K/ft	Max. total resistance, K/ft
GPC	N-S	5.4×10^5	46.5×10^5
	E-W	2.3×10^5	21.4×10^5
LHC	N-S	3.1×10^5	10.2×10^5
	E-W	3.5×10^5	11.8×10^5

Fig. 8. Embedded structure foundation model.

Seismic Analysis Methods

Seismic analyses of structural systems require 1) selection and application of a method of analysis to calculate the structural response; 2) construction of a mathematical model of the structure and foundation, and 3) definition of the loading. We discuss below the methods available for seismic analysis and the rationale used for selection of a method of analysis. We also indicate the choices we made for the NFS facility.

The purpose of the analysis is to estimate maximum stresses and displacements and to use these estimates to assess the safety of the structure. The methods of structural analysis used for seismic loading can be divided into two main categories: static and dynamic. The static methods of analysis attempt to furnish a distribution of seismic forces that approximate distributions obtained from dynamic analyses. The dynamic analyses include the inertia effects of the structure as well as the time-varying nature of the forces in the analysis.

Probabilistic methods are also available to conduct seismic analysis. However, further research in this area is required before these methods will be practical.

The methods of analysis we considered for the NFS facility are shown in Fig. 9.

STATIC METHODS

These methods of analysis are used to obtain a set of static lateral (horizontal) forces for structural design. The lateral forces are generally assumed proportional to the weight of the structure as

$$F = C'W,$$

where

F = total effective equivalent static lateral force,

W = weight of structure (includes dead weight plus percentage of live load assumed to be effective during an earthquake), and

C' = seismic coefficient.

The selection of the seismic coefficient depends on the seismicity and soil characteristics at the site as well as the dynamic characteristics, type, and function of the structure. The major difference between static methods is the way in which the seismic coefficient C' is obtained. There are two ways commonly used to obtain a value for C': the Uniform Building Code¹⁰ approach and the maximum horizontal ground-motion approach. Both of these methods assume the structural behavior is governed by the response of the first fundamental mode of vibration.

Fig. 9. Seismic analysis methods.

Uniform Building Code Approach

The method of seismic analysis specified in the Uniform Building Code is a static method intended for the design of most residential, industrial and commercial structures in the United States.

This approach provides minimum standards so that structures will resist:

- Minor earthquakes without damage.
- Moderate earthquakes without structural damage
- Major earthquakes without collapse (but possibly severe structural damage).

Many municipalities adopt the Uniform Building Code for minimum design requirements.

It is not reasonable to use the Uniform Building Code approach directly for the design or analysis of critical structures in nuclear facilities. These structures are different from those on which the Building Code is based. Moreover, critical nuclear facilities are required to function during and after a strong earthquake, with little or no structural damage.

Maximum Horizontal Ground-Motion Approach

Another approach to static analysis is to use the maximum horizontal acceleration postulated for the site as the seismic coefficient C' . This would better incorporate the local

seismicity of the area into the analysis but would not include the dynamic characteristics of the structure.

In many cases, this is an acceptable approach for stiff structures on rigid foundations (i.e., systems with frequencies greater than 33 Hz). The set of static forces for design could be obtained by the Building Code method of distribution.

This method would not be used if dynamic amplification or soil-structure interaction is an important factor.

DYNAMIC METHODS

Dynamic methods of analysis determine the distribution of forces in a structure by including the dynamic characteristics of the structure (mass, damping, and stiffness) in the equations of motion. The following methods are generally employed.

Response Spectrum Method

This is an approximate method of analysis that requires an acceleration response spectrum as a description of the ground motion. Mode shapes and frequencies of the mathematical model must be calculated.

The solution is obtained by calculating the response (i.e., displacement, acceleration, and stress) in each mode and then adding up the modal responses to get the total response.

This mode superposition is usually achieved by a square root of the sum-of-the-squares (SRSS) procedure.

This procedure is modified when closely spaced modes occur in a problem.¹¹ This method of analysis is approximate because of the way the modal quantities are combined to get the total response. The SRSS procedure of combining modal quantities is used since the maximum value in each mode may not occur at the same time. Enough modes must be included in the analysis to capture the complete structural response.

The advantages of this approach over static methods are 1) the mass and the stiffness characteristics of the structure can be included in the analysis, 2) the foundation compliance can be modeled (in a linear manner), and 3) the method permits the incorporation of the site response characteristics.

Time-History Methods

The time-history method of analysis is an exact method for determining the structural response to a known force-time history or acceleration-time history.

A time-history analysis is conducted by one of two approaches: mode superposition or direct integration. The mode superposition approach solves the eigenvalue problem associated with the model to determine the mode shapes

and frequencies of vibration. The response in each mode is determined and combined to get the total response. This method is limited to the linear response of structures.

The direct integration method is used to obtain a solution by step-by-step integration of the equations of motion directly. Solution of the eigenvalue problem is not required, and nonlinear effects may be included if required.

A direct integration analysis is equivalent to an analysis by the mode superposition approach in which all mode shapes and frequencies are included and the same time step Δt is used. Both methods involve a large amount of computer time to solve for the dynamic response of a structure since small time steps are required to achieve meaningful results. Both methods also require knowledge of acceleration-time history input. Table 4 compares some of the advantages and disadvantages of these methods.

ANALYSIS OF THE NFS FACILITY

For analysis of the NFS facility, we chose to conduct a dynamic analysis by the response spectra approach. We felt this provided a practical and economical way to get a realistic estimate of the structural response and the onset of failure without having

to choose or synthesize acceleration-time histories. This approach reduces the computation effort required to get a solution as well as the time required to display and interpret the results.

Our analysis will involve approximations since we are using a linear elastic procedure to assess the onset of failure. Theoretically, when one element fails, or commences inelastic behavior, a linear elastic analysis is no longer valid. In reality, however, a certain amount of nonlinear behavior will not appreciably affect the response of the rest of the structure. We feel our analysis will indicate weak areas in the structure and will provide estimates of the beginning of failure.

We examined the effect of the number of modes included in the study by running one case with 10 and 20 modes and comparing the displacement responses. Since both responses were reasonably close, we chose to use 10 modes for efficiency.

COMPUTER PROGRAMS USED IN STRUCTURAL ANALYSIS

The structural analysis can be performed by many available computer programs. We conducted our analysis with the SAP IV¹² program. SAP IV is a structural analysis program for computing the static and dynamic response of three-dimensional linear

Table 4. Advantages and disadvantages of methods of analysis.

