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**Seismic and Geologic Investigations of the
Sandia Livermore Laboratory Site and Structural
Investigations of the Tritium Research Facility**

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**SEISMIC AND GEOLOGIC
INVESTIGATIONS OF THE
SANDIA LIVERMORE LABORATORY SITE
AND
STRUCTURAL INVESTIGATIONS OF THE
TRITIUM RESEARCH FACILITY**

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**CONTRACTOR'S TECHNICAL REPORT
CONTRACT NO. 87-5028
SANDIA REQUESTOR W. Thompson
CONTRACTING REP. Jack P. Hubbel
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prepared for
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prepared by
URS/John A. Stume & Associates, Engineers
130 Jessie Street
San Francisco, California 94105

PREFACE

This report consists of two parts. Part I, Seismic and Geologic Investigations of the Sandia Livermore Laboratory Site, covers the review and investigation of existing data as well as the acquisition and analysis of new data from field explorations and field examinations of the site. Part II, Structural Investigations of the Tritium Research Facility, covers: (a) the structural evaluation of the existing buildings and vital systems of the facility, and (b) concept studies of various schemes to strengthen the buildings and systems.

PART I

SEISMIC AND GEOLOGIC INVESTIGATIONS
OF THE SANDIA LIVERMORE LABORATORY SITE

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INTRODUCTION

This report describes results of a seismic and geologic investigation by URS/John A. Blume & Associates, Engineers (URS/Blume), in the vicinity of Sandia Laboratories (Sandia) property and Sandia's Tritium Building at Livermore, California. The investigation was done for Sandia to define any seismically capable faults in the immediate area and to obtain necessary information to support estimates of future possible or probable ground motions. The work included a variety of geophysical measurements, trenching, seismologic studies, geologic examination, and evaluation of possible ground surface rupture at the site. Ground motions due to the maximum potential earthquake are estimated, and probability of exceedance for various levels of peak ground acceleration is calculated.

Descriptions of the various calculations and investigative techniques used and the data obtained are presented. Information obtained from other sources relevant to subsurface geology and faulting is also given. Correlation and evaluation of the various lines of evidence and conclusions regarding the seismic hazard to the Tritium Building are included.

Location and Site Description

Sandia is located in Livermore Valley about 2 mi east of the town of Livermore and about 38 air mi east of the city of San Francisco. The general location is shown in Figure 1. The laboratory property is located at the center of section 13, T.3S., R.2E. in the Altamont 7-1/2 min quadrangle. It is located in the east end of Livermore Valley, which is situated in a portion of the coast ranges about midway between the San Francisco Bay Area to the west and the San Joaquin Valley to the east. The laboratory area is bounded on the south and east by low hills that are a portion of the coast ranges, while the level floor of Livermore Valley extends to the west and north. Drainage in the area is to the northwest.

The section immediately north of Sandia across East Avenue is occupied by Lawrence Livermore Laboratory (LLL). Surrounding areas to the north and east of LLL property consist of open grazing land, and an oil field is situated in the area to the northeast of LLL. South and west of LLL, the lands

FIGURE 1 LOCATION MAP

are partly in pasture and partly under agricultural cultivation. Some residential development has taken place in the sections to the west.

Previous Work

"The geology of the Tesla quadrangle," a report by Huey (1948), was the first comprehensive study of the area surrounding Sandia. Earlier work that related to the region was reviewed by Huey. In 1966, a study of the ground-water hydrology in the Livermore Valley was published (California Division of Water Resources, 1966), and the U.S. Soil Conservation Service published a soil survey of the Alameda area (Welch et al., 1966).

In 1971, John A. Blume & Associates, Engineers (Blume), prepared a geologic and seismologic report, which was followed in 1972 by a definitive study of faulting in the area for the adjacent LLL.

Helley et al., 1972, published a geologic map of late Cenozoic deposits in Alameda County, and Wight (1974) authored a seismic evaluation of LLL for the University of California. More recently, in 1977, the U.S. Geological Survey (USGS) released an open file report on faulting in eastern Alameda County (Herd, 1977).

There have been geologic investigations of the area by oil companies and others, but the information has not been made public.

GEOLOGY

General Statement

Sandia is located in the southeast part of Livermore Valley. The valley is a structural depression, elongated in an east-west direction but dominated by northwest-southeast trending geologic structures. To the west, the valley is terminated by the Calaveras fault. On the east, it is bordered by the Carnegie fault. Both are extensive, well-documented right lateral faults bounding the valley. The southern border of the valley is determined by the plunge of structures that rise to the south, with some faulting possibly involved (Figure 2).

"The geology of the Tesla quadrangle" (Huey, 1948) covers the area of interest of this report. It indicates a complex history of tectonic activity with repeated episodes of deposition and erosion. This complexity is evident in the abrupt appearance and disappearance of surface and subsurface stratigraphic units, which has led to various and often conflicting geologic interpretations.

Stratigraphy

Franciscan. The Franciscan is the oldest stratigraphic unit and is the basement rock for this region. It is recognized as a subduction-derived melange, and its very heterogeneous nature has led to its being called a group, formation, tectonic unit, and terrane. Sediments predominate, consisting mainly of greywackes, shales, argillites, and cherts. Also prominent are volcanics and low-grade metamorphics, especially greenstones, serpentine, and greenschists. The Franciscan crops out about 3 mi southeast of Sandia. Its surface dips about 20° to the northwest, and it is overlapped by Livermore gravels. Generally, the age is regarded as Jurassic-Cretaceous.

Panoche Formation. The Panoche Formation is composed of indurated siltstones, silty shales, and dirty lithic feldspathic sandstones, with lesser amounts of conglomerate. Genetically they are regarded as deep-water marine turbidites with an eastern source. The sediments are hard and moderately well indurated as a result of their deep deposition and subsequent burial. The

Qal Alluvium

Qt Terrace Deposits

Qp Livermore Gravels

T Tertiary Deposits

K Cretaceous Deposits

Kjf Jura-Cretaceous Franciscan Foundation

Fault

Syncline

0 5 10
Scale: Miles

NOTE:

Modified after geologic map of California, San Jose Sheet, California Division of Mines & Geology.

FIGURE 2 GENERALIZED REGIONAL GEOLOGIC MAP

present surface exposure is more than 1-1/2 mi northeast of Sandia, but deep gas wells have penetrated the Panoche, and data indicate it is present underground about 1/2 mi to the northeast. The Panoche is upper Cretaceous, 65 to 90 million years old, and depositional contacts with all other formations are unconformable. In the Tesla quadrangle, it is reported to be up to 10,000 ft thick.

Tesla Formation. The Tesla Formation consists of friable, mature, quartzose sands, clays, shales, and low-grade coal. The lower part was predominantly deposited in brackish water, but the upper part was laid down under marine conditions. These factors indicate that the sediments accumulated in shallow waters during a marine transgression. The low density and lack of induration are probably due to the shallow depth of accumulation and lack of subsequent deep burial. The distance between Sandia and the nearest outcrop is 4 mi; however, a deep well has penetrated Tesla rocks 2 mi to the northeast, and it is probable that Tesla is present within a mile of Sandia in the subsurface. The Tesla Formation is middle Eocene in age and is unconformable in its depositional relations with other formations. The formation is up to 2,000 ft thick but is probably not over 1,700 ft thick in the Sandia region.

Clerbo Formation. The Clerbo Formation consists of friable, conglomeratic, poorly sorted sands, carbonaceous shales, and tuffs with lignite and petrified wood. The sands are superficially similar to those of the Tesla Formation, and the sediments were apparently deposited in a shallow transgressive sea. Lack of induration appears to be related to lack of deep burial. The nearest outcrop is 1-3/4 mi to the northeast, but it is less than 1/2 mi northeast in the subsurface. The Clerbo is up to 1,000 ft thick but probably not more than 900 ft in the Sandia region. The age is upper Miocene and possibly Mio-Pliocene. All depositional contacts of the Clerbo are unconformable.

Neroly Formation. The Neroly Formation can be divided into lower and upper parts. The lower part is composed mainly of blue sandstone with andesitic boulder conglomerates and tuffaceous clay-shales. The upper part consists mainly of clays with some soft sands, gravels, and blue sandstone. The Neroly is of transgressive marine origin with a few brackish water deposits.

