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**EARTHQUAKE ANALYSIS
OF THE DIAGNOSTIC CHEMISTRY BUILDING,
LAWRENCE LIVERMORE LABORATORY**

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Abstract

An investigation of the Diagnostic Chemistry building at the Lawrence Livermore Laboratory in Livermore, California was conducted to evaluate its ability to withstand the most critical of several postulated design-basis earthquakes for the Laboratory site. The methods of analysis used and the results obtained are discussed. Also presented are a detailed description of the building; brief discussions of site

geology, seismicity, and soil conditions; and the approach used to postulate the design-basis earthquakes.

Based on results it is concluded that the main structural system of the building should withstand the ground motions of the postulated design-basis earthquakes without catastrophic failure. However, damage should be expected. The most critical areas are pointed out.

Introduction

The Diagnostic Chemistry building (251) at the Lawrence Livermore Laboratory in Livermore, California was investigated for its compliance with the seismic requirements of the Atomic Energy Commission Minimum Requirement for New Plutonium Facilities, dated April 1971. Figure 1 shows the building from two different angles.

The Diagnostic Chemistry building was selected for study because most of the chemistry work carried on in the building involves radioactive materials of high-level alpha activity, in some cases in relatively large amounts. The building was constructed in seven increments. Different types of chemistry experiments are conducted in the various increments. Increment 1 houses a 6-in. lead cave facility for high Ci level work, a glove box laboratory room for chemistry work

supporting the cave operations, and a chemistry laboratory. Increment 2 is a single room containing a built-in water tank for handling and storing irradiated slugs prior to processing in the lead cave. Increment 3 contains chemistry laboratories utilizing glove box enclosures, while Increment 4 houses a mass separator for purifying and separating isotopes, additional physical and radiochemistry laboratories, and a radioactive waste disposal area. Increment 5 contains two laboratories and a storage room for radioactive-contaminated equipment. The southern portion of Increment 7 is a high-bay area providing space for a shielded-cell operating complex and related support activities such as a waste disposal, used box storage, reagent preparation, manipulator repair, and air lock. The northern end of Increment 7 provides space for the

a. View looking northeast

b. View looking southwest

Fig. 1. Diagnostic chemistry building.

instrument and counting room, assembly and storage room, machine shop office, and personnel facilities. Increment 5 is the only increment that does not house any of the potentially hot laboratories.

This report will summarize the results of the investigation. The earthquake analysis criteria used conform to the requirements stated in Minimum Requirement for New Plutonium Facilities. A summary of

those requirements is given in the Appendix.

Basically, the requirements are that such facilities be designed for two hypothesized earthquakes—a design-basis earthquake (DBE) and an operating-basis earthquake (OBE). The DBE is customarily defined as an earthquake that produces the maximum possible ground motion that could occur at the site, whereas the OBE is defined as the maximum probable

earthquake motion that could occur at the site.

Critical areas of the structure and their associated critical equipment should be designed to withstand the DBE. Remaining areas of structure and their associated equipment should be designed to remain operational under the OBE. Critical items are those structures, systems, and components whose integrity or operability is essential to prevent an accident or to mitigate the consequences of an accident should one occur. The design of critical items such as main structures should be made using dynamic analysis techniques and the El Centro spectrum or its equivalent.

The main structural system of the Diagnostic Chemistry building was considered a critical item and was analyzed to evaluate its ability to maintain its structural integrity after experiencing vibratory ground motion from a DBE.

Building Description

The Diagnostic Chemistry building has plan dimensions of approximately 149 ft $\times$ 207 ft. The building was constructed in 7 increments over a span of 12 yr. Figure 2 shows a plan of the building and its separate sections. All seven increments are contiguous with each other. However, while Increments 1-6 are structurally tied together, Increment 7 is structurally independent. Increments 1-6 are a one-story structure extending 12-16 ft above the finish grade. Increment 7 has both a low-bay and a high-bay portion; the latter extends approximately 24 ft above the finish grade.

Each increment of the building was independently designed in accordance with

To postulate a DBE, the site geology, geophysical properties, and soil conditions must be investigated and the faulting and seismicity of the area reviewed. An understanding of the manner in which structures respond to transient ground motions is also essential. These items are discussed in detail in Bernreuter and Tokarz.¹ However, this report includes brief discussions of the site geology, seismicity, soil conditions, and postulated DBEs.

The entire building was analyzed using the DBE response spectra developed for the site. In addition, Increment 7 of the building was analyzed using two 3-dimensional lumped-mass models of the structure. Both a static and a dynamic time-history analysis were performed on the models. The static analysis used the DBE response spectra. The dynamic analysis subjected the models to earthquake motions normalized to the peak motions postulated for the Laboratory site.

the minimum requirements of the Uniform Building Code, Type 3 construction.² Lateral forces were determined using a seismic coefficient of 0.133 (i.e., $V = 0.133 W$). Roof loads were assumed at 20 psf in contrast to a current roof loading of 30 psf, as shown in Fig. 3.

