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Seismic evaluation of commercial plutonium fabrication plants in the United States

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PREFACE

The Office of Nuclear Material Safety and Safeguards of the U.S. Nuclear Regulatory Commission (NRC) is examining the effects of adverse natural phenomena on several commercial plutonium fabrication facilities. The examination resulted from a change in the regulations that establish procedures and criteria for the issuance of licenses to possess and use (and, thereby, fabricate) special nuclear materials. These regulations are contained in Title 10 Part 70 of the Code of Federal Regulations, commonly denoted as 10 CFR 70.

In part, 10 CFR 70 describes the content of license applications and supporting documents and requirements for approval of applications and issuance of licenses. After the change in regulations on Sept. 2, 1971, applications for licenses were required to contain a description and safety assessment of the design bases of the principal structure, systems, and components of the plant, including provisions for protection against natural phenomena. Therefore, facilities for which the license application was filed after Sept. 2, 1971 must be designed to provide protection against natural phenomena.

For existing facilities, protection against the effects of natural phenomena was addressed in the required preamble to the rulemaking change. The Statement of Considerations states that existing licensed plutonium fabrication plants would be examined with the objective of improving, to the extent practicable, their abilities to withstand adverse natural phenomena without loss of capability to protect the public. For its examination, the NRC assembled teams of experts from the fields of seismology/geology, surface hydrology, normal- and severe-weather phenomena, structural analysis, source-term characterization, meteorological dispersion, demography, ecology, and radiological impact. These experts reviewed each facility and realistically assessed the range of credible consequences of natural phenomena and the likelihood that any might occur. The natural phenomena considered were (1) severe weather, including both tornadic and nontornadic winds, (2) earthquake, and (3) flood.

This report is a synopsis of the structural analyses of the facilities to evaluate the effects of earthquakes, which were performed under the direction of R. C. Murray and F. J. Tokarz of the Lawrence Livermore National Laboratory

(LLNL). The report highlights two methodologies: techniques to estimate seismic hazard, and techniques to determine the ultimate capacity of critical structures and equipment. D. L. Bernreuter of LLNL was task director for the seismic hazard characterization, which was done by TERA Corp., Berkeley, California. D. W. Coats of LLNL directed the structural analysis, which was done by Engineering Decision Analysis Company, Inc., (EDAC) Irvine, California.

The authors wish to thank W. Burkhardt and J. Ayer of the NRC for their support and critical review of this project. We also gratefully acknowledge the assistance of C. K. Chou of LLNL, L. H. Wight of TERA Corp. and both D. A. Wesley and R. P. Kennedy of EDAC, Inc.

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ABSTRACT

This report is an overview of Lawrence Livermore National Laboratory's seismic assessment of six commercial plutonium fabrication plants licensed by the U.S. Nuclear Regulatory Commission (NRC) before September 2, 1971. The seismic assessment generally has three parts: (1) documentation of the structural condition of each facility and its critical equipment; (2) characterization of the seismic hazard (i.e., determination of peak ground acceleration vs return period for each site); and (3) evaluation of seismic capacity to determine ground motion levels at which critical structures and equipment fail. The failure evaluation used structural capacities of median-centered strength characteristics of the as-built configurations from (1) and seismic hazard input from (2). Results of the assessment were partial input for an overall natural risks study by the NRC.

INTRODUCTION

The NRC Office of Nuclear Material Safety and Safeguards is evaluating commercial plutonium fabrication facilities to determine the effects on them of three adverse natural phenomena--earthquake, flood, and high wind. The evaluation affects only those facilities that were licensed before Sept. 2, 1971, when regulations were changed (See Preface). Table 1 lists the facility locations. The NRC will use the results of the evaluation in license renewal reviews to determine how much retrofitting, if any, each facility needs for adequate protection of the public against adverse natural phenomena. Reference 1 provides complete background information on the NRC natural risks evaluation.

Table 1. Commercial plutonium fabrication facilities considered in this study.

Facility	Acronym	Location	Construction period
Westinghouse Plutonium Fuels Development Laboratory	PFDL	Cheswick, Penn.	1967-68 1973 (additions)
Babcock and Wilcox Facility	B&W	Leechburg, Penn.	1959-70 (5 stages)
Battelle Memorial Institute Columbus Laboratories West Jefferson Site	BMI	West Jefferson, Ohio	1960, 1964, 1967, 1971
Exxon Mixed Oxide Fuel Plant	MOXP	Richland, Wash.	1971
Atomics International Nuclear Materials Development Facility	NMDF	Santa Susana, Calif.	1966-67
General Electric Vallecitos Nuclear Center ^a	VNC	Pleasanton, Calif.	

^aSeismic hazard characterization only.

The NRC assembled a wide range of experts in a task-force approach to the evaluation (See the Preface). Figure 1 is a flow chart for the seismic portion of the natural risks evaluation. The hydrology and strong-wind evaluations follow similar patterns. The Lawrence Livermore National Laboratory (LLNL) provided direction and expertise for the "Seismic

FIG. 1. A flow chart of the seismic assessment portion of the NRC's overall natural risks evaluation. Shaded ovals were under the direction of LLNL and are the subject of this report.

Characterization" and "Structural Analysis" portions of the seismic evaluation (the shaded ovals in Fig. 1). This report is a synopsis of those two LLNL-directed parts of the seismic evaluation, which were largely done by two contractors.

TERA Corporation of Berkeley, Calif., characterized the seismic hazard for each site.^{2-6,28-29} Note that in keeping with the overall natural risks evaluation and with the trend in earthquake engineering literature, we use the phrase "seismic hazard" in this report to mean the expected occurrence of an earthquake. Seismic risk, on the other hand, defines the expected consequences (i.e., loss of life, property damage) of an earthquake.

Engineering Decision Analysis Company, Incorporated, (EDAC) of Irvine, Calif., performed structural assessments of critical facilities at all but the GE Vallecitos Nuclear Center in two phases or tasks (structural assessment of the GE site was not within the scope of this study). The Task I efforts documented the present condition of each facility, including:

- Review of drawings and specifications related to the structure and critical equipment.
- Identification of the critical locations within each facility.
- Presentation of details of the critical process equipment and structural systems that can carry lateral loads.
- Description of the analytic procedures to be used in the Task II seismic capacity evaluation of each facility.

In addition to providing a data base for structural evaluations by EDAC, the Task I documentation (Refs. 7-11) was intended to provide structural data for the extreme wind load evaluation by other consultants.

The Task II analysis of each building structure and all critical equipment established the peak ground acceleration, A_{max} , at which critical areas or equipment collapsed or otherwise resulted in loss of confinement of hazardous chemicals (Refs. 12-16). For purposes of the overall natural risks study, critical areas are those locations in which hazardous chemicals are processed or stored in a dispersible form that makes release to the outside possible should confinement barriers be breached. Similarly, critical equipment is equipment for processing materials that comprise hazardous chemicals in dispersible form; moreover, the equipment structure serves as a primary confinement barrier. The primary focus of Task II is the building

structure (final confinement barrier), architectural walls or partitions (secondary confinement barrier), and glove box equipment (primary confinement barrier) associated with the critical areas. The mode of release resulting from the earthquake hazard is the loss of primary confinement by (1) direct glove box failure, or (2) indirect glove box damage caused by adjacent equipment and connections, collapsing structural elements, or structure-supported equipment components.

This report is a synopsis of the 17 documents generated during the seismic assessment. Its purpose is to present, in one place, the methods and results of the seismic hazard characterization and the structural analysis. For the seismic hazard characterization, results are in the form of curves of peak ground acceleration $A_{\max}$ vs return period for each site. Results of the structural analysis are those values of $A_{\max}$ at which critical structures or equipment fail.

The probabilistic nature of these results is in keeping with the fact that the LLNL study is only one part of the overall natural risks assessment (Fig. 1). Thus, although we may report short return periods for substantial ground acceleration at certain sites or low capacities for certain structures, the NRC will recommend necessary modifications in the light of results from the overall study.

EXECUTIVE SUMMARY

This report is an overview of LLNL's seismic assessment of six commercial U.S. plutonium fabrication plants licensed by the U.S. NRC before Sept. 2, 1971. The seismic assessment generally had three parts:

- (1) Documentation of the structural condition of each facility and its critical equipment
- (2) Characterization of the seismic hazard (i.e., determination of peak ground acceleration vs return period for each site)
- (3) Evaluation of seismic capacity to determine ground motion levels at which critical structures and equipment fail.

The failure evaluation (3) used structural capacities of median-centered strength characteristics of the as-built configurations from (1) and seismic hazard input from (2). Results of the assessment provide partial input for an overall natural risks study by the NRC. Figure 2 shows the location, estimated peak ground acceleration for a 10^3 yr return period, and site soil conditions for each facility.