Method	Advantages	Disadvantages
<div>Equivalent static</div> <div>Static</div>	<ol style="list-style-type: none"> 1. Structure usually statically determinate which greatly simplifies structural analysis. 	<ol style="list-style-type: none"> 1. No guarantee that equivalent static forces represent actual dynamic loading. 2. Cannot obtain time history of response.
<div>Response spectra</div> <div>Response spectra approach</div>	<ol style="list-style-type: none"> 1. Faster than both time-history methods for most problems with base acceleration. 2. Provides a simple way of determining maximum response quantities. Results reasonably accurate. 3. Spectra bounding many earthquake acceleration-time history may be used as input. 	<ol style="list-style-type: none"> 1. Solution of large eigenvalue problems can be time-consuming. 2. Must combine maximum modal quantities to get total solution. 3. Cannot obtain time history of response. 4. Limited to base acceleration inputs only. 5. Interpretation of stresses requires careful attention.
<div>Time history</div> <div>Direct integration of equations of motion</div>	<ol style="list-style-type: none"> 1. Can include nonlinear material and geometric effects. 2. For very large problems, may be faster than mode superposition methods. 3. Time history of response obtained directly. 4. Approximations involved in neglecting higher modes in mode superposition method not needed. 5. Results accurate for a given acceleration-time history. 	<ol style="list-style-type: none"> 1. Usually requires a large amount of computer time to solve problem. 2. Results from different acceleration-time histories both consistent with the same response spectra may be different.
<div>Integration of uncoupled modal equations</div>	<ol style="list-style-type: none"> 1. Faster than direct integration for most problems since solution may be expressed by first few modes. 2. Once frequencies and mode shapes are obtained, a variety of response analysis can be conducted efficiently. 3. Results accurate for a given acceleration-time history. 4. Time history of response obtained directly. 	<ol style="list-style-type: none"> 1. Solution of large eigenvalue problems can be time consuming. 2. Not applicable to non-linear analysis. 3. Results from different acceleration-time histories both consistent with the same response spectra may be different.

systems. The structural system that can be analyzed may be modeled with a combination of different structural elements. The program contains the following elements: truss element, beam element, plane stress, plane strain, and axisymmetric solid elements, 3-D solid element, thick shell element, thin plate or shell element, pipe element, and boundary element. They may be used to model problems for static or dynamic analyses.

SAP IV can solve static problems and has three solution options for dynamic problems: response spectra analysis, and time-history analyses by mode superposition, and direct integration.

We used SAMPP¹³ (Structural-Analysis Movie Postprocessor) in conjunction with PREPOST¹⁴ for plotting the finite element model and its response. PREPOST processes SAP IV results for input to SAMPP and calculates maximum shear stresses.

SAMPP is a general purpose routine for the display of three-dimensional, finite element models and the results from both static and dynamic analyses. The program may be used to produce line drawings as well as animated line-drawing movie sequences. SAMPP uses a hidden-surface processor for the display of realistic perspective views.

Structural Loading

DEAD LOAD

A static dead-weight analysis was conducted on the model to estimate the initial state of stress prior to the seismic event. This analysis included the weight of the structure and supported equipment. The weight was increased uniformly to account for the percentage of live load that may be acting on the structure during an earthquake.

Our estimates of dead weight are as accurate as the construction drawings supplied to us by NFS for the

primary structural elements within the constraints of our model. Data supplied by DRAVO for equipment masses were used directly.

SEISMIC LOAD

We needed seismic input for the structural analysis. Since there are no existing guidelines for the selection of the seismic input for fuel reprocessing plant structures, we had two alternatives: 1) to select recorded earthquake accelerograms and generate response spectra, or 2) to

derive an accelerogram that statistically represents a typical accelerogram. We have chosen the latter approach. Rather than perform the statistical calculations ourselves, we have, for convenience, used the information in USNRC Regulatory Guide 1.60.¹⁵

Guide 1.60 defines the seismic input to nuclear power reactors based on a statistical treatment of some 50 earthquake records. It gives both the horizontal and vertical response spectra for different values of structural damping. Both are normalized to a maximum horizontal ground acceleration of 1.0 g. Once the intensity of ground motion for a site is specified in terms of maximum horizontal ground acceleration, both horizontal and vertical spectra can be defined simply by scaling.

The spectra given in Guide 1.60 are based on values one standard deviation over the calculated mean values of spectral acceleration. Guide 1.60 is intended for sites underlain by either rock or soil deposits, and it covers all frequencies of interest. For unusually soft sites, modification of this procedure is required.

We have used the Guide 1.60 values to define our input response spectrum. There are no data for determining whether this spectrum is conservative for the NFS site or not. However,

the same spectrum is used for design of nuclear power plants in the Eastern United States so its use at the NFS site is considered reasonable.

Damping values used are consistent with Regulatory Guide 1.61¹⁶ (shown in Table 5), which delineates damping values acceptable for elastic dynamic analysis. The damping values account for energy dissipation and reflect both material and structural damping for stresses less than yield.

In this study we selected a damping ratio of 7% to account for the reinforced concrete structure and the foundation interaction.

The horizontal acceleration response spectrum used in this study is shown in Fig. 10. This spectrum was used as input in both the north-south and east-west directions, and two-thirds of it was used as input for the vertical motion. The input motion was applied to the structure uniformly at foundation level.

In Fig. 11, we compared the Regulatory Guide spectrum with the spectrum at foundation level computed from an analysis of the West Valley site conducted by LASL.¹⁷

The shapes of these spectra are comparable in the range of periods of 0- to 0.45-sec. At periods greater than 0.45 sec, the LASL generated spectrum bounds the Regulatory Guide spectrum. The range of fundamental periods of the four cases we studied

Table 5. Damping values.¹⁶

Structure or component	Operating basis earthquake or 1/2 safe shutdown earthquake	Safe shutdown earthquake
	Percent of critical damping	
Equipment and large-diameter piping systems, pipe diameter greater than 12 in.	2	3
Small-diameter piping systems, diameter equal to or less than 12 in.	1	2
Welded steel structures	2	4
Bolted steel structures	4	7
Prestressed concrete structures	2	5
Reinforced concrete structures	4	7

Fig. 10. Horizontal acceleration response spectrum normalized to 1g used in the analysis of the NFS facility.

Fig. 11. Comparison of Regulatory Guide 1.60 spectrum with LASL-generated spectrum.

was 0.22 to 0.50 sec. In this range, the spectra are comparable. Thus, the application of the Guide 1.60 surface spectrum at foundation level appears reasonable.

COMBINATION OF EARTHQUAKE COMPONENTS

When a response spectrum analysis is conducted, there are two methods commonly used to combine the structural response caused by horizontal and vertical earthquake motions. These methods are summarized below:

- Method of Linear Additions - The maxima in each direction are combined directly. This method provides a conservative estimate of the response since each of the maxima in the three orthogonal directions do not necessarily occur simultaneously.
- Square Root of the Sum-of-the-Squares (SRSS) Method - This

method attempts to estimate the most probable response of the structure by combining the maximum values in a way that recognizes that these values do not occur simultaneously.