It underlies Sandia and is considered Mio-Pliocene in age. In the Tesla quadrangle, the lower member is up to 700 ft thick, and the upper member is up to 2,000 ft thick. Subsurface data indicate the undivided Meroly is up to 1,800 ft thick below Sandia. Depositional contacts are unconformable.

Earlier works have shown an arbitrary stratigraphic division along the trends of the Corral Hollow fault and/or Ramp Thrust. To the east the beds have been called Meroly, and to the west they have been called Orinda. The Meroly and Orinda are actually contemporaneous facies: the Orinda is essentially a nonmarine deposit with some marine beds, and the Meroly is essentially marine with a few brackish water deposits. Both formations are Mio-Pliocene age (Delmontian Stage of Kleinpell, 1938). Because the type area for the Orinda is in the Berkeley Hills, and because the beds in this area are not known to be nonmarine, the name Meroly seems most appropriate.

Livermore Gravels. The continental Livermore gravels make up the low hills to the south and east of Sandia. The formation dips beneath recent alluvium of Livermore Valley and overlaps older rocks. It is composed chiefly of Franciscan detritus with rounded clasts mixed with pebbles, sands, silts, and clays in various proportions. Deposition took place on alluvial fans and debris cones, with intermixed deposits ranging from thick debris flows to thin fluvial mud laminae. Bedding ranges from massive to subparallel, with tabular clasts and pebbles showing local imbricate structure. Cut-and-fill structures cause abrupt bed terminations. The gravels are generally low dipping, but they are locally steeply tilted in the ridge south of Sandia. In the vicinity of Sandia, subsurface data indicate the Livermore to be about 1,200 ft thick. Recent studies (Sarna-Wojcicki, 1976) show that a tuff bed near the base of the formation is 4-1/2 million years old. This age agrees with the Plio-Pleistocene estimate of Huey (1948).

Geologic Structure

Folding. Two folds are relevant to the Sandia site. They are the Livermore syncline, shown on the geologic map (Figure 3), and a Franciscan basement nose adjacent to and southwest of the syncline. Both structures are oriented and plunge in a northwesterly direction. The syncline contains Mesozoic, lower and upper Tertiary, and Quaternary beds, while only upper Tertiary and Quaternary beds overlie the Franciscan nose. These structures have been

delineated by drilling data, unpublished oil company geophysical data, and a USGS gravity map (Holden, 1976).

A less prominent but possibly important structure is exposed on the hill south of the Tritium Building where northwesterly dipping beds of the Livermore gravels indicate that the north slope of the hill is probably the flank of a southwesterly trending anticline.

Faulting. There is a clear relationship between the northwesterly folding and faulting in the area. The Livermore syncline is bordered by the Carnegie fault on the northeast and the Tesla fault on the southwest. These faults have strong thrust components, southwest on the Carnegie and northeast on the Tesla, reflecting the compression associated with folding. The Franciscan nose is bordered by the Tesla fault on the northeast and the Mocho fault on the southwest. The Mocho fault trends in a northerly direction toward the Tesla fault and is upthrown to the east. Its description is based on oil company subsurface geology and seismic data (Huey, 1972).

Two possible faults are pertinent to Sandia: Tesla and Las Positas. The Tesla fault trends northwest-southeast and is the most strongly documented fault near Sandia. In this area, it is composed of two strands, with Tesla No. 2 the most critical and recently active. Tesla No. 2 has been located by deep drilling, geomorphic and geophysical evidence, and by apparently related surface rupture. Deep drilling and magnetic data define Tesla No. 1, which is apparently inactive.

Tesla No. 2 Fault. Geologic information shows that Tesla No. 2 dips to the southwest (Figure 4) and has lateral and reverse components, with the southwest side up, in the southeast Livermore Valley. Less than a mile southeast of Sandia, Tesla No. 2 was found in Hancock Oil Company's Signal-Hanson No. 41 well. It cuts the well at 2,360 ft and causes a repeat of approximately 380 ft of Neroly Formation, demonstrating that there is a reverse component to the fault at this location (Figure 4).

Relatively recent movement on Tesla No. 2 is seen in the area midway between Sandia and the Hancock well. On Greenville Road is a small surface fault in Livermore gravels. This rupture falls on the trend of Tesla No. 2, strikes

FIGURE 4 CROSS SECTION A-A' SHOWING SUBSURFACE GEOLOGY AND DEEP DRILL HOLES

N37°E, has a vertical dip, and apparently fits the model for an antithetic fault associated with a right-lateral wrench fault. Relatively recent right-lateral wrenching of Tesla No. 2 is also apparent in spurs where Arroyo Seco cuts through the ridge immediately south of Sandia. West-trending spurs, adjacent to and northeast of Arroyo Seco, turn abruptly to the northwest. These features are consistent with drag associated with right-lateral faulting.

On June 21, 1977, an earthquake of magnitude 4.6 occurred in the southeast Livermore area. The epicenter was plotted on the Tesla fault, and seismologic data show both strike and dip components consistent with right-lateral motion and relative dip movement up on the southwest.

The 1972 Blume report presented geomorphic evidence showing that Tesla No. 2 had captured the drainage of Arroyo Seco and directed it to the northwest, through the ridge at the south border of Sandia. This showed relatively recent movement on the fault and indicated the fault to be up on the west. The report also presented evidence that Tesla No. 2 may have displaced soil layers in a Pacific Gas & Electric (PG&E) pipeline trench and was identifiable by discontinuities on gravity profiles.

Approximately 1 mi northwest of the Sandia site, a reflection seismic line was obtained from Hershey Oil Company. This line shows a reversal of dip across the projected trend of Tesla No. 2 that is consistent with fault data to the southeast.

The Las Positas Fault. The Las Positas fault was first named by Herd in a USFS open file report (Herd, 1977). Many lines of evidence were cited, but the Las Positas and other faults mapped are based essentially on air photo lineaments. A lineament trending southwest-northeast at the base of the north slope of the ridge south of Sandia was designated the Las Positas fault. This lineament had been recognized earlier (Blume, 1972), but evidence did not warrant its designation as a fault.

The fault is described by Herd as dip slip, down to the north. A cross section shows Las Positas as a vertical fault with minor displacement, down to the north, of a contact between the Livermore gravels and undivided Greenville and Tassajara Formations. (The Greenville and Tassajara Formations crop out north of Livermore Valley and are roughly equivalent to the Livermore gravels.)

Several types of evidence are presented in the USGS report. They include alluvial stratigraphy, geomorphology, and a postulated groundwater barrier.

Field Evaluation of Faulting, Las Positas Trace. In January 1978, a survey of the surface geology in the Sandia area of the Livermore Valley was carried out. The purpose of the survey was to review the local geology with special consideration of structure in general and faulting in particular. Emphasis was placed on examination of field evidence cited by Herd (1977) relevant to the reported Las Positas fault.

Previous work has shown several northwest-trending faults southeast of the Sandia area. Along the flanks of the southwest-trending ridge bordering the southern part of the laboratory area, air photo lineaments were indicated as possible fault traces.

In 1977, an open file report covering the area was released by the USGS. This report, 77-689, showed the photo lineaments as definite faults. Further, it asserted that the easterly trending faults were the dominant structures in the area, a situation virtually unknown in California outside of the Transverse Ranges.

The fault bordering the northwest flank of the ridge was named Las Positas fault zone. The fault described along the southeast side of the ridge was not named.

The new fault evidence is preponderantly geomorphic. Only two exposures are described for the Las Positas fault and only one for the fault along the southeast flank of the southwest-trending ridge.

A great deal of the report is based on a stratigraphic breakdown of the Quaternary sediments. This stratigraphy is enlisted as evidence of faulting and sequences of fault movement. Unfortunately, criteria for recognizing and differentiating the stratigraphic units are not presented, making it impossible to evaluate them, or evidence based on them, in the field.

The first area examined was along Greenville Road, between East Avenue on the north, and Tesla Road on the south. Greenville Road transects the south-

west-trending ridge to the east of Sandia. Two of the reported fault exposures occur in road cuts along this road. A transverse along the road indicates that the southwest-trending ridge is an apparent anticline. Evidence is not absolute, as the core of the ridge is composed almost entirely of massive silty clayey sands and sandy, silty clays. However, vertical dips on the north flank progressively flatten in a southerly direction until they dip only 4° to 6° to the north. South of the massive sediments, the bedding dips steeply to the south. Imbricate sedimentary structures in a road cut indicates a north-to-south current flow during some of the Livermore gravel deposition.