The building's roof is composite and gravel applied to a metal pan roof deck on Increments 3-6 and the low-bay portion of Increment 7. On Increments 1-2 and the high-bay portion of 7 there is a pyrofill gypsum roof deck. The roof decks are supported by steel beams and steel joists that bear on 8-in.-thick reinforced concrete masonry walls. The roof decks are

Fig. 2. Plan and sections of diagnostic chemistry building.

The foundation of all increments consists of continuous and spread footing, generally at a depth approximately 3 ft below the surface. In the high-bay area, which contains the hot cell, the foundation extends approximately 16 ft below the finished surface.

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Site Geology, Faulting, and Seismicity

The Lawrence Livermore Laboratory is in the southeast portion of the Livermore Valley. The geology, faulting, and seismicity of the area have been thoroughly reviewed and discussed in Ref. 1. The review was based solely on a literature search of the geology, faulting, and seismicity of the area. The most recent review of the subject was that reported by John A. Blume & Associates, Engineers,³ which was undertaken to establish seismic design criteria for a proposed plutonium facility at the Laboratory site. Other sources of information are Refs. 4-7.

GEOLOGY

The location and geological setting of the building site within the eastern portion of the Valley are shown in Fig. 7. The site rests on a gently sloping alluvial plain. Within a mile to the south are low hills called the Livermore Uplands. To the east and northeast are hills generally referred to as the Altamont Uplands.

Figure 8 shows two geological sections that are at right angles to each other and intersect very near the building site. The locations of these sections are shown in Fig. 7. It may be observed that the site is blanketed by unconsolidated alluvial deposits directly overlying the Livermore formation. These formations are of recent to Plio-Pleistocene origin and consist of an active deposit of gravels, sands, and clays. The thickness of the deposit varies along the sections. It is estimated to be on the order of 400 ft at the site. Older Miocene deposits underlie the Livermore formation. A more detailed des-

cription of the general characteristics of the rock and soil deposits in the area is included in Fig. 7.

FAULTING AND SEISMICITY

The locations of the faults considered most pertinent to this study are shown in Fig. 9. These include a number of the major faults which transect the Livermore Valley in close proximity to the Laboratory, as well as the San Andreas, Hayward, and Calaveras faults to the west. The faults within the vicinity of the Laboratory include the Tesla, Carnegie, Mocho, Greenville-Patterson Pass, and Corral Hollow faults.

All these faults are within the coastal portion of Central California, which is one of the most seismically active regions in the United States. Earthquake epicenters within a 60-mi radius of the Laboratory are also shown in Fig. 9.

Historical earthquake records indicate that several destructive earthquakes have occurred along the San Andreas fault. The largest had a Richter magnitude of 8.3 and occurred in 1906; the most recent was in 1957, with a magnitude of 5.3.* Along the Hayward fault destructive earthquakes occurred in 1836 and 1868. Modified Mercalli intensities for these earthquakes are estimated at VIII and X. Along the Calaveras fault it appears that in 1861 there was a major earthquake with intensity estimated between VIII and IX.

Closer to the Laboratory no earthquakes larger than 4.5 magnitude have occurred.

* All magnitudes in this report are based on the Richter scale.

Geologic section B-B' (as shown in Fig. 7)

Geologic section A-A' (as shown in Fig. 7)

LEGEND

- PIEZOMETRIC SURFACE
- - - - - PIEZOMETRIC SURFACE NOT PRECISELY DEFINED
- ? — ? — ? CONTACT NOT PRECISELY DEFINED
- - - ? - - - CONTACT APPROXIMATE
- // FAULT - DISPLACEMENT INDICATED BY ARROWS, DASHED WHERE INFERRRED

HORIZ. SCALE IN FEET
0 2000 4000 6000

Fig. 8. Geological sections through site. Information obtained from Ref. 3.

Fig. 9. Seismicity and fault locations.

In April 1943 an earthquake of magnitude 4.2 occurred in the vicinity of the Tesla fault near the intersection of East Avenue and South Vasco Road.

In the same month two seismic shocks, with magnitudes of 4.0 and 4.1, were recorded along the Mocho fault in the Livermore Valley.

Fig. 10. Design basis earthquake acceleration spectra.

Design Basis Earthquake

Before the structural integrity of the Diagnostic Chemistry building could be ascertained, a DPE had to be developed for the Laboratory site. It was felt that a reasonably conservative prediction of maximum possible ground motion at the site would be 0.8 g, (see Table 1) with a frequency distribution as shown in Fig. 10. A brief discussion of the approach used to arrive at the DBE follows.

MAGNITUDE AND LOCATION

It is generally accepted that earthquakes occur along active faults. The previous discussion on geology and faulting briefly identifies the major faults of interest. For ease of discussion those faults in the proximity of the Laboratory site will be considered together. The others to be discussed are the San Andreas, Hayward, and Calaveras faults. Historical seismicity records of the area give considerable evidence that the faults are active, as Fig. 9 indicates. Evidence of recent movement also exists for these faults. Therefore, these faults are considered as the probable locus of any major earthquake occurring near the Laboratory site.