For analysis of the NFS facility, we chose to use the more conservative component combination scheme: the method of linear additions. This decision was made to keep the number of computer runs down and to avoid developing a program to perform the SRSS procedure for earthquake component combinations.

COMBINATION OF DEAD AND SEISMIC LOADING

Results of the seismic analyses were combined with the dead-load analysis results to estimate the worst-case loading situation. Results were not scaled by load factors.

Analyses Conducted

We conducted a static dead-load analysis and several dynamic analyses (all linear-elastic) to determine the total state-of-stress, including foundation effects, during an earthquake.

The seismic analysis of this facility resulted in the solution of

four separate problems. These four cases developed from considering the upper- and lower-bound values for embedded structure spring stiffness and pile spring stiffness (vertical as well as lateral). These maximum and minimum values were determined to bound the variations expected in

soil-structure interaction. These four cases are summarized in Table 6.

We then combined the dead-load results with the results of the four dynamic analyses. Acceleration levels at the onset of failure were determined by comparison with threshold of failure values. The threshold of failure shear values were determined as follows:

$$\tau_{\max} = \sqrt{\left(\frac{\sigma_x \pm \sigma_y}{2}\right)^2 + \tau_{xy}^2},$$

where

σ_x, σ_y = normal stress components in the x and y directions,
 τ_{xy} = shear stress acting with the normal stresses.

The response spectrum analysis gives the magnitude of $\sigma_x, \sigma_y, \tau_{xy}$; however, their sign is lost.¹⁸ This fact results in two unique values for the calculated maximum shear stress. The maximum of the two values is the quantity compared with the ultimate values.

Table 6. Summary of foundation model cases for seismic analysis.

File	Embedded structures	Minimum embedded structure stiffness	Maximum embedded structure stiffness
		<u>Case 3</u>	<u>Case 1</u>
Minimum pile stiffness		This case was believed to provide the maximum load to the upper areas of the facility (shear-wall failures).	This case was selected to represent the worst loading for the embedded structures and also represent pile tension failure from rocking.
		<u>Case 2</u>	<u>Case 4</u>
Maximum pile stiffness		This case was believed to represent the worst loading case for lateral pile springs (Lateral Pile Failure).	It was felt that this case could produce shear wall failures where cases 1 and 3 did not.

Failure Definition

The purpose of conducting the seismic analysis was to determine the peak ground acceleration level that would

initiate failure of the structure. The primary consideration for evaluating the performance of the structure

was its ability to provide confinement for radioactive materials. Therefore, it was decided that excessive cracking of elements providing confinement would provide the primary criterion for determining the onset of failure. It is our belief that state-of-the-art methods for seismic analysis are not sufficiently developed to allow determination of the size of cracks formed during an earthquake. The second area for investigation was to point out regions of structural weakness, not necessarily directly related to confinement.

It is important to recognize that an analysis to determine actual failure of an existing structure is considerably different from an analysis for designing a new structure. For design, load factors and conservative assumptions are used to ensure that a failure level will not be reached. For failure analysis, one must attempt to determine the conservatism and safety factors in allowable or ultimate values specified in building codes so that they may be removed. For example, Fig. 12 gives an indication of differences between the actual failure levels of concrete shear walls and the strengths predicted by design formulas.

The potential modes of failure for the NFS Process Building include:

- Reinforced concrete shear wall failure.

- Concrete block shear failure.
- Foundation failure.
- Large relative displacements at sliding joints.

The fourth failure mode will not be addressed herein because the model has rigid connections and cannot simulate sliding joints. The criteria for evaluating the first three failure modes will be discussed separately.

REINFORCED CONCRETE SHEAR WALL FAILURE

The onset of failure for reinforced concrete was defined to be the point where the unfactored loads became equal to the ultimate strength of the structural elements. The ultimate shear strength was computed based on provisions of the ACI specifications,²⁰ ACI background paper,¹⁹ SEAOC code,²¹ Joint ASCE-ACI Task Committee 426 (ref. 22), and Uniform Building Code.¹⁰ The code ultimate-strength values are modified by a capacity reduction factor ϕ , which provides for variations in material strength, workmanship, and dimensions. It is generally recognized that the strength of concrete increases with age. This, combined with an assumption of standard workmanship led to the adoption of $\phi = 1$ for process building components.

The ACI provisions for shear capacity relates primarily to static cases. The ultimate capacity, V_u , is composed of two components:

Fig. 12. Comparison of measured and calculated shear strengths (f'_c = compressive strength of the concrete and v_u = ultimate shear strength).

$$v_u = v_c + v_s$$

where v_c is the capacity of the concrete in diagonal tension and v_s is the capacity of the horizontal reinforcement in the wall. The shear capacity of the concrete is influenced by axial and flexural forces. For NFS, it was assumed that the crosswalls

act as flanges to resist any flexural and axial forces. Therefore, axial and flexural contributions were not considered in the calculation of shear capacities.

ACI^{19,20} indicates that v_c may be taken as $2\sqrt{f'_c}$ if the wall is not in tension. The SEAOC code says that the ultimate concrete shear should not

exceed $v_c = 2\sqrt{f'_c}$. This is also stated in the Committee 426 report for walls subjected to load reversals, which also states that the total v_u should be limited to $8\sqrt{f'_c}$. It was the consensus of the references that to develop fully the capacity of the horizontal reinforcement in resisting shear forces, an equal amount of vertical reinforcement should be provided. Therefore, for this structure, only the portion of horizontal steel area matched by vertical steel was used to determine the shear capacity caused by the steel. That is,

$$v_s = \rho_{\min} f_y$$

where $\rho_{\min}$ was determined for each element by taking the minimum ratio of steel area to gross concrete area, considering both vertical and horizontal steel as shown on the plans and f_y = yield strength of the reinforcement. Thus, the ultimate shear strength was determined from

$$v_u = v_c + v_s$$

or

$$v_u = 2\sqrt{f'_c} + \rho_{\min} f_y$$

where

f'_c = compressive strength of the concrete (2500 psi),

f_y = yield strength of the reinforcing steel (60 ksi),

$\rho_{\min}$ = minimum of the ratio of shear reinforcement to gross concrete area in either the horizontal or the vertical direction.

The resulting ultimate shear capacities may seem conservative based on code static design values. However, consideration of energy absorption, flexural strength, and reinforcement details dictate these stress levels.

CONCRETE BLOCK SHEAR WALL FAILURE

The allowable working stress values for masonry walls in shear were calculated in accordance with the provisions of the Reinforced Masonry Engineering Handbook.²³ The following assumptions were employed:

- The ultimate compressive masonry strength, $f'_m = 1500$ psi.
- All masonry units are grouted solid with concrete of grade A or N.
- Special inspection was employed.
- All shear stresses are carried by masonry and grout, none by reinforcement.