An exposure of the Las Positas fault is given in the report at 80 m south of the intersection of East Avenue and Greenville Road, on the east side of the road. It is stated that Neroly sandstone, in a south block, is faulted against colluvium composed of Livermore gravels in a north-facing scarp. Investigation found the sediments of the south block to be olive-tan to olive-brown clayey sands and sandy clays with beds of very light gray to whitish marl. The sediments are soft and unconsolidated, appear to be fluvial and/or lacustrine, and not Neroly. At the north end of the road cut, there is a change to reddish soil. The nature of the contact is not clear, but it appears to be a disconformity or unconformity. There is no overt evidence of faulting. An old road or trail cut into the northwest-facing slope may have been confused for a fault scarp.

The southern, unnamed fault is reported as crossing Greenville Road approximately 0.5 km north of the Tesla Road intersection. This exposure is clear cut. A vertical, northwest-facing scarp strikes $N37^{\circ}E$ into the east side of a cut ($N65^{\circ}E$ in the report). Because the fault is entirely within Livermore gravels, and because of a small slide immediately adjacent to the scarp, it is not possible to determine the throw. This fault is described elsewhere in this report as a minor antithetic fault associated with a right lateral wrench fault (Tesla No. 2). A small gravity anomaly on this feature indicates that near-surface displacement has been downward on the north side.

The only other fault exposure reported in the area is located in the south bank of Arroyo Seco, near the northwest edge of the southwest-trending ridge.

Here, a small fault is exposed, dipping 87°NW. It cuts sands and gravelly sands presumed to be part of the Livermore gravels. The throw appears to be up on the northwest, making this a high-angle reverse fault. This is diametrically opposite to the type of fault and direction of throw shown on the report map. It is significant that the fault does not cut the overlying alluvium. A poor exposure at the base of the bank appears to be a small drag fold. It is located about 10 ft north of the shear and is concordant with the direction of throw. About 400 ft north of the fault zone, a gravel with imbricate structure occurs in alluvium. The indicated direction of flow is slightly west of south, directly opposite the present creek.

Other fault locations shown on the map accompanying the USGS report were visited. Evidence of faulting, slope changes, and diffuse contacts of dissimilar lithologies are not persuasive. The variations of slopes may well be scarps, but it is most probable they were caused by erosion or ground failure. The contacts appear to be erosional disconformities or unconformities.

Some of the data cited as evidence of the Las Positas fault, such as lineaments and a groundwater barrier, could be explained as a depositional contact between two highly contrasting lithologic units within the Livermore gravels. The Livermore gravels, near Sandia, strike parallel to the Las Positas lineament and dip to the northwest. Dense, gravelly beds, appearing to be very resistant, compose a dip slope on the hill south of Sandia. In some trenches, the strata in the north end away from the base of the hill are predominantly fine materials.

It appears that beds once overlying those on the hill slope were easily eroded fine sands and silts which would have lower density and distinctively different porosity and permeability than the gravelly beds.

Gravity data do not support a significant displacement of bedrock along the Las Positas lineament but do indicate a gravity trough along the north side of the lineament caused by a density contrast in the upper 200 ft of material.

A northeasterly striking depositional contact between coarse and fine sediments of the Livermore gravels could be responsible for the observed gravity anomaly, the lineament, and the groundwater barrier.

The fault mapping of the Sandia area in USGS open file report 77-689 (USGS, 1977) is too positive. The field evidence is much too weak to support it. Faulting does exist in the area, as shown at two locations, but the magnitude cannot be ascertained by the field evidence. Further, accurate dating of fault movement is not possible in the field as the two proved fault exposures are both confined to the Livermore gravels, limiting the most recent movement dating to Plio-Pleistocene. At this time, with the data available, it would seem best to continue to regard the location of the possible fault as postulated, based on the two known fault locations and the photo lineaments.

LINEAMENT STUDY

Several types of imagery were examined for visible evidence of possible active faulting near Sandia. Anomalous linear markings and discolorations that did not appear to be man-made (e.g., plowmarks, buried pipelines, or abandoned roads) or by known geologic structure were compiled in Figure 5. Evidence of this kind is suggestive but not conclusive of faulting and should be verified by other geotechnical methods.

Air photos were studied in stereo on a scale of 1:20,000 (flown in May 1966) and on 1:1,250 enlargement (flown in May 1976). Several sets of lineaments were observed. The most prominent of these were the Tesla No. 2, Tesla No. 3, and Las Positas lineaments. The Las Positas approximately parallels the PG&E gas line and possible old pipeline routes near the base of the gentle hillside in the southern part of the Sandia property. Distinct lineations were observed about 1-1/2 mi to the southwest of Sandia, trending approximately northeast. However, they do not clearly connect as a throughgoing feature. Several lineaments were observed trending east-northeast obliquely across Tesla Road south of Sandia. These closely follow old trails and the base of the hill.

Infrared (IR) scanner imagery lines were examined. These film strips (flown in 1972) included imagery of heat radiation at three wavelengths, flown both in the afternoon and at night and at altitudes of 500 and 4,000 ft above the ground surface.

Strong lineations on the nighttime IR imagery were observed corresponding to the general trend of the Las Positas about 2 mi southwest of Sandia. This lineament trends northeast bounding a dark area on its northwest side. Dark areas indicate cooler temperatures on infrared imagery, possibly due to higher soil moisture content. One possible interpretation of this lineament is a groundwater barrier with higher water levels on the northwest side toward the Livermore Valley. However, this explanation does not seem satisfactory because it appears that groundwater recharge would be from the south and water drawdown from the north; thus, the lowest water depth should be to the north.

FIGURE 5. LINEAMENT MAP, LAS POSITAS ALIGNMENT

The Tesla No. 2 and Tesla No. 3 were not observed on night IR. However they appeared on afternoon low-altitude IR imagery, and the Las Positas did not.

High-altitude U2 false-color infrared imagery at a scale of 1:120,000 (flown in 1971) showed a faint lineation roughly corresponding to the Tesla No. 3 and another unrelated trend 1 mi northwest of Sandia.

Side-looking radar imagery (SLAR) at a scale of 1:400,000 showed a strong lineament corresponding to the Las Positas trend several miles southwest of Sandia. Diffuse lineaments possibly on line with the Tesla No. 2 and Tesla No. 3 were observed to the southeast.

Lineament data is summarized in Table 1. This comparison shows that the Las Positas is the most prominent of the linears that intersect Sandia. The most distinct portions of this trend lie more than a mile to the southwest, but do not clearly connect with the Las Positas lineament identified at Sandia. Several types of photography indicate a Tesla No. 3 lineament. Evidence for the Tesla No. 2 lineament is weaker but was confirmed on IR imagery.

TABLE 1
COMPARISON OF LINEAMENTS NEAR SANDIA

Imagery	Approximate Scale	Tesla No. 2	Tesla No. 3	Las Positas	
				North	South
Air Photo	1:1,250	+	+	+	+
Stereo Photos	1:20,000	0	+	+	+
IR (afternoon)	1:10,000 to 45,000	+	+	0	0
IR (night)	1:40,000	0	-	++	0
IR-Stereo (false color)	1:120,000	-	-	-	-
SLAR	1:400,000	0	0	-	+

Key:

++ strong lineament

+ lineament observed

0 diffuse lineament or not consistently locatable

- no lineament observed

EXPLORATORY TRENCHING

General Statement

A program of exploratory trenching was undertaken on January 25, 1978, at Sandia. The purpose of trenching was to expose and evaluate subsurface evidence of faulting along the trace of the reported Las Positas fault. One trench was also excavated on the probable location of the Tesla fault. The trench locations shown in Figure 6 were selected to intersect lineaments and features distinguished by examination of air photos and imagery, from geophysical surveys conducted for this study, and from findings of a geotechnical investigation by Blume (1972) and the USGS (Herd, 1977).

The trenching revealed inconclusive evidence of faulting along the lineament identified as the Las Positas fault by Herd and did not reveal any evidence of faulting in the trench that was intended to transect the Tesla fault.

