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**Summary of the Seismic Analyses of the
Nuclear Fuel Services Reprocessing Plant at
West Valley, New York**

Issued: March 1978

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of the University of California
LOS ALAMOS, NEW MEXICO 87545

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Elton G. Endebroock

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CONTENTS

ABSTRACT	1
EXECUTIVE SUMMARY	1
I. INTRODUCTION	2
A. Historical Summary	2
B. Seismic Analyses Summaries	2
1. LASL's Analysis	2
2. LLL's Analysis	3
3. CPD's Analysis	4
II. DISCUSSION OF RESULTS	4
A. Introduction	4
B. Topical Discussions	4
1. Seismic Motion Characteristics	4
2. Soil-Structure Interaction	5
3. Modeling of the Facility	5
4. Modeling of Nonmonolithic Connections	6
5. Natural Frequencies	6
6. Combination of Modal and Spatial Responses	7
7. Determination of Stresses or Stress Resultants	7
8. Pile Distress	7
9. Parameter Variation Studies	8
10. Fiberboard Insulation About the General Purpose Cell	8
11. Mechanical Crane Room Flexure Distress	8
12. Process Mechanical Cell and General Purpose Cell	8
13. Facility Movement	9
14. Cell Status Summary	9
a. Acid and Off-Gas Recovery Cell	9
b. Chemical Process Cell	9
c. Analytical Cells	9
d. Equipment Decontamination Room	9
e. Extraction and Product Purification Cells	9
f. General Purpose Cell	9
g. Liquid Waste Cell	9
h. Mechanical Crane Room	10
i. Process Mechanical Cell	10
j. Scrap Recovery Cell	10
k. Uranium Product Cell	10
l. Ventilation Wash Room	10
III. COMMENTS	10
IV. MODIFICATIONS	10

SUMMARY OF SEISMIC ANALYSES OF THE NUCLEAR FUEL
SERVICES REPROCESSING PLANT AT WEST VALLEY, NEW YORK

by

Elton G. Endebrock

ABSTRACT

Results are presented from the seismic investigations of the Nuclear Fuel Services Fuel Reprocessing Plant conducted by the Chemical Plants Division of Dravo Corporation (CPD), the Los Alamos Scientific Laboratory (LASL), and the Lawrence Livermore Laboratory (LLL). Results of the different analytical procedures are summarized.

The LASL studies showed that structural distress would initially occur in two places, the building piles and the walls of the Mechanical Crane Room. This structural distress would occur at 0.14g's. The LLL investigation showed that the Liquid Waste Cell and the General Purpose Cell would start to show structural distress at 0.09g's, and that lateral pile distress would begin at 0.11g's.

EXECUTIVE SUMMARY

A seismic analysis of the Nuclear Fuel Services (NFS) Fuel Reprocessing Plant was performed by the Chemical Plants Division (CPD) of Dravo Corporation, Pittsburgh, Pennsylvania; the Los Alamos Scientific Laboratory (LASL), Los Alamos, New Mexico; and the Lawrence Livermore Laboratory (LLL), Livermore, California. This report presents the analytical procedures used by these agencies and the principal results obtained. Also included are discussions delineating the qualitative relations between the reported results and the analytical methods used. These discussions are based mostly on the seismic investigations of LASL and LLL, with CPD's results and analytical procedures included where appropriate. To CPD it was a design problem, whereas to LASL and LLL it was a problem in analysis. The reports of the LASL, LLL and CPD studies are listed as Refs. 1-4.

The earthquake g-values listed in this report do not imply failure or loss of function of the structural member under consideration but can be considered as representing the point of

distress. The g-values correspond to the point at which seismically induced loads or stresses equal the corresponding load or stress limit for the structural member under consideration. Seismic loads and stresses were obtained using an elastic analysis. The loads and stresses representing the point of structural distress of reinforced concrete members were computed using the American Concrete Institute Building Code as a guide. Specially developed methods were used to obtain the structural distress point in the case of piles. Although the word "failure" is used in Refs. 1-3, the g-values given are identical with the point of structural distress as described above. The loads and stresses computed for seismic loadings were based on a linear analysis; hence, the progression of structural distress could not be determined.