The measure of failure of the concrete block shear walls was the attainment of their ultimate strength, which was estimated by multiplying the working stress values by a factor of two. This value was obtained from people

knowledgeable in the field of masonry construction.²⁴

FOUNDATION FAILURE

The direct consequences of failure of portions of the foundation are difficult to determine. However, there are two potential effects of foundation failure. One is the effect on the integrity of the rest of the structure, and the other is the effect on the validity of an elastic analysis subsequent to failure. As a result of discussions with NRC and LASL,² it was decided that 10 - 15% of the piles could fail before the rest of the structure would experience a significant effect. For this study, we decided to report the onset of foundation failure by using the 10% figure while allowing the elastic analysis to continue until the 15% failure level was reached.

The failure modes for piles were divided into two parts, vertical and lateral. The vertical failure mode occurs as a result of the connections between the piles and the structure. There is virtually no tensile capacity for this joint which could result in separation of the building and pile during rocking motion. Impact loads that could result during subsequent load cycles were considered unacceptable. Therefore, the attainment of tension in a pile was defined as failure for that pile.

A lateral pile failure could result from pile-cap tearout, shear in the pile, or axial load-bending moment interaction. The controlling failure mode was the onset of yielding caused by axial load-moment interaction. The moments in the piles subjected to lateral force were determined by beam-on-elastic-foundation techniques. The calculations for the various conditions and the resulting maximum capacities are given in UCID-17453.

DETERMINATION OF FAILURE LEVEL

The initial structural loading was determined by conducting a static dead-load analysis as well as a 0.2g peak ground acceleration dynamic analysis for each case. Because a linear elastic response spectrum analysis technique was employed, dynamic loads for other peak ground accelerations may be obtained by direct proportion.

The stresses from the structural loading were compared with ultimate values for each of the elements. If the maximum ultimate value for an element was exceeded, the dynamic load contribution was linearly proportioned until the sum of static and dynamic loads equaled the ultimate. This defined the peak ground accelerations at the onset of failure, which are the values reported.

Results

The analysis of the NFS facility yielded several points of interest. One interesting item is that by varying the parameters that govern the soil-structure interaction phenomena, any one of the first three modes of vibration can be tuned to the period where the peak spectral acceleration occurs. Fig. 13 illustrates rocking

and twisting modes referenced to the major axes of the structure. Figures 14 through 17 show how the periods of the structure vary from one foundation case to another. It can be shown that the first three modes for each of the cases are caused primarily by soil-structure interaction. For example, the fundamental

Fig. 13. Major axes of the structure showing rocking and twisting mode.

Fig. 14. Case 1 input spectrum with periods of the first ten modes super-imposed.

Fig. 15. Case 2 input spectrum with periods of the first ten modes superimposed.

Fig. 16. Case 3 input spectrum with periods of the first ten modes superimposed.

Fig. 17. Case 4 input spectrum with periods of the first ten modes superimposed.

mode for case one (Fig. 14) is primarily a rocking mode about a north-south axis. Mode two is primarily a rocking mode about an east-west axis while mode three is twisting about a vertical axis. The first three modes for cases two and three (Figs. 15 and 16) involve essentially the same type of motions. Mode one is basically twisting about a vertical axis while modes two and three are primarily translations back and forth in an east-west and north-south direction, respectively. Case four (Fig. 17) has a fundamental mode that is primarily twisting about a vertical axis while modes two and three are a combination of rocking (about a north-south axis and about an east-west axis, respectively) and twisting about a vertical axis.

The basic items of concern for this analysis are the various types of failure. The investigation determined that there are four potential types of failure. These are shear wall and pile tension failure, lateral failure of piles, and PMC-to-GPC connection failure.

SHEAR WALL FAILURE

The shear walls for the facility consist of reinforced concrete block walls as well as the poured-in-place reinforced concrete walls of the major cells. The onset of failure for the

elements that exceed their maximum ultimate shear stress at or below a peak ground acceleration of 0.2g are given in Table 7. The location of some of these elements within the facility as well as the ranges where the onset of failure occurs are shown for the various cases in Figures 18 through 21. The ultimate shear stress value for each element is given in UCID-17453.

The data generated for the four cases indicate that the concrete block shear walls begin to fail at about 0.03 to 0.07g. The two areas where this occurs are the shielding door tower (0.03g) on top of the CPC and the exterior walls of the AOC, (0.07g).

The onset of failure for the reinforced concrete walls first occurs in the GPC at about 0.09g. The other major cells that have elements where the onset of failure occurs prior to a 0.2g ground acceleration are the LWC and the CPC.

If the maximum allowable shear stress in the south wall of the CPC is 50 psi (as indicated in the Dravo Report), rather than 267 psi, our analysis indicates the onset of failure for these elements will occur at approximately 0.08g.

PILE TENSION FAILURE

Figures 22 through 25 show the location of the modeled pile springs

Table 7. Peak ground acceleration where the onset of failure in shear elements occur (g's).

Cell	Elem. no.	Case 1, g	Case 2, g	Case 3, g	Case 4, g	Min g	Absolute min, g
AOC (concrete block)	2	0.08	0.11	0.10	0.08	0.08	0.07
	3	0.15	0.20	0.19	0.14	0.14	
	5	0.12	0.17	0.17	0.12	0.12	
	6	0.16	0.25	0.23	0.17	0.16	
	8	0.11	0.17	0.16	0.11	0.11	
	11	0.07	0.08	0.08	0.07	0.07	
	14	0.07	0.07	0.07	0.07	0.07	
	15	0.19	0.24	0.23	0.19	0.19	
	20	0.12	0.14	0.14	0.12	0.12	
CPC	24	0.11	0.16	0.17	0.12	0.11	0.09
	25	0.12	0.15	0.14	0.11	0.11	
	43	0.19	0.15	0.16	0.15	0.15	
	181	0.09	0.29	0.30	0.10	0.09	
	186	0.09	0.29	0.31	0.10	0.09	
	223	0.09	0.22	0.25	0.10	0.09	
	231	0.24	0.24	0.24	0.19	0.19	
	233	0.12	0.35	0.35	0.13	0.12	
	273	0.09	0.10	0.09	0.09	0.09	0.09
GPC	274	0.14	0.14	0.14	0.13	0.13	
	276	0.17	0.16	0.15	0.16	0.15	
	289	0.17	0.15	0.15	0.15	0.15	
	290	0.17	0.15	0.14	0.13	0.13	
	291	0.11	0.11	0.11	0.10	0.10	
	308	0.13	0.21	0.28	0.17	0.13	
	309	0.06	0.12	0.26	0.15	0.06	
	310	0.03	0.06	0.12	0.08	0.03	
	311	0.06	0.12	0.21	0.15	0.06	0.03
TOP (concrete block)	312	0.07	0.13	0.22	0.14	0.07	
	331	0.13	0.29	0.71	0.40	0.13	
	332	0.07	0.14	0.26	0.18	0.07	
	333	0.14	0.31	0.72	0.41	0.14	
	334	0.07	0.16	0.43	0.24	0.07	

Case 1 = minimum pile/maximum embedded stiffnesses.