Field work consisted of visual examination of the soils and logging of trench walls on a scale of 5 ft/in. Logs of the soils observed in the trenches are compiled in Appendix A. Selected trench wall details are shown in Figures 7a through 7d. A complete color photographic record is available for inspection.

Trench excavation was accomplished with a Case 580-8 rubber-tired backhoe. This machine, which has a maximum depth reach of about 14 ft, was used to excavate eleven trenches totaling 1,047 ft in length and ranging up to 13-1/2 ft in depth. The dimensions of the trenches are listed in Table 2.

Elevation control was provided by a topographic survey conducted by Sandia during late January 1978.

General Description of Soils

The subsurface soils exposed in trench excavations were found to consist primarily of dense brown sandy gravel with fines of about 5% cobbles and boulders (GM). Sand and clay content varied so that smaller interfingering lenses of clayey gravel (GC) and gravelly sand (SW) were common. Distinctive

FIGURE 6 SITE PLAN SHOWING EXPLORATORY TRENCH LOCATIONS

FIGURE 7a PORTION OF LOG OF EXPLORATORY TRENCH T2
ACROSS LAS POSITAS TRACE

FIGURE 7b PORTION OF LOG OF EXPLORATORY TRENCH T3
ACROSS LAS POSITAS TRACE

LEGEND

- well-defined contact
- - - gradational contact
- diffuse contact

FIGURE 7c PORTION OF LOG OF EXPLORATORY TRENCH T6
ACROSS LAS POSITAS TRACE

FIGURE 7d PORTION OF LOG OF EXPLORATORY TRENCH T10
ACROSS LAS POSITAS TRACE

TABLE 2
EXPLORATORY TRENCH DIMENSIONS

Trench No.	Length (ft)	Average Depth (ft)	Date
T1	204	12	1/26/78
T2	171	12-1/2	1/27/78
T3	71	7-1/2	1/30/78
T4	104	12-1/2	1/31/78
T5	111	12-1/2	1/31/78
T6	115	12	3/29/78
T6a	35	*	--
T7	40	11-1/2	4/05/78
T8	37	11	3/30/78
T9	72	12-1/2	4/01/78
T10	87	12-1/2	4/05/78

*Trench caved before it could be secured for logging.

lenses of light yellow-brown cohesive fine sand (SH-SC) were also found in some of the trenches.

A soil layer varying from about 1 to 4 ft thick unconformably overlies these gravelly materials. The unconformity surface at the base of the soil horizon was found to be undulating and generally deeper and less distinct toward the north. At the northern ends of some of the trenches, the change with depth from soil to underlying gravel was gradational.

Exploratory Trenching, Tesla Fault

Trench T1 was excavated to explore the most probable location for the Tesla fault as determined in prior studies by URS/Blume and from the results of the gravity survey performed during this investigation. The gravity data, described elsewhere in this report, are compatible with a fault striking along Arroyo Seco Creek, along which the south side had moved upward. The trench was located so as to transect this gravity anomaly.

The Tesla fault was not exposed in this trench; however, a great deal of evidence, discussed elsewhere in this report, indicates that the fault is present in the subsurface.

Trench T1 was begun about 150 ft east of the Tritium Building and extended 204 ft in a northeasterly direction (see Figure 6). Horizontally stratified alluvial sediments were revealed, and there was no visible evidence indicating the presence of a fault.

Exploratory Trenching, Las Positas Alignment

The possible trace of the Las Positas fault was explored by excavating four trenches 300 to 400 ft southeast of the Tritium Building and oriented south-eastward across the projected alignment (see Figure 6). Trenches T2, T4, and T5 exposed suggestive but inconclusive evidence indicating the presence of a possible fault zone approximately paralleling the 675-ft elevation contour.

Trench T3 was excavated across a slight topographic depression. This trench was a short distance south of Trench T2 and revealed normally stratified alluvial sediments only.

A second series of trenches was excavated across the Las Positas lineament in the area about 400 ft due south of the Tritium Building. No evidence to suggest faulting was found in four of these trenches (T6, T7, T9, and T10). Some features suggestive of faulting were found in trench T9. However, these were inconclusive and less pronounced than the features observed in trenches T2, T4, and T5.

The lineament approximately coincides with the location of a buried PG&E gas main in the vicinity of trench T7 and it therefore could not be extended far enough to the south to be certain of intersecting the desired alignment.

Trench Features Indicative of Possible Faulting

Visible evidence suggestive but not conclusive of faulting was found in four of the eight trenches across the Las Positas alignment. The evidence is categorized below:

1. Silty fine sand strata were found to terminate abruptly against gravelly beds in three trenches -- T2, T4, and T5 (see trench logs).
2. Thin clayey lenses with near-vertical orientation were found in trench T9 in the vicinity of the lineament. Bedding planes in T9 were approximately horizontal.
3. Tabular gravel clasts were found with their long axes oriented approximately vertical near the possible fault zone, whereas elsewhere they were found lying essentially horizontal. Clusters of these vertical clasts were found in four trenches (T2, T4, T5, and T9).
4. Downward-curving contacts and faint discolorations oriented in such a way that the curvature could be explained as being due to drag on a fault were observed in two trenches (T4 and T5).
5. Small water seeps were found within the possible fault zone in trench T2. However, seepage unrelated to possible faulting was found in other trenches (e.g., T7).
6. There was a tendency for the trench walls to cave within the possible fault zone in three trenches (T2, T4, and T5). However, troublesome caving unrelated to disruption also affected trenches T6, T6a, T8, and T9, apparently due to a local lack of cohesive binder in the soils.

7. The topsoil layer was found to be notably thinner and more abundant in gravel and cobbles upslope of the possible fault alignment. Soil variations of this kind were found in trenches T2, T4, T5, and T9. Similar concentrations of cobbles occurred in trenches T6 and T10 with no apparent relationship to a possible fault.

Observations Arguing Against Significant Faulting

No conclusive evidence of a fault motion was found along the Las Positas alignment. Nowhere within the depths explored were there features normally associated with a fault plane, such as a fault gouge or slickensided material. Vertical clasts did not show signs of post-depositional rotation. Close inspection of the contacts bounding abruptly terminated beds revealed that the contacts were not sharply defined or sheared.

An apparently unbroken and conformable sequence of northward-dipping beds was found in four of the eight trenches across the lineament (T6, T7, T8, and T10). In these trenches, there was a fairly distinct contact between the topsoil and the underlying gravel beds. No offset was observed on this apparent unconformity surface.

These observations indicate that if the Las Positas fault exists, its trace is discontinuous at least in the near-surface soils. The anomalous features observed in some of the trenches suggest but do not require a fault explanation. Other processes could be called upon -- for example, basin subsidence or progressive folding and fracturing of the bedrock along an axis parallel to the valley margin.

Water Line Trench

A shallow trench was dug for placement of a water main along the entire western margin of the Sandia property. Most of the trench was 6 ft in depth and ranged up to 8 ft deep in some locations. The excavation between stations 700 and 3,400 was inspected during construction and photographed for later examination. No signs of faulting were observed.

The area crossing the Las Positas alignment was of particular interest. Near station 3,320, about 150 ft south of the PG&E gas line, the trench revealed

a light brown gravelly unit dipping gently northward beneath dark brown topsoil. This dipping contact was observable and continuous over a length of about 40 ft. No signs of faulting were observed.

Conclusions

Tesla Fault. No evidence of faulting was found within the depths explored; therefore, the hazard of surface fault rupture is considered to be very remote.

Las Positas Fault. Although positive evidence of a fault (such as a fault gouge, a zone of slickensided material, or a clear-cut line of separation) was not found within the depths explored, other features seen in trenches T2, T4, and T5 suggest the presence of a fault zone, and the potential hazard of surface fault rupture along the fault trace must be regarded as a real possibility.

However, the trend of this possible fault as determined by the exposures in trenches T2, T4, and T5 indicates that it lies some distance south of the Sandia Tritium Building. Empirical data relating total fault length and rupture length indicate that surface rupture on a fault such as the reported Las Positas, whose total length has been shown to be only 10 mi, is highly unlikely. However, if surface rupture should take place along this possible fault trace, it would not affect the Tritium Building.

The hazard due to vibratory motion associated with such an event is discussed elsewhere in this report (see pages I-95, I-98, I-99).