This report is a synopsis of the 17 documents generated during the seismic assessment. Its purpose is to present, in one place, the methods and results of the seismic hazard characterization and the structural evaluation. For the seismic hazard characterization, results are in the form of curves of peak ground acceleration $A_{\max}$ vs return period for each site. Results of the structural analysis are those values of $A_{\max}$ at which critical structures or equipment fail. Both types of results are site-specific. They are reported by facility in the Summary of Results section of this report and summarized in Table 2.

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FIG. 2. The location, estimated peak ground acceleration for a 10^3 yr return period, and site soil conditions for each facility.

Table 2. Summary of LLNL's structural capacity evaluations of the five facilities and corresponding seismic hazard characterization.

Facility	Structural capacity, g			Corresponding lower bound return period, yr	Comment
	Low	Median	Upper		
Westinghouse Plutonium Fuels Development Laboratory	0.33	0.49	0.7	30,000	Longitudinal frame collapse
Babcock and Wilcox Facility	0.14	0.21	0.31	2000	FAB 1 and FAB 6 walls collapse
Battelle Memorial Institute Columbus Laboratories West Jefferson Site	0.11g	0.17	0.26	600	Collapse of the main laboratory roof/wall system under E-W shaking
Exxon Mixed Oxide Fuel Plant	1.09	1.37	1.72	>10 ⁵	Loss of diaphragm and truss support for panel walls under N-S shaking
Atomics International Nuclear Materials Development Facility	0.41	0.6	0.9	100	Loss of diaphragm support under E-W shaking
General Electric Vallecitos Nuclear Center	Assessment limited to seismic hazard characterization				

SEISMIC HAZARD ANALYSIS

INTRODUCTION

There are two distinctly different approaches to seismic hazard characterization--deterministic and probabilistic. In the deterministic approach, the analyst

- Decides that an earthquake of a given magnitude or intensity occurs at a certain location.
- Attenuates the ground motion from the earthquake source to the site.
- Determines the effects of that earthquake.

The problem with this approach is that it is difficult to define the margin of safety in the resulting design parameters. As a result, the analyst generally uses upper-bound estimates of ground motion, which are typically overly conservative.

In a probabilistic approach, on the other hand, the analyst quantifies the uncertainty in the number, size, and location of possible future earthquakes and can thus present a trade-off between more costly designs or retrofits and the economic or social impact of a failure.

Although the probabilistic approach requires significantly more effort than does the deterministic approach, we used it to develop seismic hazard characterizations in order to:

- Quantify the hazard in terms of return period.
- Rigorously incorporate the complete historical seismic record.
- Incorporate the judgment and experience of seismic experts.
- Account for incomplete knowledge about the locations of faults.
- Assess the hazard at the site in terms of spectral acceleration, velocity, displacement, or earthquake intensity.

The method is particularly appropriate for eastern facilities where the seismicity is very diffuse and cannot be correlated with surface faulting as it can be in the western United States. The location of the design earthquakes in the eastern United States is therefore particularly uncertain. The strength of the probabilistic approach is its ability to quantify these

uncertainties. Its major weakness is the lack of plentiful statistical data from which to characterize the various input parameters in probabilistic terms. Nevertheless, the credibility of the probabilistic approach has been established through detailed technical review of its application to several important projects and areas. Recent applications include assessments of the seismic hazard in Boston (Ref. 17), in the San Francisco Bay Area (Ref. 18), in the Puget Sound Area (Ref. 19) and the continental United States (Ref. 20). Results of these studies have been applied to, among other areas:

- Development of long-range earthquake engineering research goals.
- Planning decisions for urban development.
- Environmental hazards associated with the milling of uranium.
- Design considerations for radioactive waste repositories.

This diversity of application demonstrates the inherent flexibility of the probabilistic hazard assessment approach.

INPUT

We used the probabilistic approach to characterize the seismic hazard for each site in this study. The input to a probabilistic hazard assessment comprises earthquake frequency relations, attenuation functions, and a specification of local source regions. Because hazard assessment calculations are very sensitive to the particular composition of the input, we consulted with experts in local and regional seismology during the preparation of input for each facility. These experts are acknowledged in the Preface.

METHODOLOGY

The product of a probabilistic approach is a measure of the seismic hazard expressed in terms of return period, or reciprocal annual probability. The methodology to determine seismic hazard at a site is usually divided into the following steps, which are discussed in greater detail below:

- Specify the geometry of local seismic regions.

Based on the geology and historic seismicity of the region, sources are identified as line sources (faults) or area sources (Fig. 3a).

The largest earthquake (in terms of magnitude or intensity) associated with each source is established.

- Describe past seismicity in terms of earthquake occurrence.

The recurrence of earthquakes of various magnitudes is based primarily on historical seismicity. A straight line or a set of straight lines is fitted on the data using regression analysis to develop these relationships (Fig. 3b).

- Develop an earthquake recurrence model.

The model assumes that the earthquake occurrence follows a Poisson distribution in time. This is a standard assumption.

- Choose a transfer function to mathematically transmit information from the epicenter to the site under consideration in terms of parameters that a structural engineer can use (Fig. 3c).

Using a combination of theory and data, derive the peak accelerations at a given site that result from earthquakes of various sizes at different source locations. As discussed below, different approaches are needed for sites in the western vs the eastern United States. Then, combining the potential activity of all sources, the probability that a certain acceleration will not be exceeded within a given time period t is determined (Fig. 3d). This is the seismic hazard model.

Source Regions

A basic assumption in any seismic hazard analysis is that seismic source zones can be defined within which the future seismicity is homogeneous and can be represented best by the historical seismicity.

The seismic zone about each site can significantly influence the site hazard; therefore, particular care was taken in the zone definitions for this project. The zones prepared by Algermissen and Perkins²⁰ were a starting point in the definition of source regions. Their zones were generally considered inadequate for the site-specific results sought by this program because their objective was regional seismic hazard assessment. We critically examined the Algermissen and Perkins zones from the perspective of our seismic data base as well as from various geologic, geophysical, and tectonic data. Their zones were modified as needed to capture the site-specific assessment required for the structural analyses.

FIG. 3. For a probabilistic hazard assessment, an analyst generally (a) specifies the geometry of important seismic regions, (b) characterizes the relative frequency of earthquakes of various sizes, develops an earthquake recurrence model (usually a Poisson distribution in time, not shown), and (c) selects a transfer function that transforms information about the earthquake at the epicenter into information at the site, such as ground acceleration, that a structural engineer can use. The result of such an assessment is a plot of return period vs peak horizontal ground acceleration (d).

Source Region Seismicity

We updated the data base to 1977 and found that for many source regions there was little change in the earthquake statistics from the 1974 data used by Algermissen and Perkins to calculate the rates at which earthquakes occur in each source region. We carefully compared the seismicity model calculated with our data against the model prepared by Algermissen and Perkins and reconciled any differences. As was the case with the development of seismic source regions, we used the Algermissen and Perkins results as a starting point in the development of the necessary seismicity models. We often researched data points that were crucial to the statistics, and we always compared our model with all appropriate previous models. In one case for which the historical seismic data were insufficient to credibly define a model, we complemented the data with certain geologic information in order to model several specific faults.

In all cases consistent with conventional practice, the resulting seismicity model was characterized by

$$\log N_c = a - bM \quad (1)$$

where

- M = measure of the earthquake severity
(magnitude or intensity)
- N_c = cumulative number of earthquakes greater
than M
- a, b = parameters of a straight line.

One critical component of the seismicity model is the upper limit to the above relation, known as the maximum-magnitude earthquake (Fig. 3b). The maximum-magnitude earthquake is a very uncertain parameter, particularly in the less seismic areas of the eastern United States. Algermissen and Perkins selected maximum magnitudes corresponding to the largest historically observed earthquakes. We characterized the uncertainty in this choice by assuming that each region could have earthquakes of roughly one-half magnitude unit greater than the largest magnitude earthquake in the historical record. When required, the well-established Gutenberg-Richter relationship was used to

relate historical epicentral intensity I_e to magnitude:

$$M = 1.3 + 0.6 I_e. \quad (2)$$

To account for uncertainty in not only this relationship, but also in the physical arguments leading to a specific maximum-magnitude earthquake, we allowed for upper cutoffs of plus and minus one-half magnitude unit about the maximum earthquake. For calculational simplicity, we also specified a lower cutoff magnitude for each region. Sensitivity studies showed that earthquakes with magnitudes less than the lower cutoff do not affect the hazard at the site.