The three agencies made seismic evaluations of the facility confinement structure as it was described in the most recent NFS license application.⁵

The primary structural distress modes that were identified by the three agencies were pile distress, shear wall distress, and flexure or shear distress of a cell section.

A summary of the earthquake levels and the general locations at which the impending structural distress was predicted are given in Table I. Earthquake levels above 0.20g's were not considered.

We concluded that the differences in the acceleration levels at which the NFS confinement structure would remain structurally competent are due to the different models, analysis techniques, and failure criteria used by the three agencies. A large percentage of the differences can be explained whenever the conservatism inherent in the various analytical procedures is considered.

I. INTRODUCTION

A. Historical Summary

Nuclear Fuel Services, Inc. submitted a construction and operating license application to the Nuclear Regulatory Commission (NRC) for a modification and expansion of their reprocessing plant at West Valley, New York. The original license issued in 1966 did not require consideration of acceleration levels (other than

the Uniform Building Code) in the design for seismic events; however, the present NRC seismic requirements became a consideration for the facility modification. NFS retained the CPD as a consultant to determine the seismic competence of the existing facility and to recommend methods for updating the facility to satisfy updated NRC seismic requirements.

To aid the NRC in the license review, LASL and LLL were asked to provide seismic analyses of the existing facility. The principal objective was to identify the level of ground motion at which particular structural elements could first experience mechanical distress. LLL and LASL were to conduct independent analyses but were allowed to exchange information.

LASL and LLL personnel visited CPD for a briefing on the facility structure. Also the NFS West Valley reprocessing plant and site were inspected. Specific information required by LLL and LASL was supplied by NFS and its consultants through drawings, letters, and private communications.

B. Seismic Analyses Summaries

1. LASL's Analysis. The LASL seismic evaluation was separated into the distinct tasks of evaluating soil-structure interaction,

TABLE I
SUMMARY OF RESULTS

Substructure or Structural Member	Earthquake Levels at Impending Structural Distress and Locations					
	A		B			
	Chemical Plants Division of Dravo		Lawrence Livermore Laboratory		Los Alamos Scientific Laboratory	
	Acceleration (g's)	Location	Acceleration (g's)	Location	Acceleration (g's)	Location
Shear Wall (Reinforced Concrete)	0.04*	CPC**	0.09 0.15 0.17	GPC CPC LWC	>0.20	N.R.
Shear Wall (Concrete Block)	N.R.	-	0.03 0.07	TOP AOC	N.C.	
Pile Distress (Combined Loading)	0.08	N.R.	0.11	***	0.14	N.R.
Pile Distress (Tension)	0.05	EPC	0.17	***	0.16	EPC
MCR Section (Flexure or Shear)	N.C.		>0.20		0.14	MCR
Intersection of CPC & PNC	N.R.		>0.20		>0.20	
Joint Pounding	N.C.		N.C.		>0.20	EPC, CPC

N.R. - Not Reported
N.C. - Not Considered

EPC - Extraction & Product Purification Cells
CPC - Chemical Process Cell
GPC - General Purpose Cell
PNC - Process Mechanical Cell
MCR - Mechanical Crane Room
AOC - Acid and Off-Gas Recovery Cell
LWC - Liquid Waste Cell
TOP - Concrete Block Structures on top of CPC

*Using different assumptions, a 0.12g competence level was once reported.
**Based on CPD's interpretation of steel splice integrity in the CPC south wall.
***Perimeter of building, the level at which 10% of piles would be in tension.

determining overall motion of the confinement structure, and analyzing substructures.

Two-dimensional finite element models of the soil-structure system were developed for use with the code LUSH (described in Ref. 3). Input to LUSH was based on the NRC Design Response Spectra. Foundation-level response spectra were developed from LUSH output data for use in obtaining the seismic response of the confinement structure.

A fixed-base finite element model of the confinement structure was developed using plate elements to represent the walls and three-dimensional solid elements to represent the floors and roofs. The weights of heavy equipment and adjacent structures were represented by appropriately located lumped masses. Connections between adjacent cells were modeled as realistically as possible within the constraints of the analytical tools used. The dynamic responses of the confinement structure were obtained using the linear analysis code SAP IV (described in Ref. 3) and the foundation-level response spectra developed during the soil-structure interaction analyses. Total structural responses were obtained by first combining the modal dynamic responses by the square-root-of-the-sum-of-the-squares method (SRSS) with adjustments for closely spaced modes. The spatial components were then combined by the SRSS method.