Case 2 = maximum pile/minimum embedded stiffnesses.

Case 3 = minimum pile/minimum embedded stiffnesses.

Case 4 = maximum pile/maximum embedded stiffnesses.

- Onset of failure @ 0.03 - 0.08g
- Onset of failure @ 0.08 - 0.12g
- Onset of failure @ 0.12 - 0.16g
- Onset of failure @ 0.16 - 0.20g

Fig. 18. Case 1 onset of shear wall failure.

- Onset of failure @ 0.03 - 0.08g
- ▨ Onset of failure @ 0.08 - 0.12g
- ▩ Onset of failure @ 0.12 - 0.16g
- ▧ Onset of failure @ 0.16 - 0.20g

Fig. 19. Case 2 onset of shear wall failure.

- ▨ Onset of failure @ 0.03 - 0.08g
- ▩ Onset of failure @ 0.08 - 0.12g
- ▤ Onset of failure @ 0.12 - 0.16g
- ▧ Onset of failure @ 0.16 - 0.20g

Fig. 20. Case 3 onset of shear wall failure.

- ▨ Onset of failure @ 0.03 - 0.08g
- ▤ Onset of failure @ 0.08 - 0.12g
- ▥ Onset of failure @ 0.12 - 0.16g
- ▧ Onset of failure @ 0.16 - 0.20g

Fig. 21. Case 4 onset of shear wall failure.

- ▲ Onset of failure @ 0.08 - 0.12g
- Onset of failure @ 0.12 - 0.16g
- Onset of failure @ 0.16 - 0.20g

Fig. 22. Case 1 pile tension failure.

- ▲ Onset of failure @ 0.08 - 0.12g
- Onset of failure @ 0.12 - 0.16g
- Onset of failure @ 0.16 - 0.20g

Fig. 24. Case 3 pile tension failure.

- ▲ Onset of failure @ 0.08 - 0.12g
- Onset of failure @ 0.12 - 0.16g
- Onset of failure @ 0.16 - 0.20g

Fig. 23. Case 2 pile tension failure.

- ▲ Onset of failure @ 0.08 - 0.12g
- Onset of failure @ 0.12 - 0.16g
- Onset of failure @ 0.16 - 0.20g

Fig. 25. Case 4 pile tension failure.

that experience a tensile loading with a peak earthquake ground acceleration of less than or equal to 0.2g. Note that the modeled springs incorporate varying numbers of piles. The load cases that used the upper-bound soil spring stiffnesses (cases one and four) resulted in the greatest number of pile spring failures before a 0.2g acceleration was reached. Ten percent of the piles failed in tension at 0.17g.

Table 8 tabulates, for each of the load cases, the pile springs that fail in tension and the g-level intensity when tension first occurs. From these tables, it can be seen that pile tension first occurs at a 0.11g peak ground acceleration (cases one and four). The failure sequences that result when a single pile spring fails is difficult, if not impossible, to predict. Figures 22 through 25 indicate, however, that several pile springs in concentrated areas indicate tensile loads. At these areas, other related structural damage is expected.

LATERAL FAILURE OF PILES

Figures 26 and 27 show the location of modeled pile springs that exceed their maximum lateral load capacity at a peak ground acceleration $\leq 0.2g$. The load cases where the minimum soil spring values were used (cases two and three) were the only cases where lateral pile failure occurred.

Table 9 tabulates for each of the load cases the pile springs that exceed their lateral capacity and the g-level when these values were exceeded. From these tables it can be seen that the initial pile spring exceeded the allowable value for case two at 0.05g. The initial failure for case three occurred at 0.07g. Ten percent of the piles failed due to lateral load at 0.11g.

PMC TO GPC CONNECTION

Our analysis of the PMC to GPC connection indicates an adequate margin of safety.

Table 8. Pile tension failure.

CASE 1

Foundation node number	Pile element number	Number of piles	Total piles	"g" level
352	204	1	1	0.111
217	104	1	2	0.121
220	107	1	3	0.123
339	200	1	4	0.124
297	160	4	8	0.128
354	207	3	11	0.130
296	157	3	14	0.135
250	137	4	18	0.138
358	213	1	19	0.140
223	110	4	23	0.144
226	113	2	25	0.146
65	47	2	27	0.147
355	210	2	29	0.147
61	44	2	31	0.150
89	62	1	32	0.151
103	65	1	33	0.153
121	72	1	34	0.156
139	76	2	36	0.157
57	41	3	39	0.160
363	222	1	40	0.162
45	32	2	42	0.167
360	216	8	50	0.168
----- 10%				
53	38	3	53	0.170
68	50	2	55	0.173
344	201	1	56	0.180
41	29	3	59	0.181
49	35	7	66	0.183
299	166	3	69	0.183
74	56	1	70	0.188
232	119	2	72	0.189
257	140	8	80	0.189
----- 15%				
29	20	3	83	0.190
298	163	5	88	0.190
37	26	1	89	0.192
229	116	9	98	0.192
71	53	4	103	0.193
142	79	3	106	0.194
207	100	9	115	0.195
157	82	1	116	0.197
364	225	4	120	0.198

Table 8. (Cont).

CASE 2

352	204	1	1	0.129
354	207	3	4	0.155
358	213	1	5	0.159
220	107	1	6	0.161
363	222	1	7	0.165
339	200	4	11	0.168
65	47	2	13	0.170
217	104	1	14	0.172
89	62	1	15	0.175
61	44	2	17	0.178
103	65	1	18	0.178
355	210	2	20	0.180
121	72	1	21	0.181
139	76	2	23	0.181
226	113	2	25	0.189
57	41	3	28	0.199 ^a

^aLess than 10% of the piles failed at 0.2g

CASE 3

Foundation node number	Pile element number	Number of piles	Total piles	"g" level
352	204	1	1	0.127
339	200	1	2	0.142
358	213	1	3	0.154
354	207	3	6	0.156
363	222	1	7	0.159
65	47	2	9	0.170
220	107	1	10	0.178
89	62	1	11	0.180
355	210	2	13	0.185
103	65	1	14	0.186
217	104	1	15	0.186
74	56	1	16	0.188
344	201	1	17	0.189
68	50	2	19	0.193
61	44	2	21	0.195
121	72	1	22	0.197
360	216	8	30	0.197 ^a
139	76	2	32	0.199 ^a

^aLess than 10% of the piles failed at 0.2g

Table 8. (Cont.).