Attenuation Relationships

Once the seismic activity has been determined within a source region, attention focuses on the effect at the site from such activity. Attenuation relationships are transfer functions that carry the information from the source to the site in terms structural engineers can use (i.e., acceleration, velocity, spectral acceleration). These attenuation relationships are inexact because of the lack of understanding in earthquake-generation phenomena, variations in travel paths, variable site conditions, and the limited descriptive capability of the parameters used. Probabilistic models consider the whole spectrum of uncertainties associated with these relationships for any event at any location. Thus, all possible outcomes at the site are covered, from the most favorable to the most adverse, each expressed in terms of how likely it is to occur.

To develop attenuation relationships, we need strong ground motion data. Such data are widely available in the West, but practically nonexistent in the East. For eastern facilities, we developed the functional form of the relationship by synthesizing the results of several previous investigations. The specific relationship is then defined by a fit of the resulting functional form to the only available strong motion data in the central and eastern United States.

The functional form is developed from three separate regression analyses that resulted in best fits to the data for

- Site intensity I vs distance Δ and epicentral intensity I_e .

- Peak acceleration $A_{\max}$ vs site intensity, earthquake magnitude M , and distance.
- Earthquake magnitude vs epicentral intensity.

The first of these relationships is based on a study by G. A. Bollinger of the Charleston, South Carolina, 1886 earthquake (Ref. 21). Bollinger's analysis of the 800 intensity observations from that earthquake resulted in the development of a new intensity attenuation relation that is similar to other published relations but has the added credibility of being based on the most complete set of East Coast data. Bollinger's use of the actual intensity observations, rather than the isoseismals, permits the specification of percentile variations to the fit. Bollinger's 50 percentile relationship is

$$I = I_e + 2.87 - 0.0052 \Delta - 2.88 \log \Delta \quad (3)$$

where

- I = site intensity (MM)
- I_e = epicentral intensity (MM)
- Δ = epicentral distance (km).

The acceleration relation that couples with Bollinger's relationship was derived from analysis of nearly 1500 world-wide accelerograms (Ref. 22). Extensive statistical analysis resulted in the following correlation:

$$\log A_{\max} = 0.14 I + 0.24 M - 0.68 \log \Delta + \gamma \quad (4)$$

where

- $A_{\max}$ = peak horizontal ground acceleration
- M = earthquake magnitude
- γ = region-specific parameter.

We used the Gutenberg-Richter relationship, Eq. (2), to relate earthquake magnitude to epicentral intensity. Combination of Eqs. (2), (3), and (4) gives

$$\log A_{\max} = 0.47 M + 0.0905 - 1.08 \log \Delta - 0.0007 \Delta + \gamma \quad (5)$$

The effect of the Δ term is small--(less than 0.01 g reduction at 500 km)--and, for convenience, it is neglected in the calculations. The value of γ was determined by comparison of the expression with the available

eastern United States strong-motion data (Ref. 23, 24). A value of 0.9 provided the best fit to the data and is consistent with other studies.

Attenuation for the western United States can be obtained much more directly from the abundant data. It is important to consider the magnitude of the data dispersion about a median attenuation relationship (Fig. 3c). The component relationships that were synthesized into an attenuation relationship were themselves best fits to data with associated dispersion. Because the data sets used in these individual analyses are diverse and not readily available, we choose to assess the dispersion for our attenuation relationship by considering other data sets and other attenuation analyses.

Hazard Calculations

We used the total probability theorem to calculate the probability that peak ground acceleration A will occur. This is calculated by multiplying the conditional probability of A , given earthquake magnitude m and epicentral distance r , times the probabilities of m and r , and integrating over all possible values of m and r :

$$P[A] = \iint P[A/m \text{ and } r] f_M(m) f_R(r) dm dr.$$

In our analysis, $A_{\max}$ is related by the attenuation relationship, Eq. (5), to epicentral distance and earthquake magnitude. The distribution on earthquake magnitude, $f_M(m)$, was readily derived from frequency relationships of the form of Eq. (1). The distribution on distance, $f_R(r)$, depends on the geometry of the source region. For simple geometries, the distributions can often be integrated analytically. Realistic geometries, however, require numerical evaluation of the integral.

We used versatile computer programs that incorporates the theory presented above with a numerical integration scheme to evaluate complex source-site geometries. For circular sectors within each source region at the site, the computer code calculates the expected number of earthquakes causing accelerations greater than a specified acceleration. This is done for each source region and the host region. The expected numbers are summed for each region, and the resulting hazard calculated from

$$\text{hazard} = 1.0 - \exp(- \text{total expected number}).$$

This expression follows from the conventional assumption that earthquakes have a Poisson distribution in time.

The return period associated with the specified acceleration is then the reciprocal of the hazard. It follows from the definition of return period that accelerations with a particular return period have a 63 percent probability of being exceeded within the return period.

Our estimate of the seismic hazard represents the weighted results from individual calculations for a base case and perturbations of input parameters about this base. The perturbations are weighted by subjective estimates of their probability of occurrence. The parameters that are considered uncertain and are included in the estimate of the hazard are the maximum earthquake in each source zone and the intercept of the attenuation relation expressed through the value of γ .

We examined the sensitivity of the results to possible uncertainty in other elements of the input and established that the return period accelerations are relatively insensitive to reasonable variations in

- Slope of the attenuation function
- Boundaries of the source zones
- Historical earthquake statistical parameters within each zone.

RESULTS

Results of our probabilistic seismic characterization are presented as plots of return period (time between earthquakes that result in a given peak horizontal ground acceleration) vs peak acceleration. These peaks are presented in the "Summary of Results" section of this report.

RESPONSE SPECTRA

Our results define the peak horizontal accelerations at the facilities for various return periods. We also determined appropriate response spectra for the sites because some structures and equipment have fundamental frequencies in the range of spectral amplification of the ground motion. The response spectrum for a site clearly cannot be developed in association with a specified earthquake; our return period accelerations represent an integrated effect at the site from a wide variety of earthquakes, and the response

spectrum must reflect this. Accordingly, we judged that the shape of the spectrum should be similar to the Newmark statistically-based spectra from which Regulatory Guide 1.60 evolved (Ref. 25). This intuitive judgment is supported by a separate analysis we performed in which the hazard is assessed through peak velocity. This calculation shows that the ratio of peak velocity to peak acceleration is very close to the ratio developed in WASH 1255 for alluvial sites.

Except for the AI site at Santa Susana, Calif., all the facilities investigated are founded on soil. The median response spectra for rock presented in WASH 1255 (Ref. 26) were used for the AI facility characterization, and the median response spectra for alluvium from WASH 1255 were used for the characterization of the other facilities. Response spectral accelerations were determined by scaling the median response spectra in WASH 1255 to the desired peak accelerations (Fig. 4).

FIG. 4. Median response spectra used in the analyses for alluvium (a) and rock (b) scaled to $1g A_{\max}$ for 0.5%, 2%, 5% and 10% of critical damping, from Ref. 26. Note that the rock spectra have been modified for the approximate inelastic analysis (See Fig. 5).

STRUCTURAL ANALYSIS

The structural evaluation of the plutonium fabrication facilities involved two tasks. First, we documented the condition of each facility through

- Review of existing drawings and reports
- Inspections of each site and facility
- Requests for additional information.

These Task I reports⁷⁻¹¹ identified critical locations in each facility, described in detail the process equipment and lateral-load-resisting structural systems in these locations, and proposed analytic techniques for Task II, the actual structural evaluation and presentation of results. The key findings from Task II for each facility are presented in the Summary of Results section.

ELASTIC BEHAVIOR

We used the response-spectrum method to analyze the elastic dynamic behavior of structural systems. In this procedure, each structural system is modeled by assuming that its mass, stiffness, and damping properties are concentrated into discrete physical elements. The natural frequencies and mode shapes of the structural system are found by an eigenvalue/eigenvector analysis of the model. Usually in earthquake analyses, linear combinations of several mode shapes with the lowest frequencies are sufficient to represent all different deformations of the structure.

Seismic loading is usually represented by a response spectrum, which shows how an idealized single-degree-of-freedom model would respond to a given earthquake. In practice, many strong earthquakes are combined into a single response spectrum as is done in R.G. 1.60. This representative seismic loading is then applied to the model, and the peak responses of each of the system's modes is calculated. Because these peak responses are usually out of phase, they must be combined by a method that takes this into account, for example, the square-root-of-the-sum-of-the-squares (SRSS) method.