Several substructures were analyzed to estimate the significant forces acting within them. These forces, or stress resultants, were compared with established or assumed distress criteria for the specific substructure to estimate the earthquake level that would cause structural distress. The substructure forces were obtained from the SRSS acceleration fields and the substructure masses.

The evaluation indicated that the initial structural distress would occur almost simultaneously in two different structural elements: the building piles and the Mechanical Crane Room (MCR) walls. Distress was computed to occur at 0.14g's.

2. LLL's Analysis. The LLL seismic evaluation was based on a three-dimensional

finite element assemblage of plate and membrane elements. The plate elements were used to represent the floors and roofs; the membrane elements were used to represent the walls. Additional lumped masses were added to appropriate locations to represent adjacent structures and heavy equipment. Rigid connections were used between substructures.

The soil-pile system was modeled as uncoupled horizontal and vertical linear springs. The lateral spring stiffnesses were based on piles embedded in an elastic half-space with the soil modulus assumed to vary linearly with depth. The vertical pile stiffness range was based on end-bearing and friction pile behavior. The spring stiffness range for the embedded structures was determined using bounding estimates from different methods, geometries, and soil properties. The total stiffnesses were divided among the nodes of each embedded structure. The four foundation cases listed below were studied.

CASE 1: Maximum embedded structure stiffness
Minimum pile stiffness

CASE 2: Minimum embedded structure
stiffness
Maximum pile stiffness

CASE 3: Minimum embedded structure stiffness
Minimum pile stiffness

CASE 4: Maximum embedded structure stiffness
Maximum pile stiffness

The structural responses were obtained using the program SAP IV with pre- and post-processors for plotting and computing maximum shear stresses. Dead load stresses were computed from a static analysis of the facility. The NRC Design Response Spectrum for 7% damping was used for horizontal motion input, whereas the vertical input was taken as two-thirds of the horizontal. Total responses were obtained using the SRSS method for combining modal responses and direct addition of the spatial

components. The earthquake levels at structural distress were obtained by comparing the computed stresses from dead and seismic loads to the computed stress limits of each element.

Shear wall, pile tension, and pile distress modes were identified. Reported values are based on the lowest g-value from the four foundation cases studied. Shear wall distress was calculated to begin in the Liquid Waste Cell (LWC) and the General Purpose Cell (GPC) at 0.09g's. Lateral load pile distress began at 0.11g's and pile tension distress began at 0.17g's.

3. CPD's Analysis. CPD represented the facility by using a fixed-base lumped mass and spring model for predicting the structural responses. The mass points were arbitrarily chosen and were not related to the torsion, stiffness, or mass centers. Substructures were divided at floor levels, and the mass and stiffness evaluations were performed independently. The stiffnesses were obtained using finite element assemblages of the substructures. Triangular membrane elements were used to represent the walls, floors, and roofs. They used a code called STRUDL for all the linear static and dynamic analyses.

Structural responses were also obtained using STRUDL. The NRC Vertical and Horizontal Design Response Spectra were applied to the foundation boundary points. The fundamental mode appears to be the only one considered in obtaining the structural response. The earthquake levels at impending distress were obtained by comparing computed stresses to allowable stresses in some cases and by comparing seismically produced stress resultants to computed ultimate stress resultants in other cases.

Using present NRC guidelines, CPD calculated that the NFS facility would withstand an earthquake acceleration of $0.04g's \pm 0.01g's$. The first element calculated to show structural distress was reported to be the south wall of the Chemical Process Cell (CPC). The pile loading and shear wall loading were discussed qualitatively in some detail.

II. DISCUSSION OF RESULTS

A. Introduction

This section contains a more detailed description of the methods used by each agency in its treatment of specific topics. Also included are discussions to account qualitatively for the differences in results obtained by each agency. The discussions are mostly about LLL's and LASL's investigations. Since CPD's approach was from a design viewpoint rather than an analytical viewpoint, its results are not always comparable.