GRAVITY SURVEY

Field Procedures

A gravity survey was carried out near Sandia in the southeastern portion of Livermore Valley during two periods: November 12 to 16, 1977, and March 3 to 14, 1978. The purpose of this survey was to determine whether evidence of faulting could be discovered by means of variations of gravity along the Las Positas trace proposed by Herd (1977) as well as along projections of the Tesla fault traces in the vicinity of the Tritium Building (Blume, 1972). The second survey was carried out to augment data from the first, as well as to look for small-scale gravity anomalies along that part of the possible Las Positas fault in the vicinity of the Tritium Building and where it intersects Sandia's restricted area.

The first phase of the survey was composed of measurements taken on a 13-by-13-station grid, encompassing a $1/2\text{-mi}^2$ area centered on the Tritium Building, and four traverse lines along roads east and west of the Sandia property. Stations along the traverse lines were spaced at intervals of 50 ft. The second phase consisted of confirming some earlier measurements and taking gravity measurements along one traverse adjacent to a trench and four traverses within the Sandia security area. The traverses were intended to determine, if possible, the exact location of the possible fault by employing close station spacing of 10 to 20 ft and by comparing measurements with features revealed in the trenches. The locations of the road traverse lines and grid are shown in Figure 8, and the locations of the traverse lines internal to the Sandia property are shown in Figure 9.

The data were obtained using a La Coste Romberg gravimeter in both surveys. In the first survey (November 1977), the instrument drift rate was determined by using closure on three separate road lines (J, E, and D), and a computer program was used to remove tidal gravity variations due to the moon and sun.

In the second survey (March 1978), the lines were close sufficiently often to reference or base stations to allow for a correction involving both instrumental drift rate and tidal variations. In both surveys, the instrumental accuracy on hard surface (e.g., asphalt) was found to be ± 0.01 mgals, and ± 0.03 mgals on softer surfaces (e.g., soil and grassy surfaces) and in areas where there was a large noise component (wind, cars, etc.).

FIGURE 8 LOCATIONS OF GRAVITY SURVEYS, THE TESLA FAULT, & THE REPORTED LAS POSITAS FAULT

FIGURE 9 GRAVITY SURVEY LINES, SANDIA

Elevations of stations where measurements were made were determined to an accuracy of ± 0.01 ft for both surveys, using a Zeiss level and Philadelphia rod.

Data Reduction

Data reduction consisted of the following procedures:

1. Removal of instrumental drift rate and tidal variations.
2. Calculation of the free-air and Bouguer plate corrections to obtain the simple Bouguer anomaly.
3. Removal of the best-fitting linear regional trend from the data (by a least-squares computer program) to obtain the residual anomaly.
4. Graphical portrayal of the residual anomalies and station elevations to determine the importance of terrain effects, as well as to identify and describe any significant gravity anomalies.
5. Graphical portrayal and inspection of the residual anomaly data to determine whether any correlation exists with locations of possible fault traces.

Description of Data

Following is a description of residual anomalies and their locations for each of the traverse lines and for the Sandia grid. Residual anomaly plots are shown in Figures 10a through 10j. The layout of the gravity grid and the anomaly patterns observed are shown in Figures 11 and 12.

1. J-Line: length = 1,600 ft; station spacing = 50 ft; rms deviation of residuals = 0.061 mgals.

Observations: Bouguer gravity shows a decrease to the north of approximately 1.05 mgals, with a modest downward curvature. The best-fitting linear gradient was found to be approximately 6.8×10^{-3} mgals/ft. The residual anomaly plot shows no significant anomaly patterns.

2. E-Line: length = 2,400 ft; station spacing = 50 ft; rms deviation of residuals = 0.068 mgals.

Observations: Bouguer gravity shows a decrease to the north of approximately 2.2 mgals, with a slight downward curvature. The best-fitting linear gradient was found to be approximately 1.02×10^{-3} mgals/

FIGURE 10a PROFILE OF RESIDUAL GRAVITY ALONG J-LINE

FIGURE 10b PROFILE OF RESIDUAL GRAVITY ALONG E-LINE

FIGURE 10c PROFILE OF RESIDUAL GRAVITY ALONG SOUTH D-LINE

8C-1

FIGURE 10d PROFILE OF RESIDUAL GRAVITY ALONG NORTH D-LINE

FIGURE 10e PROFILE OF RESIDUAL GRAVITY ALONG K-LINE

FIGURE 10f PROFILE OF RESIDUAL GRAVITY ALONG L-LINE

FIGURE 10g PROFILE OF RESIDUAL GRAVITY ALONG M-LINE

FIGURE 10h PROFILE OF RESIDUAL GRAVITY ALONG N-LINE

FIGURE 101 PROFILE OF RESIDUAL GRAVITY ALONG 0-LINE

FIGURE 10j PROFILE OF RESIDUAL GRAVITY ALONG P-LINE

FIGURE 11 GRAVITY SURVEY GRID, SANDIA

Contour Interval .05 mgals

rms Deviation of
Residuals = 0.13 mgals

SCALE(ft)

FIGURE 12 SANDIA GRID, CONTOUR MAP OF RESIDUAL GRAVITY ANOMALY

ft. There is a significant anomaly located approximately 1,100 ft north of Tesla Road on Vasco Road. The anomaly has a maximum variation of approximately 0.20 mgals and a wavelength of approximately 200 ft and indicates a substantial decrease in g towards the north; it appears to be the only significant anomaly on this line.

3. South D-Line: length = 1,900 ft; station spacing = 50 ft; rms deviation of residuals = 0.141 mgals.

Observations: Bouguer gravity shows a decrease to the north of approximately 2.65 mgals, with a slight downward curvature. The best-fitting linear gradient was found to be approximately 1.68×10^{-3} mgals/ft. There is a significant anomaly located approximately 550 ft north of the crossing of Arroyo Seco and Greenville Road. The anomaly has a maximum variation of approximately 0.38 mgals and a wavelength of approximately 250 ft and indicates a moderate decrease in g toward the north. The minimum in g at Arroyo Seco is most likely a terrain effect associated with the creek, as the measurement was taken on the highway bridge, approximately 12 ft above the creek bed. The anomaly described above appears to be the only significant anomaly on this portion of the D-Line.

4. North D-Line: length = 1,200 ft; station spacing = 50 ft; rms deviation of residuals = 0.044 mgals.

Observations: Bouguer gravity shows a decrease to the north of approximately 1.2 mgals, with little curvature. The best-fitting linear gradient was found to be approximately 1.02×10^{-3} mgals/ft. There is a moderately large anomaly, in the form of a minimum in g, located approximately 350 ft south of East Avenue on Greenville Road. The anomaly has a maximum variation of approximately 0.06 mgals and a wavelength of approximately 250 ft and indicates a minimum in g, as noted above. This anomaly appears to be the only significant anomaly on this portion of the D-Line.

5. K-Line: length = 1,200 ft; station spacing = 50 ft; rms deviation of residuals = 0.085 mgals.

Observations: Bouguer gravity shows a nonuniform decrease to the north of approximately 1.3 mgals, with little curvature. The best-fitting linear gradient was found to be approximately 1.34×10^{-3} mgals/ft. There are no significant anomaly patterns that have amplitudes differing significantly from the overall variation in g along this line. The modest gravity minimum in the vicinity of Arroyo

Seco is most likely a terrain effect associated with the depressed stream bed.

6. L-Line: length = 400 ft; station spacing = 10 ft; rms deviation of residuals = 0.031 mgals.

Observations: Bouguer gravity shows a uniform decrease to the north of approximately 0.40 mgals, with little evidence of curvature. The best-fitting linear gradient was found to be approximately 1.0×10^{-3} mgals/ft. There is a small anomaly, in the form of a minimum in g, located approximately 75 ft north of the PG&E gas pipeline. The anomaly has a maximum variation of approximately 0.14 mgals and a wavelength of approximately 60 ft. There are no other significant anomalies on this line.

7. M-Line: length = 340 ft; station spacing = 10 ft; rms deviation of residuals = 0.019 mgals.

Observations: Bouguer gravity shows a systematic decrease to the north of approximately 0.29 mgals. The best-fitting linear gradient was found to be approximately 8×10^{-4} mgals/ft. There are no significant anomalies on this line.