Dead weight axial forces were included in the analyses in terms of their effect on the moment capacity of main columns. Concurrent vertical

accelerations of the magnitude considered possible at each facility generally did not appreciably affect the distribution of dead weight stresses. As indicated in Ref. 27, peak values of the vertical component of earthquake ground motions are on the order of one-half to two-thirds of the peak value of the largest horizontal component. In addition, the horizontal and vertical values do not occur concurrently. Reference 27 suggests that a maximum of 40% of the peak vertical amplitude be used when combining the concurrent effects of vertical acceleration and horizontal acceleration. Thus, concurrent vertical accelerations are less than 27% of peak horizontal acceleration levels.

Soil-structure interaction is the modification of free-field ground motion due to feedback effects resulting from the presence of the structure and its foundation. For these relatively lightweight structures and generally stiff foundation materials, soil-structure interaction effects were considered negligible and were not included in the analyses.

INELASTIC BEHAVIOR

To determine the peak ground accelerations that characterize failure or collapse, behavior in the inelastic range must be considered. The nonlinear response of shear wall systems is generally small compared with other structural systems because of fewer energy-absorption and ductility mechanisms. Sources of nonlinear response before such systems collapse come from cracking of concrete, yielding of steel, and working or tearing of connections. Where significant cracking of concrete and yielding of steel are involved prior to collapse, energy absorption is enhanced.

Inelastic behavior relates to the capacity of the structure to absorb energy after critical members have yielded. The energy-absorption capacity of a structure is determined from the force-deflection curve by noting where permanent inelastic deformation in one load cycle causes a member to be unable to carry a load in a subsequent cycle. Such degradation of strength results in the loss of energy absorption capacity during subsequent cycles. The area under the inelastic force-deflection curve is then considered to be the total energy that can be absorbed and is often called the reserve energy. In the reserve-energy method, nonlinear response is included by analyzing an equivalent elastic system. Equivalency is maintained by equating the reserve energy of the inelastic system to the strain energy of the equivalent elastic system. The technique is explained more fully and a sample calculation is given in Ref. 13.

In some cases, we used the ductility modified response spectrum method of analysis as an approximate way to assess nonlinear response and capacity of structural systems. The ductility method is an adaptation of the response-spectrum method of linear dynamic analysis in which an inelastic response spectrum is developed to reflect the reduction in response for systems above the yield point. The amount of reduction depends on the ductility of the system as well as the region of the response spectrum as shown in Fig. 5.

FIG. 5. The ductility modified response spectrum method of analysis is an adaptation of the response-spectrum method of linear dynamic analysis in which an inelastic response spectrum is developed to reflect the reduction in response for systems above the yield point. The reduction factor R depends on the ductility of the system μ (the maximum response deflection of a single-degree-of-freedom structure to its yield point deflection) as well as the region of the response spectrum.

UNCERTAINTY BOUNDS

The seismic capacity evaluation results reported here are part of an overall natural risk analysis (see Fig. 1). To provide compatibility with this overall analysis, results are required in terms of estimated median capacities and estimated one-standard-deviation (one-sigma) upper- and lower-bound capacities. We obtained median capacity results for structures and equipment by using the procedures described above with median values of parameters. A probabilistic approach based upon the general statistical properties of a log-normal distribution was used to obtain the one-sigma upper and lower bound variation of each random variable or parameter that affects the results, including material properties, evaluation procedures, and seismic-input definition. The expected variation in such important parameters as yield strength, damping, and ductility, were developed during the Task I effort and are presented in the Task I reports.⁷⁻¹¹ The parameters were considered to be log-normally distributed for purposes of the approximate uncertainty bound analyses performed under the Task II evaluations.

SUMMARY OF RESULTS

WESTINGHOUSE PLUTONIUM FUELS DEVELOPMENT LABORATORY (PFDL)

Geology and Seismology

The Westinghouse facility is situated in the township of Harmar, Penn., approximately 10 mi northeast of Pittsburgh. It is built atop a large remnant of a broad alluvial terrace within the valley of the Allegheny River. The elevation of this terrace ranges between 960 and 980 ft, approximately 220 ft above the river's flood plain. Unconsolidated alluvial sands, gravels, and occasional layered clays underlay the site. These terrace capping gravels are underlain by a 16,000-ft sequence of nearly flat-lying Paleozoic sedimentary rocks, the uppermost 250 to 300 ft of which are Pennsylvanian shales and sandstones of the Glenshaw Formation. These rocks are underlain by the Upper Freeport coal, a thick, laterally continuous seam that comprises the bulk of mineable coal reserves in the Pittsburgh area.

The historical record shows a striking absence of seismic activity in western Pennsylvania. Earthquake hazard is from distant sources, especially the New Madrid area in and around southern Illinois, the Fairport-Cleveland, Ohio, area, and the Attica, N.Y., area. Return period vs peak horizontal ground acceleration plots that characterize the seismic hazard at the Westinghouse PFDL are shown in Fig. 6. Additional details about the geology and seismic hazard characterization of the facility can be found in Ref. 2.

Building Description

The PFDL main building is a single-story, steel-frame, open-bay structure with a second-story penthouse that houses air-handling and other equipment. The first story is 60 by 240 ft in plan dimensions (Fig. 7) and is constructed using a series of transverse rigid frames connected together by girts and roof purlins to form a three-dimensional space frame (Fig. 8). The main plant roof consists of 2 to 4 in. of cast-in-place lightweight concrete on steel decking with built-up roofing and gravel over the concrete. The main floor is a 6-in. concrete slab on grade. The foundations for the transverse rigid frames are

FIG. 6. Return period vs peak horizontal ground acceleration for the Westinghouse PFDL, from Ref. 2. Note: return period data in the hazards model extends beyond the 10^4 yr cut-off of this and similar plots in this report.

concrete spread footings. Continuous footings that support the exterior walls extend between the column footings. The first story exterior walls consist of steel sheet-metal siding supported by steel girts. Steel studs covered with a cement plaster form the interior portion of these walls. Throughout the main plant area, there is a suspended plaster ceiling on framing channels supported by the roof purlins.

The penthouse area, which is 40 by 160 ft in plan dimension, is centered over the first-story main processing plant area. The penthouse structural frame is supported on the main structural rigid frame (Fig. 8). The penthouse roof consists of steel roof decking overlaid with rigid insulation covered with built-up tar and gravel roofing. The roof deck is supported on steel purlins that frame into the transverse rigid steel frames. The penthouse floor is constructed of a 4-in.-thick, 3000-psi, normal-weight, aggregate concrete slab with additional raised pads to support the equipment in the penthouse. The exterior walls of the penthouse area are steel siding

FIG. 7. A plan view of the first story of the Westinghouse PFDL.

FIG. 8 A series of transverse rigid frames, connected by girders and roof purlins, forms a three-dimensional space frame for the PFDL processing laboratory.

supported by steel girders. Detailed structural information about the PFDL appears in Ref. 7.

Critical Areas

PFDL representatives designated four building areas as housing equipment that contains or could contain hazardous chemicals. These general locations (cross-hatched areas of Fig. 7) are areas in which plutonium oxide is handled as a powder or liquid that could potentially be dispersed into the environment subsequent to loss of containment. Specifically, the four critical areas are the chemical processing laboratory (wet oxide), the ceramic processing laboratory (dry oxide), the process development laboratory, and the advanced fuel laboratory. The designated process equipment such as glove boxes, transfer piping, exhaust ducting, and vessels were also evaluated for structural capacity.

Modeling

Because the office area is essentially self-supporting and does not significantly affect the response of the symmetrical main processing plant structure, both torsional effects and interaction between the responses in the major orthogonal directions of the main plant are negligible. Therefore, we analyzed the longitudinal and transverse force-resisting systems as separate planar frames. The penthouse is so light that the PFDL structure also behaves effectively as a single-degree-of-freedom system, which is ideally suited to analysis by means of the approximate nonlinear ductility modified response spectrum approach.

Analysis Results

The overall longitudinal force-resisting system is composed of three structural systems that combine to carry lateral seismically induced forces. The primary system is the exterior steel siding that acts in shear. The secondary and tertiary systems consist of the longitudinal frame diagonal bracing and the main columns in weak axis bending. The weaker, but more flexible, backup systems augment the seismic capacity of the exterior siding and result in a combined median longitudinal capacity of 0.49 g (Table 3). The estimated one-sigma upper and lower bound longitudinal seismic capacities are 0.70 g and 0.33 g, respectively. The lower-bound structural capacity corresponds to a return period of well over 30,000 yr (Note: Fig. 6 and the other similar plots in this report do not go beyond 10^4 yr; however, data in the seismic hazards models, from which these plots were drawn, extend beyond the plotted values).

The transverse frames behave as ductile rigid frames, which are much stronger than the longitudinal force-resisting system. The median seismic capacity of the transverse frame is 2.1 g. Only minor damage occurs at accelerations that characterize the collapse of the longitudinal system.