B. Topical Discussions

1. Seismic Motion Characteristics.

The three agencies used the NRC Design Response Spectra as the basis for defining the site seismic motion characteristics. However, different damping values were used, and the application of the response spectra differed. CPD applied both the horizontal and vertical NRC Design Response Spectra modified for 14% damping to its structural model. LLL applied the NRC horizontal response spectrum for 7% damping directly to its structural model. The LLL vertical response spectrum was taken as two-thirds of the horizontal response spectrum.

LASL considered the NRC response spectra as defining the site free-field seismic motion characteristics. Compatible bedrock motions were obtained for the undeveloped site. Then, with the structure and piles on the site, foundation-level response spectra for 7% damping were developed using the code LUSH with the bedrock motions as input. The developed foundation-level response spectra were combined into a single vertical and a single horizontal response spectrum.

LASL's soil-structure investigations resulted in a horizontal response spectrum that enveloped the corresponding NRC response spectrum by about 20% over the frequency range from 1 to 33 Hz. The LASL vertical response spectrum showed a peak acceleration response of 3.0g's at a frequency of 6.2 Hz; however, for the structure frequencies, the maximum vertical acceleration response was approximately 0.78g's.

LASL's horizontal and vertical seismic motion inputs were therefore more severe than LLL's.

2. Soil-Structure Interaction. Both LLL and CPD represented the soil and soil-pile system as linear uncoupled springs. CPD provides an extensive discussion on soil-pile spring stiffness effects, but details for determining the spring stiffnesses and the values obtained are lacking.

LLL used two methods for determining the lateral soil-pile spring stiffnesses along with a bounding range of soil parameters. The smallest and largest stiffnesses obtained were used in the seismic analysis. The range for the vertical pile stiffnesses was based on a bounding range of soil parameters and the behavior of end-bearing and friction piles. The soil spring stiffnesses for the embedded cells were obtained by bounding estimates that considered different geometries, methods, and soil properties.

LASL modeled the soil-pile structure system using finite elements. Special considerations were required to transform from a three-dimensional facility to two two-dimensional plane-strain soil models. The soil, pile, and structural systems were idealized, and the material properties for the two-dimensional models were obtained by considering geometries and material properties of the three-dimensional facility. Attempts were made to maintain a one-to-one correspondence between the parameters important in a dynamic analysis. No special techniques were required to obtain dynamic effects of the soil on the modeled embedded cells. LASL did not consider bounds on the soil properties but used values that were described as the best available for the site.

LLL's method for modeling the soil-pile-structure system allows for the direct determination of rocking, translational, and torsional vibration modes and an estimation of the resulting responses. The response spectra method was used to obtain the structural responses, and modal damping was not modified for the soil-structure related modes since SAP IV

accepts a single damping value only. Structural responses are overestimated if soil-structure-related modal damping is not modified.

The two-dimensional models used by LASL eliminate the soil-structure-related torsional effects. Rocking and translational effects were also excluded directly from the analyses since SAP IV uses only a single response spectrum for a given coordinate direction. Torsion of cells and structural elements relative to the fixed-base structural model is included and is evident from the mode shapes.

3. Modeling of the Facility. CPD did not report many details regarding its facility model, though some information became available at meetings. We know that CPD used a lumped mass and spring model of the facility and that finite element models of substructures were developed to determine the spring stiffnesses. Triangular membrane elements were used to represent the walls, floors, and roofs. The procedures for determining the masses and for determining forces or stresses within the structural elements were not reported.

Both LLL and LASL used SAP IV to determine the structural responses, and both used a finite element model of the confinement structure. However, the types of elements used by the two agencies differed. LLL used plane stress membrane elements to represent the walls, and plate elements to represent the roofs and floors. LASL used three-dimensional solid elements to represent the roofs and floors, and plate elements to represent the walls:

The type of element used will influence the calculated seismic response of the structure. Simulating the individual cell structure walls with membrane elements implicitly presumes that all resistance to motion is developed in the plane of the walls only. This assumption is valid for cells that have aspect ratios near unity; however, several cells in the NFS Facility (the GPC, CPC, and Process Mechanical Cell (PMC)) are relatively long and narrow and have thick walls. LASL found that, for these cells, the forces that

have components acting perpendicular to the plane of the walls produce resistance that can dominate the resistance developed by the forces in the planes of the walls. For the GPC, the out-of-plane resistance could be as high as two-thirds of the total cell resistance in the direction normal to the long dimension of the cell. The use of membrane elements would therefore lead to the prediction of shear wall failure at lower g-levels than would have been obtained using plate elements. In LASL's model, the rotational degrees of freedom of the nodal points were deleted. This prevents rotations at the intersections of the walls with the roofs and floors and leads to a minor overestimate of the structural stiffnesses and a minor underestimate of the shear forces in the shear walls.