8. N-Line: length = 380 ft; station spacing = 10 ft; rms deviation of residuals = 0.023 mgals.

Observations: Bouguer gravity shows a systematic decrease to the north of approximately 0.39 mgals. The best-fitting linear gradient was found to be approximately 1.1×10^{-3} mgals/ft. There appears to be a small anomaly, in the form of a maximum in g, located near the midpoint of this line; however, there appears to be a modest downward curvature in the residual anomaly data, so that the departure of the anomalous maximum from the mean may not be significant. There are no other significant anomalies on this line.

9. O-Line: length = 600 ft; station spacing = 10 ft; rms deviation of residuals = 0.029 mgals.

Observations: Bouguer gravity shows a fairly uniform decrease to the north of approximately 0.67 mgals. The best-fitting linear gradient was found to be approximately 1.2×10^{-3} mgals/ft. There are no obvious anomaly patterns along this line, although there may be some indication of a feature approximately 80 ft north of the PG&E gas pipeline.

10. P-Line: length = 300 ft; station spacing = 10 ft; rms deviation of residuals = 0.047 mgals.

Observations: Bouguer gravity shows a fairly uniform decrease to the north of approximately 0.41 mgals. The best-fitting linear gradient was found to be approximately 1.4×10^{-3} mgals/ft. There

appears to be an anomalous step-increase in g towards the south, approximately 120 ft south of the beginning of this line, with maximum variation and wavelength of approximately 0.09 mgals and 40 ft, respectively. The location of this anomaly appears to coincide closely with a vertical contact between sand and denser gravels found in trench T2 (see discussion on exploratory trenching in previous chapter). There are no other significant anomalies on this line.

11. Sandia Grid: dimensions = 2,400 ft x 2,400 ft; station spacing = 200 ft; rms deviation of residuals = 0.130 mgals.

Observations: Bouguer gravity shows a general decrease from southeast to northwest across the grid, with some downward curvature in both the southeast-northwest and southwest-northeast directions. The best-fitting linear gradient was found to be approximately 8.8×10^{-4} mgals/ft (north-south) and 6.18×10^{-4} mgals/ft (east-west). The residual anomaly map (Figure 12) shows several distinct features:

- a. There are local gravity "highs" in the northwest and southeast portions of the grid area, and gravity "lows" in the southwest and northeast portions, reflecting the fact that the gravity surface possesses curvature of second, and possibly higher, order. The decrease in g to the northeast may be a reflection of the "Livermore syncline" which is located to the northeast of the grid area, with an axis striking west-northwest. The decrease could also be related to possible density contrasts caused by the Tesla fault, the major strand of which approximately coincides with Arroyo Seco in this area. The significance of the + and - residuals is due partly to the manner in which the best-fitting plane was found, and partly to real lateral density inhomogeneities.
- b. The most prominent feature on the map is an east-northeast trending linear feature running through the southern portion of the grid area. This feature is characterized by an abrupt decrease in gravity from southeast to northwest of approximately 0.10 mgals over a horizontal distance of roughly 200 ft. There is a strong correlation between the gravity contours associated with this feature and the topographic

- contour lines associated with the range of low hills bordering the southern portion of the Sandia property.
- c. The linear feature described in (b) terminates abruptly at its northeast end, being intersected by a north-northwest striking feature that is characterized by gravity decreasing from southwest to northeast. Once again, the southeast-trending gravity contour lines closely parallel the topography of the eastern part of the Sandia grid area.
 - d. The linear feature described in (b) strikes almost due south of its southwest end at the southwest corner of the grid area. This trend does not correlate strongly with local topography, which strikes to the southwest beyond the grid area.
 - e. There is a general gravity low, in the rough shape of a trough, which trends northeast, just north of the linear feature described in (b).

Regional Gravity Model

Regional variations of gravity are shown on the USGS open-file Bouguer anomaly map of the Livermore Valley area (Holden, 1976), which is reproduced in part in Figure 13. Mean gravity gradients were calculated from this map and compared with gradients determined from the URS/Blume survey for corresponding locations. The results of this comparison are shown in Table 3. Examination of this table shows that the mean gradients determined from the URS/Blume survey by means of linear least-squares fitting to the data are within approximately 10% to 15% of the corresponding gradients obtained from the USGS map except for the K-Line, in which the discrepancy is ~30%. The overall southeast-northwest linear regional gradient determined from both sources lies between 5×10^{-4} mgals/ft and 8×10^{-4} mgals/ft.

An attempt was made to account for the overall regional trend in the gravity by means of simple density contrast models. The first model along with the computed gravity associated with it is shown in Figures 14a and 14b. The model assumes a basement of Franciscan rock dipping to the northwest at 20° . The basement is assumed to outcrop approximately 4 km southeast of

Reproduced from U.S.
Geological Survey Open-
File Map 76-761
(Holden, 1976)

Contour interval 2 milligals, dashed where uncertain

Reduction density 2.67 g/cm^3

Las Positas trace (Uard, 1977)

FIGURE 13 COMPLETE BOUGUER ANOMALY MAP OF THE LIVERMORE VALLEY AREA

TABLE 3
DATA FROM URS/P'UME GRAVITY SURVEY (NOVEMBER 1977, MARCH 1978)

	Mean Gravity Gradients	Data Taken from USGS Bouguer Anomaly Map (1976)
E-Line	1.02×10^{-3} mgals/ft	$\approx 1 \times 10^{-3}$ mgals/ft
South D-Line	1.68×10^{-3} mgals/ft	$\approx 1.9 \times 10^{-3}$ mgals/ft
North D-Line	1.02×10^{-3} mgals/ft	$\approx 1 \times 10^{-3}$ mgals/ft
K-Line	1.34×10^{-3} mgals/ft	$\approx 1.8 \times 10^{-3}$ mgals/ft

FIGURE 14a DIPPING BASEMENT MODEL FOR SOUTHEAST LIVERMORE VALLEY

FIGURE 14b DIPPING BASEMENT MODEL FOR THE SOUTHEAST LIVERMORE VALLEY

Sandia, and a density contrast of 0.20 gm/cm^3 is assumed between the basement rock and the overlying gravels and alluvium. The dip angle was determined from Blume (1972), and the density contrast was determined from well data in the area between the Franciscan outcrop and the Sandia property. The mean gradient determined from the graph in Figure 14b at a position 4 km to the northwest of the outcrop ($x = 0$) is approximately $7 \times 10^{-4} \text{ mgal/ft}$.

Thus, the regional gradient near the Blume survey can be reasonably well explained in terms of a simple dipping basement, as described in Figure 14a.

Fault Models

Data from the 1972 Blume report also indicated the possibility of a reverse fault (see plate 4, Appendix A) located along the reported Las Positas fault trace. This possibility stemmed from the observation that an exploratory well called the Pestana well, located north of the trace, showed the basement rock to be nearly 1,000 ft (1/3 km) closer to the surface than the corresponding basement horizon determined from wells approximately 1-1/2 mi to the south of the trace.

A model was assumed, consisting of a vertical offset in the basement rock (with the north side about 1,000 ft (1/3 km) higher than the south side), once again in the presence of a dipping basement. The model and calculated gravity are shown in Figures 15a and 15b. The calculations imply that the regional gravity in the vicinity of the Las Positas trace would be increasing to the north, exactly the opposite of what is actually observed. Thus, with the structure assumed from the well data, along with the density contrast quoted above, a reverse fault below the Las Positas lineament seems extremely unlikely. Further examination of the well data indicates that identification of features in the Pestana well was based strictly upon cuttings brought to the surface. Our review of the well log indicates that identification of Franciscan bedrock near the bottom of the hole is questionable at best and probably erroneous.

FIGURE 15a REVERSE FAULT MODEL FOR SOUTHEAST LIVERMORE VALLEY

FIGURE 15b CALCULATED GRAVITY FOR REVERSE FAULT MODEL

An analysis was made to consider whether or not a normal fault might exist beneath the Las Positas lineament in the Franciscan basement assemblage. For this model, a 1.0-km offset was assumed to have displaced the bedrock surface downward to the north. Again, a bedrock surface dipping 20° northward and a 0.20-gm/cm³ density contrast were used. As shown in Figures 15c and 15d, for a depth of 1.46 km (4790 ft) to an assumed offset of 1.0 km (3281 ft), the mean gravity slope above the fault is changed from 6.84×10^{-4} mgal/ft for the unfaulted surface to a value of 7.98×10^{-4} mgal/ft for the faulted surface.