The interior partitions and secondary architectural systems in the critical areas do not sustain major damage prior to the collapse of the main frame and, therefore, are not themselves critical in terms of release of hazardous material. The 4-in. and 8-in. masonry block walls located in the noncritical equipment mockup area are expected to collapse at median ground accelerations of 0.09 g and 0.18 g, respectively. However, their collapse is inconsequential in terms of material release.

Table 3. Summary of seismic capacities for the Westinghouse PFDL. Table entries are values of peak ground acceleration at which critical structures or equipment fail. (Source: Ref. 12)

Structural or equipment system	Seismic capacity, g		
	Lower	Median	Upper
<u>Longitudinal structural system</u>			
Exterior siding yield	0.10	0.15	0.20
Exterior siding failure	0.27	0.39	0.55
Longitudinal frame collapse	0.33	0.49	0.70
<u>Transverse frame</u>			
Anchor bolt yield	0.17	0.24	0.34
Main column plastic hinge formation	0.33	0.50	0.76
Transverse frame collapse	0.98	2.1 ^c	4.6
<u>Equipment and auxiliary systems</u>			
4-in. masonry wall collapse	0.06	0.09	0.13
8-in. masonry wall collapse	0.12	0.18	0.27
Oxygen bottle stand failure	0.07	0.12	0.20
Process vessel overturning	0.17	0.25	0.39
Pu-nitrate piping rupture ^a	0.26	0.39	0.56
Exhaust piping rupture ^a	0.26	0.39	0.56
Glove box crushing		0.49 ^b	
Hydraulic systems		0.49 ^b	
Glove box overturning (stand mounted)		0.62 ^c	
Glove box overturning (floor mounted)		0.79 ^c	
9% Hydrogen bottle stand failure		2.00 ^c	
Glove box stand failure		2.23 ^c	

^aOccurs as a result of exterior siding and diagonal bracing failure (longitudinal direction).

^bOccurs upon collapse of building.

^cIrrelevant since capacities exceed longitudinal frame capacity.

Most equipment items have greater structural capacity than the longitudinal system. They are generally only affected by

- Total facility collapse
- Large relative displacements between the floor and the main plant roof that occur just prior to collapse.

An oxygen bottle mounting stand and the process vessels located in glove box No. 241 (Fig. 7) are the only exceptions to this rule. These items fail at median ground acceleration values of 0.12 g and 0.25 g, respectively. These low capacities can be increased by minor structural modifications that should be made to eliminate equipment failure prior to collapse of the main frame.

Additional diagonal bracing would increase the longitudinal seismic capacity of the PFDL facility. However, as noted above, even the lower-bound capacity corresponds to a long return period, well over 30,000 yr.

BABCOCK AND WILCOX FACILITY

Geology and Seismology

The Babcock and Wilcox site is 0.5 miles southeast of Leechburg,, Penn., on the banks of the Kiskiminetas River, about 8 river miles upstream from its confluence with the Allegheny River. The site is crowded onto a 400- to 500- ft wide segment of the Kiskiminetas flood plain between the river channel and a relatively gentle terrace scarp that is about 140 ft high, and has an average slope of approximately 10% and a maximum slope of nearly 25%. Midway up this scarp, a 150-ft-wide artificial bench has been cut by a now inactive coal strip mine. The flood plain deposits of the site are underlain by a 16,000-ft sequence of Paleozoic sedimentary rocks. These rocks are only very gently folded, however, and only the top few hundred feet are significant to this study. These uppermost rock strata are Pennsylvanian age shale, claystone, sandstone, limestone, and coal of the Allegheny Group.

No faults are indicated on any geologic maps or in any other published literature describing the area. A striking feature of the historical record is the absence of seismic activity in western Pennsylvania--the historical seismic hazard in the Pittsburgh area has resulted from distant earthquakes. The most apparent clustering is in the New Madrid area, site of the famous 1811-12 earthquakes. Other areas of repetitive seismic activity include those around Fairport-Cleveland, Ohio, Attica, New York, and Anna, Ohio. Another important, though very distant, source region exists along the St. Lawrence Seaway. Return period vs peak horizontal ground acceleration plots that characterize the seismic hazard at the B&W facility are shown in Fig. 9. Additional details about the geology and seismic hazard characterization of the facility can be found in Ref. 3.

Building Description

The B&W facility is generally constructed of unreinforced hollow concrete block walls that support roof systems comprised of steel joists or beams with precast or cast-in-place concrete decks or metal decking.

FIG. 9. Return period vs peak horizontal ground acceleration for the Babcock and Wilcox facility. Note: return period data in the hazards model extends beyond the 10^4 yr cut-off of this and similar plots in this report.

Critical Areas

Four areas of the facility have key structural systems and equipment within them with the potential for release of dangerous chemicals (Fig. 10). These critical areas are FAB 1, FAB 6, the Nuclear Material Vault (NMV), and the Warm Equipment Storage (WES), and Drum Warehouse (DW) areas. The building components of interest are the bearing and nonbearing concrete block walls and the supported roof structure. The critical equipment items are glove boxes and filter housings in FAB 1 and FAB 6, and the vertical storage tanks in FAB 6. Of secondary concern are the overhead filter exhaust ducting (FAB 1 and FAB 6), the material storage containers (NMV), and the material storage drums (WES-DW).

The FAB 1 area is constructed of hollow 12-in-thick concrete block walls with Type-N mortar. The roof system consists of steel roof joists supporting

lightweight concrete roof planks and roofing. The roof joists span in the E-W direction and rest in mortar pockets in the N-S walls. Lateral force connectivity of FAB 1 through the corridor roof (adjacent to the east wall) to the NMV and WES areas is provided by friction of the roof deck on its supports at the FAB 1 east wall and the NMV west wall. Typical structural details of FAB 1 are illustrated in the section looking north shown in Fig. 11.

FIG. 10. Plan view of the Babcock and Wilcox facility showing critical areas.

All major walls of the FAB 6 area are also comprised of hollow 12-in.-thick concrete block with Type-N mortar. The roof system consists of steel joists supporting a steel sheet-metal roof deck that is welded to the joists. Roof joists, as in FAB 1, rest in mortared pockets within the east and west block bearing walls.

The NMV area roof construction consists of a reinforced concrete one-way slab on steel decking supported on exterior bearing walls and interior intermediate bearing walls. The walls are all of 12-in.-thick solid concrete block construction with Type S mortar. The concrete roof deck is tied into all exterior walls and bears on continuous bond beams. The WES-DW area construction is similar to that of the NMV except walls are of hollow block with brick facing. Also, the roof deck is supported on a grid of steel girders, beams, and columns. At the wall, the deck typically rests on wood blocking and is tied to the blocking. The blocking, in turn, rests on and is anchored to bond beams at the top of the wall. All columns, girders, and beams, where adjacent, are anchored to the walls by means of standard wall inserts.

FIG. 11. An E-W cross-section and close-up view shows that the lateral connectivity of FAB 1 through the corridor roof to the NMV area is provided by friction of the roof deck on its supports.

Modeling

Seismic resisting elements of the B&W facility consist of shear wall systems and transverse bending systems. In both cases, the roof structures and block walls interact. The roof systems act to distribute horizontal forces to the vertical wall systems and to tie the walls together up to the point of collapse. The roof and upper-wall sections also provide much of the mass that contributes to the inertial loading. The transverse bending systems are of greatest interest because they have the lowest collapse capacities. These systems consist of bearing walls supporting roof joists that act as ties, as well as individual nonbearing walls (See Fig. 11). Cracking at the base usually precedes major damage in such systems.

The block walls remain essentially elastic prior to cracking; hence, for the dynamic analysis, we used a single-degree-of-freedom elastic model and the response-spectrum method to calculate seismic cracking capacity. The median WASH 1255 horizontal spectra for alluvium sites were used with median system damping of 7% at the $A_{\max}$ corresponding to the wall cracking.

After cracks propagate through an entire wall, the wall, with or without the roof system, behaves as a rigid body (an inverted pendulum) with instability and collapse occurring when the deflection at the top of the wall becomes too large. The input energy required to cause rigid-body deflections of wall systems is very large compared to the energy needed to cause cracking. We selected and verified the reserve-energy method as the approximate dynamic analysis for evaluating the capacity of the building wall/roof systems in the rigid-body response mode.