CASE 4

Foundation node number	File element number	Number of piles	Total piles	"g" level
217	104	1	2	0.118
352	204	1	3	0.119
226	113	2	5	0.128
65	47	2	7	0.133
89	62	1	8	0.137
223	110	4	12	0.138
354	207	3	15	0.138
103	65	1	16	0.139
61	44	2	18	0.141
121	72	1	19	0.142
139	76	2	21	0.143
339	200	1	22	0.145
250	137	4	26	0.146
358	213	1	27	0.148
296	157	3	30	0.151
355	210	2	32	0.153
57	41	3	35	0.156
45	32	2	37	0.158
232	119	2	39	0.160
363	222	1	40	0.161
297	160	4	44	0.164
68	50	2	46	0.166
53	38	3	49	0.169
41	29	3	52	0.178
<hr/>				
229	116	9	61	0.178
29	20	3	64	0.179
360	216	8	72	0.181
49	35	7	79	0.186
<hr/>				
142	79	3	82	0.188
157	82	1	83	0.193
37	26	1	84	0.194
238	125	3	87	0.194
74	56	1	88	0.195
71	53	4	92	0.197
257	140	8	100	0.199

10%

15%

- ◆ Onset of failure @ 0.05 - 0.08g
- ▲ Onset of failure @ 0.08 - 0.12g
- Onset of failure @ 0.12 - 0.16g
- Onset of failure @ 0.16 - 0.20g

Fig. 26. Case 2 lateral pile failure.

- ◆ Onset of failure @ 0.05 - 0.08g
- ▲ Onset of failure @ 0.08 - 0.12g
- Onset of failure @ 0.12 - 0.16g
- Onset of failure @ 0.16 - 0.20g

Fig. 27. Case 3 lateral pile failure.

Table 9. Lateral failure of piles

CASE 3

Foundation node number	Pile element number	Number of piles	Total piles	"g" level
379	253	12	12	0.071
68	49	2	14	.104
74	55	1	15	.104
363	221	1	16	.121
352	203	1	17	.134
358	212	1	18	.134
364	224	4	22	.134
371	236	4	26	.134
65	46	2	28	.142
89	60	1	29	.150
103	63	1	30	.150
121	70	1	31	.150
71	52	4	35	.155
307	183	6	41	.157
13	9	2	43	.162
29	19	3	46	.162
45	31	2	48	.162
220	106	1	49	.162
-----10%				
226	112	2	51	.162
232	118	2	53	.162
238	124	3	56	.162
244	130	3	59	.162
142	77	3	62	.167
157	80	1	63	.167
86	58	1	64	.171
139	75	2	66	.178
223	108	4	70	.181
250	135	4	74	.181
-----15%				
300	167	2	76	.181
53	36	3	79	.183
57	39	3	82	.183
41	28	3	85	.184
113	67	3	88	.184
217	103	1	89	.184
296	155	3	92	.189
335	196	13	105	.197
354	205	3	108	0.197

Table 9. (Cont.).

CASE 2

Foundation node number	Pile element number	Number of piles	Total piles	"g" level
379	253	12	12	0.050
68	49	2	14	.067
74	55	1	15	.067
363	221	1	16	.077
352	203	1	17	.084
358	212	1	18	.084
364	224	4	22	.084
371	236	4	26	.084
307	183	6	32	.098
65	46	2	34	.099
89	60	1	35	.100
103	63	1	36	.100
121	70	1	37	.100
142	77	3	40	.110
157	80	1	41	.110
13	9	2	43	.111
29	19	3	46	.111
45	31	2	48	.111
71	52	4	52	.111
<hr/>				
220	106	1	53	.111
226	112	2	55	.111
232	118	2	57	.111
238	124	3	60	.111
244	130	3	63	.111
300	167	2	65	.119
53	36	3	68	.121
57	39	3	71	.121
139	75	2	73	.122
187	89	12	85	.123
<hr/>				
193	92	12	97	.123
41	28	3	100	.124
217	103	1	101	.124
86	58	1	102	.126
223	108	4	106	0.126

10%

15%

Table 9. (Cont.).

250	135	4	110	0.126
335	196	13	123	.129
113	66	3	126	.130
354	205	3	129	.130
197	95	13	142	.131
297	159	4	146	.132
299	165	3	149	.132
37	25	1	150	0.133

Conclusions

Two types of failure were identified in our analysis of this facility: 1) shear wall failure and 2) foundation failure.

Listed below are estimates of the onset of failure and their locations. These results are presented to the Nuclear Regulatory Commission to assist them in evaluating the radiological consequences of these failures. We have tried to include our intuition into the structural consequences of these failure estimates.

SHEAR WALL FAILURE

- Parts of the WC start to fail at 0.07g. This portion of the building is of concrete block construction and could experience enough damage to collapse.
- One wall of the CPC begins to fail at 0.15g. (0.08g if we

use the reduced allowable shear stress suggested by DRAVO). This is an above ground structure and, thus, may cause a breach in confinement.

- Parts of the LWC fail starting at 0.09g. These regions are below grade; thus, any liquid release would probably seep into the soil. This is a lower bound because of the way our model treats interconnections. We feel a more realistic value for this cell is the average value of 0.17g.
- Portions of GPC begin to fail at 0.09g. This cell is also located below grade.
- Concrete block structures on top of the CPC begin to fail at 0.03g.

Those shear wall elements exceeding ultimate values may experience

progressive cracking failure depending on the number of peak load cycles. This may result in significant breaches in confinement walls. The importance relative to confinement of a particular element is not known; however, the failure of secondary elements such as shafts, shielding doors, or control rooms, could affect the ability to monitor and control the plant during and after a seismic event.

It appears that the damage level experienced by the reinforced concrete processing portions of the structure at the onset of failure would not cause them to "fall down" under a 0.2g earthquake. However, the progressive failure nature of concrete subjected to cyclic loading could ultimately result in the building's demise. Because of the small ratio of reinforcement in this building, a significant reserve capacity or ductility is not available to dissipate the energy imparted by a long-duration 0.2g earthquake.

The office portions of the building have not been included in this analysis. However, as evidenced by the performance of other concrete block portions of the building, they

will most likely experience heavy damage, including possible collapse. This probably will not cause a loss of confinement but could result in loss of life for its occupants.

FOUNDATION FAILURES

Our analysis indicated lateral pile failure to begin at 0.11g while pile tension failure begins at 0.17g. The effects of these pile failures are also not known. However, this might initiate a progressive failure which could undesirably redistribute forces to critical areas of the structure.

MODIFICATIONS TO IMPROVE SEISMIC RESISTANCE

Extensive analyses beyond the scope of this project would be needed to recommend modifications for strengthening the existing facility. Except for some accessible shear walls, this structure would be difficult to upgrade for seismic resistance.