The measured gravity data are compatible with the model for an unfaulted bedrock surface. However, because of probable irregularities in the bedrock surface as well as uncertainty in its average slope and density contrast, the possibility of bedrock displacement cannot be ruled out on the basis of this particular model.

Near-Surface Anomalies

The anomalies that were located in the course of the Blume survey, with their smaller amplitudes (tenths of a milligal) and wavelengths (hundreds of feet) would generally reflect variations in structure and density considerably closer to the surface than the depths assumed for the basement rock. In the case of an active fault, offsets will exist in the medium above the basement (rocks of Tertiary age) as well as in the basement itself. Density contrasts within and between the tertiary formations older than the Livermore gravels appear to be minimal, and the likelihood that they would generate large gravity anomalies at the surface is low. Thus, it is hypothesized that the anomalies located are representative of near-surface density variations within 200 ft.

The large anomaly on the E-Line together with the smaller anomaly on the north D-Line both fall within 50 ft of the Las Positas trace proposed by Herd (1977). In addition, the linear feature running through the Sandia grid coincides closely with the reported Las Positas fault trace.

The three separate gravity anomalies along the reported Las Positas fault trace, on the E-Line, the D-Line, and across the grid suggest that there

FIGURE 15c NORMAL OFFSET MODEL

FIGURE 15d NORMAL FAULT MODEL FOR VERTICALLY DIPPING NORMAL FAULT AT LAS POSITAS TRACE

may be a more or less continuous anomaly along the trace, interrupted locally where crossing Arroyo Seco and perhaps elsewhere. This line also coincides with a break in slope where the alluvial floor of Livermore Valley meets hills rising to the south. Even on the E-Line, where the topography is very subdued, a break in slope is evident at the location of the anomaly.

The geologic structure of the area also parallels the northeast trends. Geologic mapping indicates that the Livermore gravels making up the hill to the south strike parallel to the hill and to the gravity anomaly. Beds along the north side of the hill have a dip slope, and the hill comprises the north flank of an anticline. The area within a few hundred feet of the base of the hills would probably be underlain by north-dipping strata of the Livermore gravels, which have been eroded off and covered by a thin blanket of alluvium. Because these strata have been eroded away leaving resistant gravels on the hill in place, it appears that there is a contrast in resistance to erosion. This contrast is also probably accompanied by contrasting densities.

Evidence supporting this hypothesis is seen in some of the trenches in which northwesterly dipping strata consist of more gravelly materials at the south end of the trench (e.g., Trench T10) and strata at the north end are predominantly fine grained. This feature cannot be observed in trenches exhibiting sub-horizontal stratification. There is clearly a significant density contrast between strata of the Livermore gravels, which range from dense sandy gravel to fine sandy silt.

Depositional contacts between such contrasting beds coinciding in strike and location with the gravity anomaly could have produced the observed anomaly. This model -- Figure 16 (a and b) -- is compatible with the amplitude and wavelength of the observed anomaly.

The large anomaly on the south D-Line falls close to both the Tesla No. 2 (Blume, 1972) and an unnamed southern fault proposed by Herd (1977). In the absence of additional evidence, the significance of this anomaly cannot be addressed. However, the decrease in g from south to north is consistent with the sense of motion that has been assumed for the Tesla No. 2 fault but

a. Theoretical Gravity Profile

b. Model

FIGURE 16 GRAVITY PROFILE FOR ANTICLINAL STRUCTURE

is inconsistent with movement downward to the south as suggested by Herd (1977).

While the anomaly located on the P-Line did coincide with the contact found in Trench T2, the absence of any significant anomalies along the Las Positas trace on lines L through O would seem to imply that if there is near-surface faulting in the Sandia restricted area, it is discontinuous.

Conclusion

The results of the recent gravity survey carried out by URS/Blume coincide quite closely on a large scale with the regional picture supplied by the USGS Bouguer anomaly map of the area. The mean regional gradients from both sources agree to within 10% to 15%. These regional gradients can be adequately explained in terms of a basement rock of the Franciscan assemblage dipping at approximately 20° to the northwest beneath younger sedimentary rocks and alluvium.

The USGS Bouguer anomaly map implies that there is a high-density region extending northwest into the southeastern corner of Livermore Valley. Such an interpretation is consistent with the existence of a northwest-plunging anticlinal structure, in which the Franciscan basement has been uplifted in this region through the action of northwest-striking folding and possibly faulting.

The Las Positas trace cuts across the Bouguer anomaly contours (see Figure 13) throughout most of the segments designated A and B. Along Segment A, the trace passes through a region in which the gravity gradient is very small, so that a normal offset in basement rock would have to be quite small to be consistent with the regional gravity slope in this area. At the southwest and northeast ends of Segment B, the Las Positas trace cuts sharply across the gravity contour lines. Such behavior is not consistent with a single normal fault below the Las Positas trace through Segments A and B.

In addition, examination of the gravity contour lines associated with the Calaveras and Greenville faults at the west and east sides of the Livermore Valley, respectively, indicates a much more consistent correlation between

the strike of these faults and associated gravity contour lines than is apparent in the vicinity of the Las Positas trace.

On the basis of both the URS/Blume and USGS gravity surveys, it is our opinion that the most straightforward, consistent view of the gravity data in the southern Livermore Valley does not include faulting along the Las Positas trace as a likely feature.

There is no conclusive evidence from these gravity surveys for the existence of either the Tesla No. 2 or Tesla No. 3 fault traces in the immediate vicinity of the Sandia restricted area. The decrease in gravity in the northeast portion of the grid and the anomaly located on the south D-Line could belong to the Tesla No. 2 trace, but in the absence of other anomalous features, no definite conclusions can be drawn on the basis of the gravity survey alone.

MAGNETIC AND RESISTIVITY SURVEYS

General Statement

Magnetometer and electrical resistivity measurements were made along traverses located near Sandia (see Figure 17) as part of the investigation of faulting.

The traverses were selected to cross possible northwesterly trending fault traces as well as the reported Las Positas fault trace.

Field Procedures

In the case of the magnetometer surveys, both the total field strength and vertical gradient of the Earth's magnetic field were recorded at each station, using a Geometrics model G-816 proton-precession type magnetometer. The vertical gradient was obtained by taking two readings at each station, one with the sensing head 5 ft above the ground and the other with the sensing head 12 ft above the ground. The difference between these two readings was then divided by the appropriate scale factor to obtain the vertical gradient. The vertical gradient was used as a discriminant against shallow magnetic sources (buried pipes, etc.). All readings were taken at 50-ft intervals. All lines were sufficiently short that diurnal variations in the main field were not taken into account.

The electrical resistivity surveys were carried out using a Bison model 2350-B resistivity meter. Readings were taken at intervals of 75 ft using a fixed electrode spacing of 75 ft. This technique results in a continuous profile of ground conductivity at a depth of the same order as the electrode spacing. The purpose of this survey was to detect significant vertical offsets in the groundwater level as a possible indicator of impermeable barriers associated with near-surface faulting.

The locations of both the magnetic and resistivity survey lines are shown in Figure 17.

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The locations of both the magnetic and resistivity survey lines are shown in Figure 17.

FIGURE 17 LOCATIONS OF MAGNETIC TRAVERSES AND RESISTIVITY SURVEYS

Results

Plots of magnetic and resistivity data are given in Figures 18a through 18e and 19a through 19c, respectively.

In general, the area in which the surveys were conducted is contaminated by the presence of many sources of magnetic noise, specifically pipelines, power lines, structures, debris, fencing, etc. In particular, the Las Positas lineament is locally nearly coincident with the PG&E high-pressure gas pipeline. This pipeline caused magnetic interference at distances up to approximately 300 ft. Magnetic profiles on relatively quiet traverses showed no evidence of anomalies significantly larger than normally expected background fluctuations. In conclusion, there is no strong evidence of faulting as determined from the magnetic surveys.

The resistivity surveys failed to show any anomalies along suspected traces of the Tesla fault. Anomalous features aligned with the Las Positas lineament (see Figure 19b) failed to show any consistent correlation between adjacent traverse lines. The results of the resistivity surveys must be considered inconclusive with respect to fault location.