There are several papers⁸⁻¹⁰ that attempt to correlate such parameters as earthquake magnitude, length of fault rupture, depth of focus, and the amount of fault slip. Generally, they consider all the data from the various types of faults together. The relationship between fault length and type and earthquake magnitude that might occur along the fault is not well defined. In the epicentral region the major effect of earthquakes with magnitude greater than 5 is on the duration of strong motion. For distances greater than 5-10 mi (depending on magnitude), the relation between magnitude and fault length is better defined.

There is considerable scatter in earthquake data, so only data ranges can be reasonably given. For example, Housner's correlation⁸ between length of rupture and earthquake magnitude suggests that an earthquake with a magnitude of 5-6 results from a fault rupture of only 2-5 mi and an earthquake with a magnitude of 6-7 from a rupture of 5-25 mi. Based on this correlation, any of the faults being considered for this DBE are long enough to yield an earthquake with a magnitude 6-7 (see Fig. 9).

Table 1. Magnitude and location of postulated DBEs for Laboratory site.

Location	Minimum epicenter distance (mi)	Magnitude	Maximum surface accelerations at Laboratory site (g)
San Andreas fault	35	~ 8.3	0.25-0.40
Hayward fault	17	7.0-7.5	0.30-0.40
Calaveras fault	10	7.0-7.5	0.30-0.50
Fault system near Laboratory site	0	5.0-6.5	0.60-0.80

It should be noted that the fault system in the proximity of the Laboratory site is complex and active, and apparently several of the faults are not only closely connected but are also connected to the larger Calaveras fault. However, it is difficult to establish a direct structural intersection of the different faults. Therefore, while it is not likely that an earthquake of magnitude greater than 5 will occur on the fault system, there is no way to eliminate this as a definite possibility. No published correlation exists to relate the probability of an earthquake of a given magnitude to the total fault length. Certainly along the Calaveras, Hayward, or San Andreas faults, major earthquakes can be expected. It is reasonable to expect earthquakes of a magnitude of 7-7.5 along the Calaveras and Hayward faults, while earthquakes of a magnitude greater than 8 are likely along the San Andreas fault. Table 1 lists the magnitude and location of the earthquakes which are postulated as DBEs for the Laboratory site.

MAXIMUM GROUND ACCELERATION AND FREQUENCY DISTRIBUTION

There are several different methods of predicting the maximum ground acceleration resulting from any given earthquake. Most of the methods were derived from data recorded at large distances from the epicentral region and therefore apply only for large distances from the epicentral region of the earthquake. Three approaches are generally employed. The first uses recorded surface motions from past earthquakes to extrapolate motions directly at some distance from the earthquake epicenter or causative fault.^{8, 11, 12} The second also uses recorded motions of past

earthquakes, but the records are modified to include an approximate site amplification factor before extrapolating for distances away from the earthquake epicenter.¹³⁻¹⁵ The third estimates the rock motion underlying the site of interest and then uses detailed site properties to compute the surface motion.^{16, 17} For earthquakes postulated along the San Andreas, Calaveras, and Hayward faults, all three of the above approaches were examined. The results were weighed against the seismic history of the area. It is estimated that a ground acceleration of 0.5 g is a conservative estimate of the maximum value at the Laboratory site induced by DBEs postulated along these faults.

Site factors are extremely important for distant earthquakes such as those located along the San Andreas, Hayward, and Calaveras faults. It is not clear what role they play in the epicentral region at higher ground accelerations. Therefore, for the earthquakes postulated near the Laboratory site, the maximum ground acceleration was estimated using the correlations suggested by Cloud and Perez,¹¹ with modifications to account for the Pacoima Dam record and modified results from the 1966 Parkfield earthquake. Therefore, for a DBE with a magnitude of 5-6.5 as postulated for the fault system near the Laboratory site, a maximum ground acceleration of 0.8 g can be considered a reasonable upper limit for structural design purposes.

The frequency distribution for the DBEs associated with both the fault system near the Laboratory site and the San Andreas, Hayward, and Calaveras fault systems is shown in Fig. 10. The distribution was established by using accelerograms of past earthquake records normalized to either

0.5 or 0.8 peak ground acceleration and generating response spectra for the various values of viscous damping. The smooth spectra shown in Fig. 10 were obtained by enveloping the response spectra. For the spectra associated with the three

major faults to the west of the Livermore Valley, the 1952 Taft (N21E) and 1940 El Centro (NS) spectra were emphasized. The Pacoima Dam (S74W) spectra were emphasized for the close-in fault system.