LLL used artificially stiff plate elements to eliminate local oscillations of the floors and roofs. This common practice effectively caused the floors and roofs to act as rigid diaphragms. LASL did not find it necessary to stiffen elements to eliminate local oscillations. The floors and roofs of the facility are massive; thus, the stiffening of the roofs and floors probably did not result in any significant differences in responses.

CPD's use of membrane elements also resulted in underestimating the structural stiffnesses and overestimating the shear forces in the shear walls.

4. Modeling of Nonmonolithic Connections. A number of cells in this facility are not connected to each other in the usual structural sense. Instead, they have a joint consisting of a slab that is recessed into a wall, rests on the recess, and is separated from the wall by a layer of material called Rodofam. This type of connection does not develop a stiffness in any direction until the relative motion between the connecting parts becomes large enough to close the gap. Significant tensile forces cannot develop, but compressive forces may develop upon contact between the walls and slabs. Although the force-displacement relationships of these connections exhibit nonlinear behavior, a

linear model of the connection behavior was necessary for analysis. Therefore, LASL modeled these connections using linear truss elements that have defined axial stiffness. The modeled connections did not exactly simulate the behavior of the actual connections since they could develop both compressive and tensile forces. However, they did provide for a degree of flexibility between the connecting parts.

LLL, LASL, and NRC personnel agreed in a joint meeting that LLL would model these connections as being rigid while LASL would model them as being flexible. The relative effects of modeling these connections differently is unknown and cannot be determined from LLL's and LASL's present investigations.

LASL performed some computations in which the connections were considered as both monolithic and nonmonolithic. When they were considered as monolithic, the lowest natural frequency was associated with motions of the Extraction and Product Purification Cells (EPC). When they were considered as being nonmonolithic, the lowest natural frequency was associated with motions of the Analytical Cells (AC), LWC, and the Ventilation Wash Room (VWR). The natural frequency of the EPC was lower than that of the AC.

5. Natural Frequencies. The only natural frequencies reported by CPD were 2.5 and 3.5 Hz. These were reported to be the fundamental frequencies in the horizontal and vertical directions, respectively, and were the only ones considered in obtaining the structural responses. Note that the above-mentioned natural frequencies correspond to the peaks of the NRC horizontal and vertical response spectra, respectively.

LLL obtained the 10 lowest natural frequencies and corresponding mode shapes for its soil-pile-structure system model. The lowest frequency obtained was about 2 Hz and the highest was about 16 Hz; this range includes all foundation cases considered by LLL. These two extreme values did not occur in the same case, but represent the bounds on all cases considered. The lowest three frequencies are

associated with soil-structure interaction and correspond to rocking and torsional motions of the facility. The difference in response caused by including modes with frequencies above 16 Hz was reported to be small.

LASL obtained the lowest 24 natural frequencies and their corresponding mode shapes. The lowest structural frequency was about 12 Hz and the highest was about 33 Hz. Many of the frequencies were closely spaced. This set of frequencies did not include those related to the soil-structure system. Soil-pile-structure natural frequencies ranging from 1 to 7 Hz were apparent from the analyses. However, because a fixed-base structural model was used, frequencies below 12 Hz were not used directly in calculating the structural responses.

6. Combination of Modal and Spatial Responses. CPD used only the fundamental frequencies, and no modal responses were combined. Spatial components were combined by the SRSS method.

LLL combined modal responses using the SRSS method. No adjustments for close frequencies were made. The spatial components were combined using direct addition.

LASL also used the SRSS method for combining modal responses; however, adjustments were made for close frequencies. Spatial components were combined using the SRSS method.

Direct addition of spatial components leads to slightly more severe structural responses than does the SRSS method.

7. Determination of Stresses or Stress Resultants. The method that CPD used to determine stresses or stress resultants is unknown.