Analysis Results

The most probable modes of failure for FAB 1 and FAB 6 occur with the east and west walls subjected to E-W horizontal ground motions. These walls respond as tall cantilevered beams subjected to transverse flexure until cracked through at the base. Subsequent response (up to failure) is considered to be in a cantilevered rigid-body mode, and collapse will most likely occur when these bearing walls become unstable because of excessive deflections (several inches) at their tops. We estimate that such deflections will occur at the ground accelerations given in Table 4. The return period corresponding to an $A_{\max}$ of 0.21 to 0.22 g (FAB 1 and FAB 6) is

TABLE 4. Seismic capacities of the Babcock and Wilcox facility under controlling E-W shaking, from Ref. 13.

Structural or equipment system	Mode of failure	Capacity in terms of A_{max} , g		
		Lower	Median	Upper
FAB 1 west wall/roof resisting system ^a	Cantilever rigid body	0.14	0.21	0.31
FAB 6 east wall/roof resisting system ^b	Cantilever rigid body	0.15	0.22	0.33
NMV non-bearing exterior wall ^c	Rigid body base and roof	0.37	0.53	0.77
WES-DW roof diaphragm ^d	Torsional shear transfer to east and west walls	0.30	0.38	0.48
<u>Equipment</u>				
Typical glove box and primary filter ^e	Seismic lateral stability—sliding	0.09	0.13	0.21
NMV material containers ^f	Buckling from missile impact	-	-	-
Storage drums ^g	Buckling from missile impact	0.30	0.38	0.48

^aControls FAB 1 collapse.

^bControls FAB 6 collapse.

^cControls collapse of NMV.

^dControls collapse of WES/DW.

^eGlove box slides.

^fNo failure with roof collapse.

^gRupture with roof collapse.

approximately 16,000 yr. Structural collapse of FAB 1 and FAB 6 would likely result in falling roof girders or joists striking and rupturing some glove boxes and thereby releasing dangerous materials.

Glove boxes may slide off their supporting stands at a median ground acceleration of 0.13 g. The corresponding return period is about 2900 yr. One-sigma bounds are 0.09 g and 0.21 g, respectively. Also at this level of

ground motion, secondary filter ducts may be torn, and primary filter boxes may be damaged upon striking the floor. Anchoring boxes to stands and stand legs to floors will increase seismic capacity of critical glove boxes, filters, and ducts to well above the collapse level of the main wall/roof systems.

Subjected to horizontal earthquake motions, the seismic-resistance structures in the NMV and WES-DW areas will act as shear wall/roof diaphragm box systems with significant seismic resisting capacities. Such capacities of the WES-DW area are limited by the shear transfer capacity between the roof diaphragm and shear walls. The median collapse level of this area is 0.38 g, with 0.30-g and 0.48-g one-sigma lower and upper bounds, respectively.

Failure of the NMV area structure is controlled by local out-of-plane response of the shear walls rather than in the in-plane shear mode. Considered for out-of-plane response, the walls act as vertical beams simply supported at the top and bottom (after cracking) and subjected to transverse seismic motions. After flexural cracking, the walls respond as two rigid-body segments pivoting about the wall base and roof line. The point of maximum elastic deflection is near mid-height of the wall, the same point at which rigid-body displacement peaks for each segment. The median collapse $A_{\max}$ for the NMV area is 0.53 g, with 0.37 g and 0.77 g one-sigma lower and upper bounds, respectively.

The return period for events with $A_{\max}$ greater than 0.3 g--the collapse levels in the NMV and WES-DW areas--is greater than 30,000 yr (Note: return-period data is from the hazards model, not the plots shown in this report). Structural collapse of the NMV area will cause the roof slab to fall, but the material containers will not rupture when struck by the slab or collapsing walls. The containers are strong and impact resistant by virtue of their construction of heavy wall pipe with heavy screwed-on end caps. The storage drums in the WES-DW areas will likely rupture if struck by falling roof beams or girders during collapse.

In summary, note that with a relatively minor remedial effort of anchoring glove boxes and stands, all significant damage and collapse would occur at a median $A_{\max}$ of 0.2 g or greater, which corresponds to a return period of 12,000 yr and greater.

BATTELLE MEMORIAL INSTITUTE COLUMBUS LABORATORIES

The Battelle Memorial Institute (BMI) Columbus Laboratories are located in West Jefferson, Ohio, approximately 17 mi west of Columbus.

Geology and Seismology

The surface geology of the BMI site is principally glacial material--clay, sands, and gravels--deposited by the Wisconsin ice sheet. Alluvium (river-laid deposits) is found in places. Beneath the glacial and alluvial deposits are several hundred feet of almost horizontal beds of limestone, dolomite, and shale, which comprise the bedrock of the area.

Examination of the historical seismic record reveals the most apparent clustering to be in the New Madrid area, site of the famous 1811-12 earthquakes. These earthquakes were the largest ever experienced in the eastern United States. Another area of significant seismicity is the region around Anna, Ohio. Other areas of repetitive seismic activity are centered on Fairport-Cleveland, Ohio, and Attica, New York.

In developing source regions for the seismic hazard characterization, we modified the Algermissen and Perkins source regions as discussed in Ref. 4. Return period vs peak horizontal ground acceleration plots that characterize the seismic hazard at the BMI site are shown in Fig. 12. Additional details about the geology and seismic hazard characterization of the facility can be found in Ref. 4.

Building Descriptions

We were concerned, in general, with the Nuclear Sciences area, and, in particular, with (1) the JN-1 Hot Cell Laboratory, which is equipped for post-irradiation examination of reactor materials and components up to and including light-water-reactor fuel assemblies, and (2) the JN-4 Plutonium Laboratory, which is equipped for research and development of plutonium-bearing materials.

Built in 1971 as an expansion immediately adjacent to the JN-1A Laboratory, the JN-1B Hot Cell Laboratory consists of a single-story, high-roof, braced-frame steel structure that houses the high energy cell (HEC) and its ancillary equipment. The main portion of the building is 86 by 74 ft in plan dimension (Fig. 13) and approximately 62 ft high. With the exception

FIG. 12. Return period vs peak horizontal ground acceleration for the Battelle Memorial Institute Columbus Laboratories.

FIG. 13. Plan view of the high energy cell, the cask washdown area, and the connecting pool in the JN-1B Hot Cell Laboratory.

of the east wall, which is a curtain wall adjacent to the JN-1A structure, the exterior walls are constructed of free-standing unreinforced concrete block with brick veneer from ground level to 10 ft above grade. The walls above 10 ft are constructed of double-layered metal panels attached to the steel framing.

The HEC is constructed of poured-in-place concrete walls and ceiling, with steel liners on all walls. Wall thicknesses are from 4 to 6 ft at floor level to approximately 16 ft elevation, and from 3 to 5 ft up to the roof. The roof is 4-ft-thick concrete. Interior dimensions of the cell are 9 by 38 ft in plan and 25 ft in height.

The JN-4 Plutonium Laboratory (Fig. 14) is a U-shaped structure with legs of unequal length and width. Completely different methods of construction characterize the various parts of the building erected in four stages between 1960 and 1968. However, no hazardous chemicals are contained in the older portion of the structure, and with the exception of some light sheet-metal flashing, the structures are separated by a 6-1/4 in. gap. As a result, the different structures will tend to behave independently during an earthquake.

The investigation focused on the main laboratory, which is constructed of unreinforced masonry block and block-and-brick walls on a concrete slab. A 16-ft-high roof is comprised of 1/2-ft-deep by 2-ft-wide by approximately 12-ft-long prefabricated concrete channel sections that span steel roof beams. The channels support 1.5 in. of rigid insulation and the built-up roof. A drop ceiling is installed at the 12-ft level to provide a crawl space for the building ventilation and exhaust systems.

Critical Areas

The area of primary concern is the HEC, but we gave additional attention to such items as viewing windows, HEPA filters, and exhaust system. The HEC provides the primary confinement barrier for hazardous chemicals in the JN-1B facility, while the building exterior walls and roof provide the final barrier.

The areas of principal concern in the JN-4 Plutonium Laboratory are glove boxes 20 and 37 (Fig. 14), which contain plutonium, uranium, or both, in dispersible forms. Other critical equipment are windows, final filters, the natural gas line in the service room, hydraulic fluid reservoirs in the main

FIG. 14. Plan view of the JN-4 Plutonium Laboratory showing critical glove boxes.

lab, and 1A-gas cylinders. The confinement barriers for the main lab portion of the JN-4 building consist of:

- Process glove boxes as primary confinement barriers
- Building walls and roof as final barriers
- Non-load-bearing concrete block walls and drop ceiling as secondary barriers within the building envelope.

Modeling

For use in the dynamic-response calculations, finite element models were developed to determine equivalent stiffness characteristics of the major structural systems and subsystems. These same models were used to determine the component stress levels resulting from given levels of ground motion. They were suitably modified or replaced with more representative models as the levels of ground motion were increased and significant nonlinear response occurred.