Site Soil Conditions and Response Analysis

Experience during actual earthquakes and limited theoretical studies indicate that site topography, site geology, and local soil conditions greatly influence surface ground response under earthquake conditions. A rather thorough ground response study employing computer analysis would be required to study local soil conditions and ground response reliably. Therefore, it was decided to make a preliminary study to evaluate the significance of the local soil conditions on surface response. Results of the preliminary study could then dictate the scope and nature of further studies and site investigations. For this site the results from the preliminary study indicated that no further study was necessary.

SOIL CONDITIONS

Subsurface soil investigations have been conducted at many building locations at the Laboratory. At the Diagnostic Chemistry building the investigation was carried out by the soils engineers of Harding, Miller Larsen and Associates.¹⁸

A total of six test borings, ranging from 4 to 80 ft, were drilled at the site. Each boring was logged by a field engineer. Selected core samples were clas-

sified by visual examination and laboratory testing. The results are shown in Fig. 11. The soils were classified in accordance with the unified soil classification system, also shown in Fig. 11.

Such investigations are generally used to define the soil characteristics for foundation design and are not sufficient to characterize the soil conditions for seismic studies. Therefore, the soils engineers of Shannon and Wilson, Inc. received a contract to provide a soil properties report that could be used for a preliminary seismic study of the ground area under the building site.

Figure 12 summarizes most of the information provided by Shannon and Wilson¹⁹ in a form most adaptable to the computer program used to evaluate the seismic response of the Laboratory site.

The soil conditions shown on the left side of Fig. 12 are based on a subsurface investigation carried out at the site of a proposed plutonium facility at the Laboratory, shown in Fig. 11. Layers of stiff clays and dense sand were found to reach a depth of 90 ft, at which depth it is felt that the Livermore formation proper begins. There is no definite information concerning the depth to the ground water table at the site; it was assumed that the water table is not more shallow than 100

Fig. 12. Soil properties used in site response analysis. Information obtained from Ref. 9.

ft. The influences of the water table elevation on the ground response spectra are of minor importance in the present case.

Laboratory tests performed during this investigation, together with previous soil data obtained at other sites within the Laboratory, provided the basic information about the unit weight, strength, and relative density of the recent alluvium. The data shown in Fig. 12 represent the average value of soil properties within that formation. Quantitative data for the Livermore formation have been estimated on the basis of the geological information.

The shear modulus values shown opposite each layer and plotted with a broken line at the right side of the figure were calculated on the basis of the shear strength

values (for clays) and relative densities (for sands), properly corrected for the higher gravel content and denseness of the granular soils. While the shear moduli vs depth variation as depicted in this figure would represent accurately the soil conditions at the location shown in Fig. 12, the average "overall soil properties" for the entire Laboratory site is better represented by the "average curve" shown in the same figure with a continuous heavy line.

In addition to the soil investigations, a seismic survey within the Livermore area was conducted by the Department of Water Resources of the State of California. The results of this survey indicate compressional-wave velocities of 2700 fps above a

depth of 90 ft and 5700 fps below 90 ft.¹⁹

Shear moduli calculated on the basis of the above field values are in the order of 5×10^6 psf for materials above a depth of 90 ft and 20×10^6 psf for the materials below 90 ft. Since these shear moduli correspond to strain levels smaller than those certain to take place during a major earthquake, the lower modulus values shown in Fig. 12 appear reasonable.

SITE RESPONSE ANALYSIS

The effect of local soil conditions on surface ground motion was evaluated by using a lumped-mass method suggested by Idriss and Seed.¹⁶ It is assumed that the ground motions developed near the surface of a soil deposit during an earthquake are attributed primarily to the upward propagation of shear waves from the underlying rock.

The analysis method involves the determination of the type and extent of the various soil layers extending from the ground surface to the underlying rock at the site. These soil characteristics are incorporated in a lumped-mass model of the soil deposit assumed fixed at the base or rock level; such a model is shown in Fig. 13. The masses are interconnected by springs that resist lateral deformations. These springs represent stiffness (shear) properties between any two lumped masses. Damping in the system is assumed to be linearly viscous.

The response of the model is calculated to base motions estimated for the rock. Using the procedures described by Seed et al.¹⁷ and Housner and Jennings,²⁰ it is possible to describe the motion characteristics of rock underlying a site as induced

by a particular earthquake, taking into account the magnitude and location of the earthquake.

Several computer runs were made to analyze the response of the soil deposit defined in Fig. 12. The calculations were made using a modified version of the computer program described by Idriss et al.²¹ The modified version was provided by Professor H. B. Seed,²² and includes more recent thinking regarding the variation of shear modulus.²³ This modification consisted primarily of using a square root variation of modulus with confining pressure instead of a cube root variation of modulus with overburden pressure.

The rock motions used in the computations were accelograms of several earthquake records normalized to peak accelerations estimated for the underlying rock at the Laboratory site, using curves suggested by Seed et al.¹⁷ Table 2 shows the estimated peak values and predominant periods of the rock motions for the DBEs postulated along the San Andreas, Hayward, and Calaveras faults. The accelograms used included the N21E component of the 1952 Taft, the NS component of the 1940 El Centro, and the S74W component of the Pacoima Dam (1971 San Fernando) earthquake records. These were selected because their predominant period matched those estimated.