If upgrading were undertaken, it should also provide adequate protection against other natural phenomena such as floods, extreme winds, and tornadoes.

Areas for Further Investigation

Possible future refinement of this analysis might be productive in the following areas:

- Definition of seismic loading.
- Model refinement.
- Analysis refinement.

DEFINITION OF SEISMIC LOADING

Investigations could be undertaken to develop site-specific spectra at various ground acceleration levels. These investigations are both time consuming and expensive but could refine the seismic input.

MODEL REFINEMENT

Review of our results indicates the structural response is strongly influenced by the stiffness coefficients of the embedded structures. The range in values we used varied by one order of magnitude. To reduce their range, more time could be spent reviewing the literature and comparing available methods for determining these stiffness coefficients.

Refined zoning (smaller elements to model shear walls) would also improve the accuracy of the calculated stresses. This may be currently possible with substructuring techniques or in the near future with larger computers and more efficient computational tools.

The model could also attempt to account for some flexibility in the interconnections between various cells. An upper bound estimate on the period could be obtained by allowing no interconnections between some cells. The model could be further refined by including linear or nonlinear springs between various

cells to model interconnections. Non-linear springs that become infinitely stiff in compression and that model frictional resistance in tension could improve estimates of failure.

REFINE ANALYSIS

With a refined model, a series of time-history analyses could be conducted to identify better the onset of failure and to study battering between adjacent structures. This could be extended further to study failure progression by conducting a series of elastic analyses where failed elements could be removed at appropriate times. Upon removal of failed elements, the g-level could be increased and the structure stiffness recalculated. By currently available methods, this would be an expensive and laborious procedure.

Other refinements in analysis that could be conducted by current techniques include: 1) combining multi-directional input by the SRSS procedure, 2) using the techniques of Regulatory Guide 1.92 for combining closely spaced modes, and 3) incorporating nonproportional damping to allow different damping values to be used for the structure and the foundation. However, we feel these effects are of secondary importance and, in general, we chose a conservative approach.

Definition of Seismic Related Terms

THE DYNAMIC PROBLEM

In describing the seismic analysis of the NFS Facility, we introduced terms which may require clarification. We constructed the mathematical model using shear walls, floor diaphragms, and uncoupled linear springs. A dynamic analysis was conducted to determine the modes of the structure including the mode shapes and periods. The loading was represented by a response spectrum for a specified damping value where the free-field ground spectrum was defined by the Regulatory Guide 1.60 spectrum. After applying the load to the structural model, the responses from each mode are combined by the SRSS procedure.

SHEAR WALLS

A building can resist horizontal forces by one or a combination of two principal structural components: frames of concrete or steel, or reinforced concrete shear walls. Shear walls are deep, relatively thin, vertically cantilevered beams. The normal assumption for building analysis is that they resist horizontal forces entirely by in-plane shear. That is, normal-to-the-plane bending does not contribute significantly to lateral resistance.

FLOOR DIAPHRAGMS

The inertial forces induced in various portions of the building during an earthquake must be transmitted to the lateral-force resisting system, in this case, the shear walls. These forces are transmitted by the floors (and roof) via diaphragm action. A diaphragm is a rigid horizontal two-dimensional element that transmits load by in-plane action only.

UNCOUPLLED LINEAR SPRINGS

The flexibility of the foundation was modeled by uncoupled linear springs. Each of these springs is an element in which its resistance is proportional to its axial extension only. They are uncoupled in that displacements along other axes or rotations do not affect their response.

MODE, MODE SHAPE, AND PERIOD

Every structural system has associated with it the attribute of normal modes of vibration, which are a function of its mass and stiffness distribution. A system has exactly the same number of normal modes as degrees of freedom. Associated with each mode is a natural period and a

characteristic shape. The distinguishing feature of a normal mode is that the system could, under certain circumstances, vibrate freely in that mode alone. During this free vibration, the ratio of the displacements of any two masses is constant with time. These ratios define the characteristic mode shape. The period of a mode is the time it takes for one complete oscillation during free vibration of that mode. The frequency of a mode is the number of oscillations per unit of a time (the reciprocal of period).

SINGLE-DEGREE-OF-FREEDOM SYSTEM (SDOF)

The dynamic response of a system subjected to a specified base motion may be estimated by assuming that its mass, stiffness, and damping properties are concentrated into single physical elements. This model is called a single-degree-of-freedom system (Fig. 28). Once the system has been modeled, a response spectrum can be used directly as input motion to determine the maximum response.

MULTIPLE-DEGREE-OF-FREEDOM SYSTEM (MDOF)

Often, dynamic response of a real structure cannot be described with single-degree-of-freedom systems. True behavior can only be described in terms of more than one displacement

Fig. 28. Single-degree-of-freedom system. Mass concentrated at single point M. Damping and stiffness characteristics concentrated in single elements D and S, respectively.

coordinate; that is, a multiple-degree-of-freedom system is required to capture the motion. Frequently, however, only the first few modes are needed to adequately describe the structure's response.

DAMPING VALUE

The response of a structure, system, or component usually has energy dissipation associated with it. These energy losses may be caused by internal energy dissipation within the material or support, or joint

slippage, and they are normally accounted for by means of a damping value.

Design damping values typically range from 0.5 to 10% of critical damping. Damping can have a significant effect on the dynamic response of the structure.

In this study, velocity-dependent damping was assumed, which is a standard modeling procedure in structural analysis.

RESPONSE SPECTRUM

A response spectrum is represented schematically in Fig. 29. A series of single-degree-of-freedom systems (cantilever pendulums) with varying fundamental frequencies are fixed to a moveable base. As the length of the pendulums increases, the fundamental frequency decreases. If the base is moved with a time-varying motion and the motion of each pendulum is recorded, a curve can be developed that plots the maximum response against the frequency of each pendulum. This curve is called the response spectrum for that particular input motion, and is also shown in Fig. 29. The response motion can be displacement, velocity, or acceleration because all are related to frequency. It is possible to generate spectra for both vertical and horizontal base motions.

FREE-FIELD GROUND-RESPONSE SPECTRUM

If the series of single-degree-of-freedom systems (pendulums) are assumed supported on the ground and are subjected to free-field ground motion resulting from an earthquake, then the response spectrum curve developed is called a ground-motion spectrum.

Fig. 29. Simulated cantilevered beams of varying response frequencies. (a) Pendulums of varying response frequency. (b) Plot of maximum response quantity for each single-degree-of-freedom system for particular input motion.

The free-field ground motion is the surface motion induced by an earthquake that has not been altered by the presence of a structure.

Figure 30a shows an example of the acceleration ground-motion spectrum for the 1940 El Centro earthquake. Figure 30b shows the measured horizontal accelerogram record used as input to develop this spectrum.