The lateral force resistance of the HEC is provided by the thick concrete shear wall box system tied together by the 4-ft-thick reinforced concrete roof slab. The HEC is rectangular in plan with a length-to-width ratio of 2.5:1; hence, its resistance to overturning in the E-W direction (long axis) is significantly greater than in the N-S direction. Because of the large mass and stiffness of the HEC and the weak coupling through the mezzanine and crane framing, the HEC will behave virtually independently of the main steel building frame.

The lateral force-resisting system of the JN-1B steel frame building is a box system comprised of vertical braced frames tied together by a roof system consisting of a roof truss and steel deck. The east wall is a curtain wall that does not extend to ground and is not tied to the JN-1A structure. Consequently, it acts as a deep flange to provide roof stiffening but does not act as a shear wall. The structure responds as an open section and all lateral loads must be resisted by the other braced frames.

In the main laboratory of the JN-4 facility, lateral force is resisted by a steel-column/unreinforced-masonry-wall system that supports the roof. Inertial roof loads are transmitted to the exterior walls at the steel column locations where the flexible columns bear against the masonry blocks. At very low levels of response, there is enough friction between the roof channels and

beams to provide diaphragm action of the roof. After the channels begin to slip, forces are developed in the individual channel slabs that oppose rotation and provide restraint against deformations of the roof system relative to the shear walls.

Analysis Results

Critical seismic capacities are summarized in Table 5. Uplift of the HEC footing was calculated to begin for rocking about the major axis at a median A_{max} of 0.23 g. Also, at A_{max} between approximately 0.3 and 0.4 g, some shield plugs may be shaken out of the HEC, further increasing the unfiltered leak path area to the atmosphere. Overturning of the cell about the same axis is not anticipated until A_{max} is 3 g or more. Collapse of the JN-1B steel frame structure was calculated to occur at a median A_{max} of 0.3 g in the N-S

TABLE 5. Summary of seismic capacities for the BMI facilities in order of increasing median values of peak horizontal ground acceleration.

Structural and equipment damage	A_{max} at which capacity is exceeded, g		
	Lower	Median	Upper
Buckling of the diagonal member in the JN-1B north wall for E-W excitation	0.08	0.11	0.15
Collapse of the steel framing and roof of the JN-4 Main Lab for E-W excitation	0.11	0.17	0.26
Initiation of uplift of the HEC	0.18	0.23	0.31
Collapse of the steel framing and roof of the JN-4 Main Lab for N-S excitation	0.17	0.26	0.38
Loss of roof truss and collapse of JN-1B Frame	0.17	0.3	0.54
Shaking loose of HEC shield plugs	0.21	0.35	0.55
Overturning of the HEC	_____	3	_____

direction. Although not enough damage will result to rupture the HEC, the exhaust fan system will fail, resulting in loss of the pressure difference across the cell boundary and allowing some unfiltered air access to the cell.

The critical equipment items that form the primary confinement barrier in the JN-4 Plutonium Laboratory exhibit higher capacities than do the building structures. In general, they will only be affected by total facility collapse or by large relative displacements between the floor and the roof, which occur just prior to collapse. Median $A_{\max}$ values of 0.23 g in the N-S direction and 0.17 g in the E-W direction were calculated for collapse of the main laboratory roof/wall system.

Thus, potential release of hazardous chemicals from any of the BMI facilities is not expected until $A_{\max}$ reaches 0.17 g. The associated return period is greater than 2000 yr (Fig. 12). Estimated one-sigma lower and upper bound seismic capacities are 0.11 g and 0.26 g, respectively.

EXXON NUCLEAR COMPANY MIXED OXIDE FUEL
FABRICATION PLANT (MOFP)

Geology and Seismology

The Exxon Nuclear Company site is in the Pasco Basin of south-central Washington. It is within the Columbia River Basalt Plateau, which extends over 50,000 square miles. The site is underlain by thick, widespread basaltic lava flows at a depth starting at 151 ft and continuing downward over 10,000 ft. The surface comprises a layer of Winchester sand, 2 to 7 ft thick, underlain by 20 ft of the Pasco gravels. The sand and gravel rest upon the deeply eroded surface of the Ringold Formation which, where not eroded, consists of a thick series of silts, sands, and gravels. The Ringold Formation rests on the basalt, the top of which is estimated to be at a depth of 151 ft below the surface.

The relatively few faults that have been mapped are directly related to folds within the basalt, the most prominent structural features in the Columbia Basin. This relationship suggests that many of the faults represent local adjustment to the folding. In general, the faults found within the basin are of limited length, of moderate offset, and characterized by the absence of structural features usually associated with recent fault activity.

Examination of the historical record suggests a relatively low hazard at the site. This is because the large historical earthquakes have occurred in distant source regions, and the seismicity of the nearby source regions is such that large earthquakes infrequently, if ever, occur. The distant source regions include the Puget Sound area, the Northern and Southern Cascades, and the Milton-Freewater area. Return period vs peak horizontal ground acceleration plots that characterize the seismic hazard at the Exxon site are shown in Fig. 15. Additional details about the geology and seismic hazard characterization of the facility can be found in Ref. 5.

Building Description

The MOFP is a windowless, two-story combination precast/cast-in-place concrete building of plan dimensions 100 x 114 ft and 28 ft in height. The general layout of the facility is shown in Fig. 16. All fuel manufacturing and processing is conducted within a 76 x 100 ft high-bay area (one story),

FIG. 15. Return period vs peak horizontal ground acceleration for the Exxon site. Note: return period data in the hazards model extends beyond the 10^4 yr cut-off of this and similar plots in this report.

separated from the remaining portion of the building by a 10-in.-thick reinforced concrete wall. A one-story, high-bay reinforced concrete vault with minimum 18-in.-thick walls is located in the northeast corner of the building. The building roof is insulated metal decking supported on steel open web joists. The vault roof is an eight-inch-thick reinforced concrete slab strengthened by wide-flange steel beams attached by bolts through the slab thickness. The slab and roof beams transfer the dead weight directly to the vault walls.

Exterior walls are precast, tilt-up reinforced concrete panels that are joined by cast-in-place columns between each panel. A cast-in-place 12 x 14 in. roof edge beam joins the columns and panels around the entire periphery of the building. Panel reinforcing steel is extended and hooked within the column and beam reinforcing cages. Each panel is placed upon the footing walls with a mortar bed. No positive connection exists between the footing wall and each panel.

FIG. 16. Plan view of the Exxon MOFP building, showing critical areas of mixed oxide fuel preparation and storage vault.

A diaphragm consisting of steel deck is welded to the main roof beams and connected to shear walls by welds to the peripheral steel chord members, which are anchored to the walls at the roof line. A unique structural feature of the MOFP building is the horizontal roof truss, which is above the deck diaphragm and does not support any roof dead load (Fig. 17). The truss acts as a redundant roof diaphragm that ties the high-bay walls together and allows an alternate path for shear transfer (through stud anchors into the roof edge beam) between wall elements.

Critical Areas

Critical areas of the MOFP are the mixed oxide preparation area and the storage vault (Fig. 16). The vault area is of secondary concern to the glove boxes, in which hazardous chemicals are processed, in dispersible form. Confinement barriers include: the process glove boxes as primary confinement barriers; the building walls and roof as final barriers; and a nonstructural

FIG. 17. Three-dimensional view of the MOFP building shows the roof truss above the deck diaphragm that supports no dead load.

gypsum board/steel stud partition in combination with a structural wall as a secondary barrier within the building envelope. Primary and secondary confinement is provided within the vault by the storage canisters in which the hazardous chemicals are transported. The canisters are supported on racks of special design. Final confinement for the storage vault is provided by the vault walls and roof.

Modeling

Seismic lateral force resistance of the structure is provided by a shear wall box system tied together by the steel roof diaphragm and the redundant horizontal roof truss (Fig. 17). The building structure may be considered to resist seismic forces as two independent systems--one for each (N-S and E-W) major building direction. Because of the diaphragm and truss flexibility and general configuration of the MOFP building with regard to mass and structural rigidity, torsional coupling of the two systems will be negligible. For both systems, the roof and the tributary wall inertia is transferred to the active panel shear walls by the diaphragm and roof truss. Shear is transferred through panel friction and dowels with shear keys at each column footing. The in-plane wall seismic shear forces are transferred to grade through the combination wall and spread concrete footings. Floor slabs are supported at grade without ties to the wall footings.