Some of the results obtained from the analysis are given in Figs. 13 and 14. A comparison of these results with the DBE spectra developed in the following section show that maximum ground surface acceleration never exceeded those defined by the DBE spectra, and that the response spectra generated are in reasonable agreement with those established by the DBE spectra for fundamental periods less than 1.5 sec.

Earthquake record	Base rock motion		Soil deposit		Surface motion			
	Maximum acceleration A_b (g)	Predominant period T_b (sec)	Fundamental period T_d (sec)	Fraction of critical damping, β_d	Maximum acceleration A_s (g)	Predominant period T_s (sec)	Spectral values	
							Fraction of critical damping	Ratio S_o/A_s
Taft (1952)	0.2	0.35	1.39	0.108	0.24	0.35	0.02 0.05 0.10	4.6 3.0 2.0
	0.5	0.35	1.80	0.153	0.44	0.63/0.79	0.02 0.05 0.10	3.8 2.8 2.0
El Centro N/S (1940)	0.2	0.5	1.40	0.111	0.31	0.56	0.02 0.05 0.10	6.1 3.9 2.3
Pacoima Dam (1971) S74W	0.2	0.4	1.41	0.111	0.29	0.50	0.02 0.05 0.10	5.3 4.0 2.9

Fig. 13. Description of model used for site response analysis and some results.

Table 2. Estimated rock motions underlying Laboratory site.^a

Earthquake location	Minimum epicenter distance (mi)	Magnitude	Predominant period (sec)	Maximum acceleration (g)
San Andreas fault	35	8.3	0.5	0.10-0.25
Hayward fault	17	7.0-7.5	0.3-0.35	0.12-0.35
Calaveras fault	10	7.0-7.5	0.3-0.35	0.15-0.30

^a Based on Reference 17.

Fig. 14. Surface response spectra generated from accelerograms calculated from the site response analysis.

Earthquake Analysis

The main structural system of the Diagnostic Chemistry building was analyzed to evaluate its ability to maintain its structural integrity after experiencing several of the DBEs postulated for the site. Increments 1-6 and Increment 7 were

treated as separate structures. Although Increment 2 and 4 are contiguous with Increment 7, there is no structural connection between them.

Two methods of analysis were used, the static method and the 3-D static/dynamic

method. The static method is based on response-spectrum analysis and was applied to the structural systems of both Increments 1-6 and Increment 7. The 3-D static/dynamic method involved the development of two mathematical lumped-mass models of the structural system associated with Increment 7. The models were subjected to set static loads and to several estimated earthquake ground motions (accelerograms). The calculations were made using a three-dimensional frame code called TRILIN,²⁴ developed in the Structural Dynamics Group of the Nuclear Test Engineering Division at the Laboratory.

Both static and 3-D static/dynamic analyses were conducted on Increment 7 for two reasons. First, a comparison would allow a check on some of the assumptions made in the static method regarding the flexibility of roof systems. Second, by conducting a time-dependent analysis of the models, a positive check could be made regarding the possibility of dynamic amplification of the ground acceleration.

STATIC METHOD

A detailed analysis of the stiffness characteristics of the metal deck roof diaphragm and masonry shear walls was made. The results revealed that the metal deck roof system lacks the ability to transfer torsional forces and that it is also extremely flexible relative to the in-plane stiffness of the shear walls. Similar conclusions appear reasonable for the gypsum roof decking. Therefore, the masonry shear walls were assumed to resist the earthquake forces independent of each

other. It was also assumed that east-west building motions are uncoupled from north-south building motions. These assumptions required that a separate analysis be made for each direction.

Each of the shear walls was analyzed by replacing the earthquake forces with equivalent horizontal static loads at the roof levels and at the mezzanine level in the high-bay portion of Increment 7. The static loads were assumed proportional to the weight (or mass) associated with each wall.

The weights associated with each wall are shown in Fig. 15. These values include the weight of the roof system and a portion of shear walls. The distribution of the weights was made by considering the relative stiffnesses between the shear walls (both in-plane and out-of-plane) and the roof decking. While the in-plane wall stiffnesses are by far the largest, the out-of-plane wall stiffnesses are the smallest. Therefore, when considering earthquake forces in some specified direction, transverse walls weights were assumed to be transferred through the roof decking to the walls parallel to that direction. The distribution also reflects differences between the flexural/shear and axial stiffnesses of the roof decking.

Conservative estimates of the fundamental periods of the shear walls were made. Periods are so small that no amplification of the ground acceleration should occur, as shown in Figs. 10, 16, and 17. Therefore, the magnitude of the static loads was assumed equal to the weight of structure multiplied by the value of the maximum estimated ground acceleration expressed in g 's. As shown in Table 1 and Fig. 10(b), the maximum value estimated for this site is 0.8 g .