In estimating shear wall failures, LLL computed the maximum shear stresses at the centroid of the membrane elements from the normal and shear stresses obtained directly from SAP IV. Distress was then determined using the largest maximum shear stress value at these points from the four foundation cases. In determining the condition of a particular shear wall, the maximum shear stress from dead and seismic loads was compared to its ultimate shear

stress. Ultimate shear stress was based on the concrete properties and the amount of steel reinforcing in each element. Pile loads were obtained by adding dead weight and seismic contributions.

LASL used resultant forces in all cases for estimating distress of structural elements. The use of plate elements yields centroidal forces and moments directly from SAP IV; however, these values were not used exclusively. Stress resultants were also obtained from force fields that were computed using the SRSS acceleration fields and the weight distributions of the substructures or structural elements.

The LLL and LASL models of the facility possessed rather coarse finite element grids. The accuracy of frequencies, displacements, stress resultants, and accelerations are not as strongly dependent upon the finite element grid size as are the stresses.

8. Pile Distress. Pile distress was a primary consideration of all the agencies. They were in complete agreement regarding the possible distress modes associated with piles. Pile tension and the combination of a compressive axial load with a lateral pile cap load were identified as the possible distress modes. There was little disagreement on criteria for pile tension distress, but the criteria used by each agency for determining the combination loading distress differed. The lateral loads and the vertical compressive load are time dependent and do not necessarily attain their maximum values simultaneously. As a result, the definition of pile distress is elusive, and an accurate assessment of the distress definition is possible only for very specialized cases.

CPD considered a pile to have reached its loading limit when the axial compressive load reached 250 kips. This value was based on 90% of the static capacity as obtained from a load test. The allowable maximum lateral load was taken to be 16 kips. This value was arbitrarily selected as 6.67% of the vertical allowable capacity. CPD did not consider lateral and

vertical load combinations for determining loading limits.

LLL determined pile integrity by combining dead load stresses on the piles with bending stresses produced by lateral loads. This combination was determined separately for each lateral load. These combined stresses were then compared with pile yield stress.

LASL's pile distress criteria were based on the SRSS of the ratios of the acting loads to their corresponding computed ultimate loads. Impending distress was considered to have occurred whenever the SRSS of the load ratios became unity. The load ratios used were P_a/P_u , M_{ae}/M_{ue} , and M_{an}/M_{un} , where P refers to axial loads, M refers to bending moments, a refers to acting loads, u refers to ultimate loads, n refers to the N-S direction, and e refers to the E-W direction.

Since little information is available in the literature on the behavior of piles during earthquakes, this report will make no judgment on the relative merits of combined load pile distress criteria.

Although the piles under the facility were not designed to resist tensile forces, the linear structural models used by all the agencies necessarily yielded tensile forces whenever the upward earthquake inertial forces exceeded the dead load on the piles. Pile tension was not considered as failure of the piles themselves, but as an indicator of the point at which the elastic solutions were no longer valid. The consequences of pile tension could not be properly evaluated since the model solution was no longer valid. Uplift introduces nonlinearities and only linear solutions could be accommodated.

Both LLL and CPD used the percentage of piles in tension as a measure of the validity of their linear solution. LASL found that pile tension or uplift began at an earthquake level of 0.16g's and LLL determined pile distress to occur at 0.17g's (level at which 10% of the piles are in tension). For pile tension commencing at this level, overturning of individual

cells was not considered to be a primary distress mode for earthquake levels below 0.20g's.

9. Parameter Variation Studies. None of the agencies performed a complete parameter variation study to determine the limits of their results. When limits were given, their bases were not reported. All of the agencies did conduct limited parameter variation studies.

CPD's model was used with and without soil embedment springs.

Because of the limited information available for determining spring stiffnesses for a soil-pile system, LLL considered different geometries, methods, and soil properties to obtain bounds.

LASL performed a parameter variation study in one case. In the soil-structure investigations, the pile lateral stiffnesses were varied from zero (no piles) to about 140 times the average pile stiffnesses. We determined that there were only small differences in the foundation response spectra in the range of the structure natural frequencies.

10. Fiberboard Insulation About the General Purpose Cell. The fiberboard insulation placed between the GPC walls and the surrounding soil is one-half inch thick. LLL, LASL, and NRC personnel agreed that the state of soundness or deterioration of this insulation would not greatly affect the response of the structure; consequently, it was not further considered.