Analysis Results

Using independent, equivalent finite element dynamic models, we found those median levels of $A_{\max}$ at which stress levels in critical structures and components exceeded our criteria for failure. The controlling values of $A_{\max}$ --1.37 to 1.8 g--were all associated with loss of diaphragm and truss support for the panel walls (see Table 6). The values were based on the uncoupled response of the structure in each principal direction as determined from the independent dynamic models. The actual behavior of the structure for ground motions in excess of 1.3 g will involve joint slippage at the panel-foundation wall interface. The two lateral force systems will become coupled due to torsional effects. However, establishing a precise value within the range 1.3 to 1.8 g appears to be unwarranted in view of the long associated return periods ($\sim 10^5$ yr, see note, Fig. 15).

TABLE 6. Seismic capacities for critical structures and equipment in the Exxon MOFP in terms of peak horizontal ground acceleration.

Structural and equipment damage	Ground acceleration, g		
	Lower	Median	Upper
Roof truss pin connection failure (N-S motion)	1.09	1.37	1.72
Roof truss parapet wall support failure (E-W motion)	1.00	1.46	2.11
Roof truss support channel failure (N-S and E-W motion)	1.10	1.60	2.30
Diaphragm internal connection failure (N-S motion)	1.22	1.80	2.65
Glove boxes leveling bolt failure	5.7	6.8	8.1

For this study, we believe that the median collapse capacity of the MOFP building may be approximated by the median seismic capacity of the N-S force-resisting system (1.37 g). We estimate standard-deviation upper- and lower-bound seismic capacities as 1.09 g and 1.72 g, respectively.

ATOMICS INTERNATIONAL NUCLEAR MATERIALS DEVELOPMENT FACILITY (NMDF)

Atomics International operates the Nuclear Development Field Laboratory (NDFL), which is located at the site of the Liquid Metals Engineering Center at Santa Susana, Calif., approximately 29 mi northwest of downtown Los Angeles. The building of interest for the Natural Hazards Study within the NDFL is Building 055, denoted as the Nuclear Materials Development Facility (NMDF). The NMDF was built in 1966 and 1967.

Geology and Seismology

The NMDF is located in the Los Angeles area of the Transverse Range Geomorphic Province of California. The geology of the southern Ventura County part of the province is dominated by the Ventura basin, which extends westerly under the Pacific Ocean. This is an east-trending region that has been down-warped and in which has been deposited a great thickness of sediments during the last 60 to 75 million years. The sedimentary rocks have been tectonically deformed and partly uplifted to form hills and mountains.

Major folding and faulting in the area took place during the last one to five million years as a result of north-to-south compression. Fault zones developed include the San Andreas, the Santa Ynez, the Newport-Inglewood, the San Gabriel, and the Malibu Coast faults. Two features characterize the historical record of the Los Angeles area--the large number of active faults (>12) and their relatively low rate of activity. Because of these features, we were unable to characterize the seismicity of individual faults based on the historical seismic record. Instead, we assessed their seismicity based on the average annual slip rate of each fault. This departure from the general methodology is explained in detail in Ref. 6, which also presents a complete review of the site's geology and seismology. Return period vs peak horizontal ground acceleration plots that characterize the seismic hazard at the AI site are shown in Fig. 18.

Building Description

The NMDF is a one-story, windowless tilt-up concrete building. It has a lightweight concrete-fill steel roof deck with roof beam supports and a

FIG. 18. Return period vs peak horizontal ground acceleration for the Atomics International site at Santa Susana, Calif.

concrete floor slab on grade. The rectangular building is approximately 62 by 202 ft in plan dimension and 17 ft high (above grade). All of the work directly related to fuel processing is carried out in the large glove box room (Fig. 19) which is separated from the rest of the building by gypsum board/steel stud partitions. A vault located on the exterior side of the west wall is a cast-in-place concrete box extending approximately 11 ft above the finished floor line. A small mezzanine (partial second floor) is located adjacent to the west wall.

The roof of the NMDF building is a relatively flexible steel diaphragm. It consists of lightweight concrete fill on steel decking that is welded to the main roof beams and connected to shear walls by welds to peripheral steel chords (Fig. 20). The chords are anchored to the walls at the roof line.

FIG. 19. Plan view of the Atomics International NMDF showing critical areas.

FIG. 20 Three-dimensional cutaway view of the NMDF shows the relatively flexible steel diaphragm roof comprised of lightweight concrete fill on steel decking that is welded to the main roof girders. The girders are welded to a peripheral steel diaphragm chord that is anchored to the walls at the roof level.

Critical Areas

Critical areas of the NMDF include the glove box room, in which mixed carbides are processed, and the storage vault (Fig. 19). The confinement barriers for the glove box room consist of the process glove boxes as primary confinement barriers, the building walls and roof as final barriers, and nonstructural gypsum board/steel stud partitions as secondary barriers. The vault area is of secondary concern.

Modeling

The lateral force-resisting system is a shear wall box system tied together by the roof diaphragm. The structure resists seismic forces as two independent systems, one for each major building direction, N-S and E-W (Fig. 21). Because of the diaphragm flexibility and the building's large length-to-width ratio 3.25:1, torsional coupling of the two systems will be negligible. For both systems, the roof and the tributary-wall inertia is transferred to the active panel shear walls by the diaphragm acting as a deep beam with chord flanges. The vault is weakly coupled to the E-W system through flexure of the above-vault wall panel (Fig. 18). The in-plane wall seismic shear forces are transferred to grade through individual spread footings and through the building floor slab. The slab is positively connected to the wall panels near the wall base for this purpose.

Analysis Results

Using independent finite element dynamic models, we found those values of $A_{\max}$ at which critical structures and equipment fail. The controlling levels of 0.60 to 0.87 g were all associated with loss of diaphragm support for the west and east walls (see Table 7). Once the diaphragm resistance is lost, the roof girder/column frames and attached wall panels will progressively collapse after only a few cycles of motion. The median seismic capacity of the E-W force resisting system is 0.60 g; estimated 1-sigma standard deviation upper and lower bound seismic capacities are 0.88 g and 0.4 g, respectively. The best-estimate curve of $A_{\max}$ vs return period (Fig. 18) indicates that the facility will experience 0.60 g with a return period of 750 yr.

FIG. 21. The NMDF structure resists lateral seismic forces as two independent systems. In both systems, the diaphragm acts as a deep beam with chord flanges to transfer roof and tributary-wall inertia to the panel shear walls.

TABLE 7. Values of A_{max} at which critical structures fail.

Structural damage	Peak ground acceleration, g		
	Lower	Median	Upper
Diaphragm chord failure (E-W motion)	0.41	0.60	0.88
Diaphragm internal connection failure (E-W motion)	0.42	0.62	0.91
Diaphragm internal connection failure (N-S motion)	0.56	0.83	1.22
Diaphragm perimeter connection failure (N-S motion)	0.59	0.87	1.28

GENERAL ELECTRIC VALLECITOS NUCLEAR CENTER

The General Electric Company operates a special nuclear materials facility at its Vallecitos Nuclear Center near Pleasanton, CA, about 30 mi southeast of Oakland. The site, which is situated within the Diablo Range of the California Coast Ranges, is underlain by deformed older sedimentary materials including the Livermore gravels.

The site is located in an area of high seismicity from the major Calaveras, Hayward, and San Andreas faults of the San Francisco Bay area. Controversy surrounds the exact locations and capabilities of smaller, closer faults, especially the Verona fault, which may pass through the site. As a result, two seismic studies were performed. In the first study, we determined the seismic hazard at the site from the Calaveras, Hayward, and San Andreas faults and from smaller unassociated earthquakes that could not be attributed to these faults.²⁸ In the second study, the controversial Verona fault was assumed to exist, and the hazard it poses to the site was investigated.²⁹

Return period vs peak horizontal ground acceleration plots that characterize the seismic hazard at the GE site from the major faults are shown in Fig. 22, as are similar plots for the Verona fault alone. Although straightforward techniques exist for combining the curves, such a combination is not recommended because the result would be conservative rather than a best estimate. The conservatism stems from the fact that the model for the major faults already includes a background model to account for seismicity in the site vicinity.

An additional probabilistic study of rupture displacement was also undertaken for the postulated Verona fault.²⁹ Because the methodology necessarily departs from that discussed earlier in this report, the reader should consult Refs. 29 and 28 for details of these seismic hazard characterizations.

Structural Evaluation

Structural evaluation of the GE Vallecitos facility was not within the scope of study for this NRC project.

FIG. 22. Return period vs peak horizontal ground acceleration for the General Electric site at Pleasanton, Calif., based on: (a) the seismicity of the Calaveras, Hayward, and San Andreas faults and smaller unassociated earthquakes that could not be attributed to these faults; (b) only the seismicity of the postulated Verona fault.

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