FIRST FLOOR PLAN
INCREMENTS 1 THRU 6

FIRST FLOOR PLAN
INCREMENT 7

MEZZANINE PLAN
INCREMENT 7

LEGEND

- WALL LOADS DUE TO NORTH-SOUTH WALLS (psf)
 - WALL LOADS (psf)
 - CONCENTRATED ROOF LOADS (psf)
 - ROOF LOADS (30 psf)
 - LOADS FROM 4" x 8" CONCRETE SLABS (psf)
- } BASED ON APPROXIMATELY 1/2 WALL HEIGHT

a. East-west

FIRST FLOOR PLAN
INCREMENTS 1 THRU 6

FIRST FLOOR PLAN
INCREMENT 7

MEZZANINE PLAN
INCREMENT 7

LEGEND

- WALL LOADS DUE TO EAST-WEST WALLS (psf)
 - WALL LOADS (psf)
 - CONCENTRATED ROOF LOADS (psf)
 - ROOF LOADS (30 psf)
 - LOADS FROM 4" x 8" CONCRETE SLABS (psf)
- } BASED ON APPROXIMATELY 1/2 WALL HEIGHT

b. North-south

Fig. 15. 1 g masonry wall loads (applied at roof or mezzanine levels).

a. East-west

▽ INDICATES MODE SHAPE
OF VERTICAL CONNECTORS

b. North-south

▽ INDICATES MODE SHAPE
OF VERTICAL CONNECTORS

Fig. 16. Mode shapes and fundamental periods of rigid lumped-mass models.

East-west walls*	Period (sec)	Mode shapes
19-20	0.054
	0.043	
8-9-10	0.053
4	0.034	
1-2	0.029	
11-12	0.028	
16-17-18	0.019	

North-south walls*	Period (sec)
O-P	0.060
Q	0.056
X-Y-Z	0.050
K	0.048
U	0.041
V	0.032
R	0.032

* See Fig 2 (a) for wall location

Fig. 17. Mode shapes and fundamental periods of flexible lumped-mass models.

3-D STATIC/DYNAMIC METHOD

Figure 18 shows a schematic of the two mathematical models used to represent the structural system of Increment 7. Both models consisted of lumped-mass representations of the structure fixed at ground level. The only difference between the models is the way in which the roof system was modeled. In one model, the roof was assumed to be a flexible diaphragm, while in the other it was assumed to be a rigid diaphragm. The models are referred to as the flexible and rigid models.

Mass points are located at both the roof levels and the mezzanine level in the high-bay area. The vertical connectors are assumed to be tied together at the mass points by a horizontal diaphragm representing either the roof systems or the concrete slabs at the mezzanine level. Each mass point was allowed to translate hori-

Fig. 18. Three-dimensional lumped-mass models of Increment 7.

zontally in both the north-south and east-west directions. Therefore, the analysis allows displacements to occur simultaneously in both the north-south and east-west directions for each set of earthquake loads.

The vertical connectors were assigned shear and flexural characteristics equivalent to those of the shear walls. Values of 1.5×10^6 psi and 0.20 were used for the modulus of elasticity and Poisson's ratio, respectively. The concrete slabs at the mezzanine level were assumed to be rigid diaphragms, and the horizontal connectors bordering these slabs were assigned shear and flexural properties accordingly.

For the static analysis, loads were assigned to each of the mass points. The magnitude of these loads was obtained using response spectra techniques and the procedures discussed in the static method. For the dynamic analysis, mass values were assigned to the mass points. These values were obtained by converting the

weights shown in Fig. 15 to mass by dividing by the acceleration of gravity. The models were then excited by accelerations of past earthquakes. The accelerograms were all normalized to the peak ground accelerations postulated by the DBEs.

The first three fundamental periods and mode shapes for the rigid model are shown in Fig. 16. For the flexible model, per-

iods are given in Fig. 17, corresponding to the lowest fundamental mode for each of the shear walls. Periods for higher mode shapes are all less than 0.020 sec except for the south wall of the high-bay portion.

RESULTS

The investigation of the building's structural ability to withstand the postulated

Table 3. Summary of maximum shear wall stresses and factors of safety (Increments 1 through 6).

Building ^a increment	East-west			North-south		
	Wall ^a	Maximum shear stress ^b (psi)	Factor of safety ^c	Wall ^a	Maximum shear stress ^b (psi)	Factor of safety ^c
1	23	7	5.7	G	36	1.1
	22	37	1.1	F	16	2.5
	14	25	1.6	E	22	1.8
				H	108	0.4
				J	10	4.0
2	21	37	1.1	J	10	4.0
	13	19	2.1	N	10	4.0
3	15	37	1.1	B	22	1.8
	7	38	1.1	E	22	1.8
				D	32	1.2
4	14	46	0.9	D	32	1.2
	13	21	1.9	M	22	1.8
	6	26	1.5			
	5	24	1.7			
5	24	26	1.5	C	22	1.8
	15	37	1.1	F	16	2.5
6	7	38	1.1	A	14	2.8
	6	26	1.5	I	26	1.5
	3	21	1.9			

^aSee Fig. 5(a) for increment and wall locations.