11. Mechanical Crane Room Flexure Distress. The MCR is monolithically connected to the PMC on one end and supported by two columns at the other end. LASL's structural response solution indicated large accelerations in the MCR, which is the north section of the PMC. The roof of the MCR is removable so the structurally significant cross section of the room is U-shaped. The bending moment and the shear force at structural distress occurred at an earthquake level of 0.14g's.

Neither CPD nor LLL reported this type of structural distress.

12. Process Mechanical Cell and General Purpose Cell. The intersection of the PMC and the GPC appeared to be a weak link from

the facility drawings. It was investigated by both LLL and LASL. Both determined that structural distress did not occur for earthquake levels below 0.20g's.

13. Facility Movement. There was agreement between LASL, LLL, and NRC personnel that movement of the facility downhill during an earthquake would be prevented by the piles and the buried cells.

14. Cell Status Summary. The earthquake levels listed below for each confinement cell were taken as the lower of the values reported by LASL and LLL. Pile distress values reported are for the first pile group under each cell that experienced distress. Unless otherwise noted, the listed g-values below 0.20g's were computed by LLL.

a. Acid and Off-Gas Recovery Cell. Parts of the Acid and Off-Gas Recovery Cell (AOC) are enclosed by concrete block walls whose shear strength is low compared to that of reinforced concrete walls. The onset of concrete block shear wall distress was predicted to occur at an earthquake level of 0.07g's. The part of this cell that depends upon concrete block walls for confinement could fail below 0.20g's. Pile tension distress was predicted to occur at 0.14g's, lateral pile distress was predicted to occur at 0.11g's.

b. Chemical Process Cell. This is a reinforced concrete cell closed with a shielding door. A steel frame and concrete block wall structure on the top of the CPC houses the shielding door when it is in the open position. Shear wall distress in this concrete block could be initiated at 0.03g's. But the shielding door would normally be closed during operation, so the confinement integrity would be maintained. Reinforced concrete shear wall distress was predicted to occur at 0.15g's in a wall section that is located above grade. According to CPD's interpretation of the CPC drawings, the reinforcing steel splice lengths in the south wall are inadequate. We do not know if this error was corrected during construction. If not, the earthquake level for reinforced concrete shear wall distress could be less than reported

above. Pile tension and lateral pile distress were predicted to occur at earthquake levels of 0.18g's and 0.11g's, respectively.

c. Analytical Cells. This structure was found to be seismically competent for earthquake levels below 0.20g's; however, if the LWC and VWR fail below 0.20g's, the analytical cell could be damaged.

d. Equipment Decontamination Room. In the Equipment Decontamination Room (EDR), shear wall distress was not predicted for earthquake levels below 0.20g's. Pile tension and lateral pile distress were found to occur at 0.13 and 0.07g's, respectively.

e. Extraction and Product Purification Cells. Shear wall distress in the EPC is not predicted for earthquake levels below 0.20g's. Pile tension distress was found to occur at 0.14g's. Lateral pile impending distress occurs at 0.13g's. This cell is the tallest confinement structure in the facility, but we concluded that overturning was not a major concern for earthquake levels below 0.20g's (LASL). A nonmonolithic connection exists between the EPC and the CPC. Serious pounding at this joint is not anticipated below 0.20g's (LASL). However, piping connecting the EPC with the other cells could be damaged at lower earthquake levels.

f. General Purpose Cell. This cell is completely buried. Shear wall impending distress occurs in the west wall at 0.09g's. Because this cell is buried, confinement would not be compromised if the wall cracked. The soil surrounding the cell is quite impervious, and radioactive products probably would not migrate to a great extent within the soil mass. Lateral pile distress begins at 0.05g's, and pile tension distress occurs at 0.11g's.

g. Liquid Waste Cell. This cell is partially buried, and the waste is in liquid form. Shear wall distress is predicted to occur at 0.17g's. Since the cell is lined with stainless steel sheeting, a penetrating crack in the concrete walls would not necessarily destroy their confinement integrity. Lateral pile distress occurs at 0.10g's. Pile tension distress

does not occur for earthquake levels below 0.20g's.

h. Mechanical Crane Room. This room is discussed in conjunction with the PMC.