Figure 31 shows the free-field ground-motion spectra currently used to define seismic input for nuclear power reactors.¹⁵ These smooth

spectra are based on a statistical treatment of approximately 47 earthquake records. They are plotted on tripartite log paper for the convenience of having maximum acceleration, velocity, and displacement values as a function of frequency on one plot. They are normalized to a maximum horizontal ground acceleration of 1.0g. Once the intensity of ground motion for a site is specified in terms of maximum horizontal ground acceleration, both horizontal and vertical spectra can be defined simply by scaling.

SRSS

When analyzing a structure by the response-spectrum technique, one obtains the maximum response for each mode considered. Since these individual modal maxima do not occur at the same time, a statistical procedure is used to combine these maxima. The procedure used is the square root of the sum-of-the-squares method (SRSS). For this method, the following procedure is used:

V_i = value for mode i ,

$$V_{SRSS} = \sqrt{V_1^2 + V_2^2 + \dots + V_N^2}$$

$$= \sqrt{\sum_{i=1}^N V_i^2}$$

Fig. 30. 1940 El Centro earthquake.

Fig. 31. Horizontal design response spectra, scaled to 1-g horizontal ground acceleration (from Regulatory Guide 1.60). Example: for 5% damping and $f = 0.25$ cps, $D = 73.8$ in., $v = 115.93$ in./s, and $a = 0.47g$.

Acknowledgment

The authors wish to acknowledge the assistance of Robert Bernero, Winston Burkhardt, A. Thomas Clark, and Ronald Clary of the U.S. Nuclear Regulatory Commission, Office of Nuclear Material Safety and Safeguards, Division of Fuel Cycle and Material Safety, for providing guidance and support throughout this study.

We also appreciate the time spent by Nuclear Fuel Services, Inc., and the Dravo Corporation. Their assistance and cooperation have been most helpful.

Finally, we wish to thank Frank Tokarz for his advice and support throughout this project, and Shirley Calvert for typing the manuscripts and preparing the figures.

References

1. *Safety Analysis Report, NRS' Reprocessing Plant, West Valley, New York*, Docket Number 50-201 (Nuclear Fuel Services, Inc., Rockville, MD, 1973).
2. NRC, LLL, LASI Meeting, Washington, D.C., private communication, August 24, 1976.
3. D. F. Arthur, R. G. Dong, R. C. Murray, T. A. Nelson, P. D. Smith and L. H. Wight, *Evaluation of Methods for Seismic Analysis of Nuclear Fuel Reprocessing Plants and Fabrication Facilities*, Lawrence Livermore Laboratory, Livermore, CA. Rept. UCRL-52159 (to be published).
4. F. J. Tokarz, R. C. Murray, D. F. Arthur, W. W. Feng, L. H. Wight, M. Zaslawsky, *Evaluation of Methods for Seismic Analysis of Nuclear Fuel Reprocessing Plants, Part 1*, Lawrence Livermore Laboratory, Livermore, CA Rept. UCRL-51802, Pt. 1 (February, 1975).
5. Naval Facilities Engineering Command, *Soil Mechanics, Foundations, and Earth Structures*, Department of the Navy, NAVFAC DM-7 (March, 1971-through Change 2-March, 1974).
6. W. V. Brewer, Lawrence Livermore Laboratory, Livermore, CA, Private communication (July, 1976). Included also in UCID-17453.
7. G. R. Johnson, P. Christiano, and H. I. Epstein, "Stiffness Coefficients for Embedded Footings," *Journal of the Geotechnical Engineering Division*, ASCE, Vol. 101, No. GT8 Proc. Paper 11522 (August, 1975), and *Electric Power and the Civil Engineer*, ASCE Power Division Specialty Conference (August 1974) pp. 469-498.
8. R. A. Parmelee, R. J. Kudder, "Seismic Soil-Structure Interaction of Embedded Buildings," *Fifth World Conference on Earthquake Engineering*, Preprint No. 241a (Rome, Italy, 1973).
9. R. A. Parmelee, and J. H. Wronkiewicz, "Dynamic Coefficients for Evaluating the Seismic Response of Soil-Structure Interaction System," *Fifth World Conference on Earthquake Engineering*, Preprint No. 330 (Rome, Italy, 1973).
10. *1973 Uniform Building Code* (International Conference of Building Officials Whittier, CA, 1973).
11. U.S. Nuclear Regulatory Commission, *Regulatory Guide 1.92, Combination of Modes and Spatial Components in Seismic Response Analysis* (U.S. Government Printing Office, Washington, D.C., February 1976).

12. K. J. Bathe, E. L. Wilson, and F. E. Peterson, *SAP IV - A Structural Analysis Program for Static and Dynamic Response of Linear Systems*, Earthquake Engineering Research Center, University of California, Berkeley, CA, Rept. EERC 73-11 (1973).
13. B. E. Brown, *Structural Analysis Movie Postprocessor (SAMPP)*, Lawrence Livermore Laboratory, Livermore, CA Rept. UCID-30097, Rev. 1 (October 1975).
14. W. H. Collins, and D. F. Arthur, Private communications, Lawrence Livermore Laboratory, Livermore, CA (1976).
15. U.S. Nuclear Regulatory Commission, *Regulatory Guide 1.60, Design Response Spectra for Seismic Design of Nuclear Power Plants* (U.S. Government Printing Office, Washington, D.C., December 1973).
16. U.S. Nuclear Regulatory Commission, *Regulatory Guide 1.61, Damping Values for Seismic Design of Nuclear Power Plants*, (U.S. Government Printing Office, Washington, D.C., October 1973).
17. E. Endebrock, Private communications, Los Alamos Scientific Laboratory (September 20, 1976).
18. *Structural Analysis and Design of Nuclear Power Plant Facilities (Draft)*, Committee on Nuclear Structures and Materials of the Structural Division, ASCE (1976).
19. A. E. Cardenas, et al., "Design Provisions for Shear Walls," *Proceedings, American Concrete Institute*, Vol. 70 (March 1973).
20. ACI Committee 318, *Building Code Requirements for Reinforced Concrete ACI 318-71* (American Concrete Institute, Detroit, MI, 1971).
21. Seismology Committee, *Recommended Lateral Force Requirements and Commentary*, (Structural Engineers Association of California, San Francisco, CA, 1973).
22. Joint ASCE-ACI Task Committee 426, "The shear strength of Reinforced Concrete Members," *Journal of the Structural Division*, ASCE, Vol. 99, No. ST6, Proc. Paper 9791 (June 1973).
23. J. E. Amrhein, *Reinforced Masonry Engineering Handbook* (Masonry Institute of America, Los Angeles, CA, 1973), 2nd ed.
24. Masonry Institute of America, Los Angeles, CA, and Western States Clay Products Association, San Francisco, CA, private communications (1975).