^bThese values were obtained using the static method for a 0.8 g maximum ground acceleration.

^cSee Table 6 for allowables.

DBEs was concentrated on three main areas—the shear walls, the roof decks, and the roof-deck-to-shear-wall attachments. The results obtained from the analyses are given in Tables 3-5.

All the results presented are based on static analyses using the DBE response spectra.

The dynamic analysis was conducted using both the flexible and rigid lumped-mass models. As expected, no amplification of the maximum ground acceleration was observed at any of the mass points. Therefore, except for the information given in Figs. 16 and 17, no other results of the dynamic analysis are presented. The accelerations of several past earthquakes were used as the exciting motion, an example

being the N21E component of the accelerogram of the 1952 Taft earthquake normalized to a peak acceleration of 0.5 g. A value of 5% viscous damping was assumed.

In Tables 3 and 4 the maximum shear stress and the factor of safety for each masonry wall are given. For Increment 7 the values are given for both the static method and 3-D static/dynamic method for the rigid model. The flexible model yielded approximately identical results to those of the static method.

In Table 5, the maximum shear load for each roof deck is given, as well as factors of safety for the roof decks and roof-deck-to-shear-wall attachments. The values given for the maximum roof

Table 4. Summary of maximum shear wall stresses and factors of safety (Increment 7).

Wall ^a	Maximum shear stress (psi) ^b		Factor of safety ^d	Wall ^a	Maximum shear stress (psi) ^c		Factor of safety ^d
	Static method	3-D static/dynamic method ^c			Static method	3-D static/dynamic method ^c	
1	22	22	1.8	R	18	41	1.0
2	20	22	1.8	X,Y,Z	46	47	0.9
4	28	29	1.4	K,L	28	34	1.2
8,10	56	50	0.8	O	78	24	0.5
11,12	29	20	1.4	P	60	40	0.7
16	15	11	2.7	U	38	33	1.1
17	15	12	2.7	V	29	27	1.4
18	16	12	2.5	S	72	41	0.6
19	36	41	1.0	T	107	53	0.4
20	45	38	0.9	W	30	65	0.6
				Q	47	38	0.9

^aSee Fig. 3(a) for wall locations.

^bValues are based on a 0.8-g loading maximum ground acceleration.

^cValues are based on rigid roof system. Flexible roof system yields same values as static method.

^dSee Table 6 for allowables.

deck loads are based on the loads shown in Fig. 15, modified for 0.8 g maximum loading. For some specified direction, the roof deck loads include loading due to both the roof deck and a portion of the transverse walls only.

The factors of safety given in these tables are based on the allowables given in Table 6. These allowables were obtained from the Uniform Building Code² and contain a one-third increase for short-duration loading.

Table 5. Summary of maximum shear loads and factors of safety of roof deck and roof-deck-to-wall-connections.

Building increment	North-south			East-west		
	Maximum ^a shear load (ppf)	Factor of safety ^b		Maximum ^a shear load (ppf)	Factor of safety ^b	
		Roof deck	Roof-deck-to-wall-connections		Roof deck	Roof-deck-to-wall-connections
1	926	1.6	1.1	1494	1.0	0.7
2	402	3.8	2.4	1874	0.8	0.5
3	614	1.2	0.6	2111	0.4	0.2
4	1730	0.4	0.2	993	0.8	0.4
5	1103	0.7	0.3	623	1.2	0.6
6	1800	0.4	0.2	567	1.3	0.6
7 High-bay	2796	0.5	0.3	1354	1.1	0.8
Low-bay	2160	0.4	0.4	809	0.9	0.9

^aValues are based on a 0.8 g maximum ground acceleration.

^bSee Table 6 for allowables.

Table 6. Allowable shear values.

Item	Building increment	Description	Allowable shear value
Roof deck	1,2,7 High-bay	Class B—pyrofill gypsum (2-1/2 in. thick)	1520 ppf
	3,4,5,6,7 Low-bay	Metal roof deck	760
Roof-deck-to-wall-connections	1,2	#3 anchor bolts @ 1 ft o.c.	980 ppf
	3,4,5,6	#4 anchor bolts @ 2 ft o.c.	365
	7 High-bay (N,S)	#6 anchor bolts @ 2 ft o.c.	730
	High-bay (E,W)	#7 anchor bolts @ 1.75 ft o.c.	1140
	Low-bay (E,W)	#6 anchor bolts @ 2 ft o.c.	730
	Low-bay (N)	#6 anchor bolts @ 4 ft o.c.	365
	Low-bay (S)	#7 anchor bolts @ 2 ft o.c.	960
Shear walls	All	Concrete block	40 psi

Conclusions

Based on the results of the analyses, several conclusions have been reached regarding the ability of the Diagnostic Chemistry building to withstand earthquake ground motions as defined by the postulated DBEs. Comments are directed to the masonry shear walls, roof decks, and roof-deck-to-wall connections.