i. Process Mechanical Cell. The MCR forms the north end of the PMC. The MCR was designed for storage and maintenance of the traveling crane that services the PMC. It is separated from the remainder of the PMC by a heavy shielding door that is normally closed during operation of the cell. It was determined by LASL that the MCR could completely separate from the PMC at a section north of the shielding door at 0.14g's (LASL). However, confinement integrity of the hot part of the PMC would be maintained if the shielding door was closed. Shear wall distress in the PMC and direct shear distress between the PMC and the GPC are not predicted for earthquake levels below 0.20g's. Pile tension distress occurs at 0.12g's and lateral pile distress occurs at 0.13g's.

j. Scrap Recovery Cell. The Scrap Recovery Cell (SRC) is monolithically attached to the GPC and could therefore be considered a part of the GPC. From earthquake levels below 0.20 g's, no structural distress was predicted in this cell.

k. Uranium Product Cell. Shear wall distress was not predicted for earthquake levels below 0.20g's in the Uranium Product Cell (UPC). Lateral pile distress occurs in this cell at 0.12g's, and pile tension distress occurs at 0.13g's.

l. Ventilation Wash Room. Structural distress was not predicted for this room for earthquake levels below 0.20g's.

III. COMMENTS

Special conditions were encountered in this evaluation that are not normally features of a structure subjected to a dynamic analysis. The confinement structure included thick-walled asymmetrical cells that were also asymmetrically located relative to each other. Cells were located both above grade and below grade (buried), and interfaces between cells were both monolithic and nonmonolithic. Although the

investigations were state-of-the-art, the complexity of the facility caused differences in the seismic analytical approaches and results. In this report attempts have been made to indicate the specific analysis or computational procedures used that led to different results. LASL attempted to balance the amount of conservatism inherent in the analytical methods so that a large factor of safety did not accumulate. LLL also recognized inherent conservatism in some of the computational methods and removed it wherever possible. CPD included safety factors based on current regulatory guidelines in each step of its analysis, thus leading to a large safety margin in the final result.

The investigations only addressed the onset of structural distress and did not attempt to include progressive structural distress. Note that structural distress is very different from confinement structure failure. Confinement structure failure occurs when radioactive products can be released to the environment; this type of failure is not addressed here. The significance of the structural distress types identified in these investigations is discussed below.

1. Pile Tension Distress. The pile caps were not designed to resist tension, and during an earthquake, cells may lift off the piles. The linear solutions used in the investigations made it necessary to include pile tension forces. Pile tension is not a crucial failure mode in that it does not necessarily result in penetrating cracks in the cells, but it may lead to them. The existence of pile tension was used by LLL as an indicator of the validity of their linear solution since pile tension or uplift results in nonlinear behavior. In discussions NRC, LLL, and LASL decided that elastic analysis would be valid up to the point where 15% of the piles were in tension.

2. Lateral Load Pile Distress. Whenever combined lateral and vertical loads acting at the pile caps reach certain magnitudes, the piles yield. The beginning of pile

yielding also indicates the start of the non-linear regime. Yielding of the piles does not directly result in loss of cell confinement integrity but may lead to the overstressing of cell parts or structural elements. The pile yielding could also aid in reducing the structural response in that more energy is dissipated.

3. Shear Wall Distress. Shear wall distress could result in a penetrating crack with possible loss of confinement integrity. Once a crack forms, load cycling will probably lead to a rapid progression of cracking. The shear walls that were identified to be among the first to reach structural distress are dominated by shear forces rather than flexural forces. These shear walls could not be expected to exhibit ductile behavior. Consequently, in considering confinement integrity of the cells, shear wall failure could lead to loss of confinement.

4. Cell Section Flexure Distress. This type of structural distress was found to be likely only in the MCR. It could lead to a complete collapse of the cell section since the maximum elastic moment is greater than its ultimate moment capacity. The potential for releasing significant amounts of radioactive materials is small since the crucial section is not a part of a hot cell.

IV. MODIFICATIONS

The possibility of modifying the reprocessing plant to increase its seismic

competence was considered briefly. The structural elements that are among the first to become distressed are not always accessible, so modifications would be costly, if not impossible. All agencies agreed that any major modification would be impractical, though in certain cases minor modifications may be feasible. Construction of new cells to serve particular functions would be more practical than attempting to upgrade the existing cells.

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