In general, the masonry walls, which provide the building's primary resistance against earthquake forces, should maintain their structural integrity to ground motions defined by the DBE without catastrophic failure. However, as indicated by the low factors of safety in Tables 3 and 4, damage should be expected. The most critical walls are the west and north walls of Increment 7.

These low factors of safety can perhaps be increased. The allowable stress of 40 psi used for masonry wall is based on the Uniform Building Code² for masonry walls, where inspection is not required during construction. When an inspection is required, an 80 psi value is permitted. An inspection was required for this building; however, because the inspection was considered to be poor, the lower value was used. Nevertheless, some increase in allowable stress may be justified.

The roof decks are not as strong as masonry shear walls. Their design was established using a 20 psf roof weight in contrast to a current value of 30 psf. This increase results from the addition of mech-

anical equipment to the roof. Because the shear wall maximum loads include the weight of the walls themselves, their capacity is not affected as significantly as that of the roof decks by the increase in roof loads.

In spite of the significant increase in roof loads, the roof is still capable of withstanding more than twice its design load. However, for ground shaking as postulated with the DBE, damage should be expected. The most critical roof deck is the high-bay portion of Increment 7. The metal roof deck of Increments 3, 4, 5, and 6 appear equally critical, as shown in Table 5. However, the allowable of 760 ppf as recommended by the Uniform Building Code² could perhaps be doubled.

The roof-deck-to-masonry-wall connections are the weakest part of the building's main structural system. Nearly all of the factors of safety shown in Table 5 are less than one. These values were based on an allowable shear stress of 10,000 psi for the anchor bolts, which tie a continuous ledger located on top of shear walls to the walls. The roof decks are welded to this ledger. Although the allowable shear values could perhaps be increased by a factor of two or three, many of the factors of safety would still remain less than one. Therefore, for ground motions as severe as those postulated by the DBEs, considerable damage can be expected.

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Appendix

Summary of Seismic Design Requirements

Stated In

Minimum Criteria for New Plutonium Facilities
U.S. AEC Construction Division, April 1971

General. The structure and its associated critical equipment, ventilation, electrical, fire protection, and utility systems will be designed so that it will provide confinement of radioactive materials in the event of a design basis accident that can be postulated for the facility, including accidents caused by natural phenomena.

Structural design, loading combinations, and construction of critical safety and fire protection features should, as a minimum, be in accordance with the latest edition of pertinent nationally recognized codes. Where local codes or regulations are more stringent than the nationally recognized codes, they should be followed.

Seismic Design Resistance. Seismic parameters should be developed for the site to determine a design basis earthquake (DBE) and an operating basis earthquake (OBE). The smaller earthquake, the OBE, is equivalent to approximately one-half the DBE in terms of ground acceleration. Critical areas of the structure and its associated critical equipment, ventilation, fire protection, and utility systems should be designed to withstand the DBE. Remaining areas of the structure and its associated equipment should be designed to remain operable under the OBE.

- a. **Determination of Size of Earthquake.** Similar procedures to those stated in Regulator Criteria, "Seismic and Geologic Siting Criteria for Nuclear Power Plants," should be used for determining the quantitative design basis for vibrating ground motion at the site due to earthquakes.
- b. **Earthquake Spectrum and Damping Factors.** The design of critical items, noted below, should be made using dynamic analysis techniques and should use the El Centro spectrum or its equivalent.
 - (1) Main structures
 - (2) Fire protection systems
 - (3) Ventilation systems
 - (4) Confinement piping and equipment
 - (5) Critical utilities, instrumentation, monitors, and alarms
 - (6) Material in process and in storage (nuclear criticality).

* May be a smooth spectrum developed from an envelope of spectra related to the measured vibratory motions caused by more than one earthquake.

The ratio of vertical to horizontal acceleration should be two-thirds unless specific site considerations justify use of a different number.

The design of critical items should use damping factors selected by a qualified consultant.

- c. Critical Items. Critical items (such as those noted in b. above) are those structures, systems, and components whose continued integrity and/or operability is essential to prevent an accident and/or mitigate the consequences of an accident should it occur.
- d. Earthquake Occurrence. The design earthquake should be assumed capable of occurring at any time, except that the simultaneous occurrence with a design basis accident or with other limiting site related event such as tornado or flood need not be considered for design purposes.
- e. Load Combinations and Stress Limits. All critical structures, systems, and components should be designed to withstand:
 - (1) Load combinations including normal design loads and the OBE loads within normal working stress or deflection limits.
 - (2) Load combinations including normal operating loads and the DBE loads within yield strength. Allowable stresses under the DBE loading should not exceed 90% of the yield stress in reinforcing steel or 75% of the ultimate strength in concrete.
 - (3) Load combinations including normal operating loads, DBE loads, and applicable design basis accident loads, without loss of function of the specific structure, system, or component.

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