FIGURE 18a MAGNETOMETER SURVEY, LINE M1

FIGURE 18b MAGNETOMETER SURVEY, LINES M2-M5

FIGURE 18c MAGNETOMETER SURVEY, LINES M6, M10, M11

FIGURE 18d MAGNETOMETER SURVEY, LINES M7-M9

FIGURE 18e MAGNETOMETER SURVEY, LINES ALONG GRAVITY GRID LINES

FIGURE 19a RESISTIVITY SURVEY, LINES R1-R4

FIGURE 19b RESISTIVITY SURVEY, LINES R8-R10

FIGURE 19c RESISTIVITY SURVEY, LINES R5, R6

SEISMOLOGY

Seismic History

The coastal portion of central California is one of the most seismically active regions of the United States. Recorded seismic history goes back to 1800. The important earthquakes that have been documented since that time begin with observations recorded by Spanish missionaries. Some crude instrumentation was introduced into the area in the late 1800s at Chabot Observatory and Mt. Hamilton; however, before the 1906 San Francisco earthquake, the sensitivity and location of these seismographs were of little value in locating the smaller earthquakes in the area. Instrumentation for accurately recording and measuring large earthquakes has been available since the early 1930s. Sufficient instrumental coverage for locating and measuring small earthquakes has been available since the early 1940s and continues to be improved up to the present time.

Earthquake epicenters for the coastal central California area are plotted on a map (Figure 20) showing the major active faults in the area. It is important to remember that many of the older earthquake locations are based on, or influenced by, historical personal accounts. Population density influenced the epicenter locations toward the centers of population, thus making many of the estimated epicenter locations inexact.

Seismic History Related to Active Faulting. The historic earthquake record indicates that the San Andreas fault was the probable source of destructive earthquakes in June 1838; October 8, 1865; April 24, 1890; April 18, 1906; and, most recently, March 22, 1957. The April 18, 1906, earthquake, perhaps the most thoroughly studied seismic event until the recent 1971 San Fernando earthquake, had a Richter magnitude of 8.25. Visible surface faulting extended for more than 200 mi from San Juan Bautista on the south to Point Arena on the north, where the fault enters the ocean. Surface faulting was noted at Shelter Cove, 75 mi north of Point Arena along what is probably an extension of the San Andreas fault. A total surface rupture in excess of 270 mi could therefore have resulted from this earthquake. Maximum horizontal movement was 21 ft in the vicinity of Olema, some 26 mi northwest of

FIGURE 20 EARTHQUAKE EPICENTER AND MAJOR ACTIVE FAULT MAP FOR THE COASTAL CENTRAL CALIFORNIA AREA

San Francisco. North of the Golden Gate, the horizontal movement was accompanied by 3 ft of vertical displacement, and as much as 6 ft of vertical displacement took place at Shelter Cove further to the north. Both the horizontal and vertical movement decreased in magnitude to the south toward San Juan Bautista. Damage from this earthquake has been estimated at between \$350 million and \$1 billion. About 700 persons were killed. Most of the loss was due to fires in San Francisco. This earthquake resulted in an estimated intensity of between VII and VIII in the Livermore Valley. However, few structures existed at the time and the intensity estimates are probably inexact.

Destructive earthquakes occurred on the Hayward fault on June 10, 1836, and October 21, 1868. Approximately 40 mi of surface rupture between Mission San Jose and San Pablo accompanied the 1836 earthquake. Surface rupture along a 20-mi length accompanied the 1868 earthquake between Warm Springs and San Leandro. Right-lateral movement accompanied both earthquakes, with 3 ft of offset associated with the 1868 earthquake and an unknown amount associated with the 1836 earthquake. No reliable estimates are available on the dollar value of damage from the earthquakes; however, almost every building was destroyed or damaged along the areas of surface rupture, and approximately 30 deaths were attributed to the 1868 earthquake. Both of these earthquakes are estimated to have had a magnitude of somewhere between 7 and 7.5.

Destructive earthquakes on the Calaveras fault have not been well documented, if indeed any at all have occurred. The Calaveras fault has been considered active for many years because of small earthquakes that are locally felt in the Livermore Valley-Amador Valley area. Possible destructive earthquakes associated with the Calaveras fault occurred on July 3, 1861, and appear to have been most severe in the San Ramon Valley, which is situated on the mapped zone of the fault traces. One of these events is reported to have caused damage in the Amador and Livermore valleys, but no damage is reported from other nearby locations such as Hayward, San Leandro, or Mission San Jose. Epicentral intensity for this earthquake has been estimated at VIII and possibly might have been as high as IX. Ground surface displacement is reported to have occurred. The magnitude for this earthquake could be as

low as 6 because of the apparent local area of damage or as high as 7.5 if the most conservative interpretation is placed on the sketchy observations of this event.

Another large shock that might have been associated with the Calaveras fault occurred on March 30, 1898. Damage was greatest at Mare Island, which, if this is assumed to be the approximate position of the epicenter, could be on the northward projection of the Calaveras fault across the Carquinez Straits.

Closer to Sandia in the Livermore Valley area are several epicenters of microearthquakes. Historically, there have been no earthquakes larger than 4.6 magnitude recorded in the Livermore Valley area, and most of these microearthquakes are less than magnitude 3.5. Although nothing indicates that these small events have occurred on faults, they tend to substantiate conclusions based on other evidence that some of the faults near Sandia are active.

For engineering purposes, the following table summarizes the recommended design earthquakes to be considered for the Sandia site. The location of a possible earthquake on the Tesla fault is based primarily on geologic evidence and is discussed elsewhere in this report.

Zone	Minimum Epicentral Distance		Magnitude
	(mi)	(km)	
Tesla fault	2	3	5.7
San Andreas fault	35	56	8.3
Hayward fault	17	27	7.5
Calaveras fault	10	16	7.5

Livermore Valley Seismicity

Earthquake Relocation. The seismicity of the Livermore Valley was reviewed to select specific earthquakes of interest to redetermine the hypocentral parameters. Almost 40 events were relocated from the time period 1940 to 1977, which are listed in Table 4. The method of relocation employed the

TABLE 4
HYPOCENTRAL PARAMETERS OF SEISMIC EVENTS
IN THE LIVERMORE VALLEY, 1943 TO 1977

Event Number	Date	hr min sec	Lat(N)	Long(W)	Depth (km)	M _L
1	03/21/1943	11 45 45.6	37° 38.6'	121° 49.7'	R(5.0)	4.2
2	03/30/1943	23 06 08.6	37° 41.1'	121° 48.8'	R(5.0)	3.6
3	04/15/1943	15 23 02.6	37° 40.4'	121° 45.9'	R(5.0)	4.0
4	04/15/1943	15 31 01.4	27° 42.2'	121° 42.3'	R(5.0)	4.2
5	04/21/1943	23 39 42.9	37° 40.4'	121° 43.5'	R(5.0)	4.2
6	04/26/1943	11 53 59.3	37° 39.1'	121° 47.5'	R(5.0)	4.1
7	05/29/1943	17 04 08.0	37° 36.8'	121° 50.0'	R(5.0)	3.7
8	05/02/1946	01 26 12.0	37° 41.0'	121° 35.7'	R(5.0)	4.6
9	05/02/1946	02 11 36.6	37° 40.7'	121° 36.1'	R(5.0)	3.2
10	08/29/1949	01 56 01.1	37° 39.8'	121° 30.8'	R(5.0)	3.4
11	10/07/1956	02 38 17.2	37° 52.7'	121° 43.9'	R(5.0)	3.6
12	10/31/1958	00 26 14.6	37° 26.8'	121° 48.0'	R(5.0)	4.2
13	03/07/1962	17 52 49.5	37° 30.3'	121° 47.7'	R(5.0)	2.9
14	03/17/1962	21 38 44.4	37° 50.6'	121° 50.7'	R(5.0)	4.1
15	05/30/1963	19 57 51.9	37° 39.8'	121° 48.2'	R(5.0)	3.4
16	01/12/1964	07 48 13.7	37° 39.3'	121° 38.6'	R(5.0)	2.6
17	06/25/1965	01 01 02.0	37° 39.7'	121° 41.8'	R(5.0)	3.4
18A	06/28/1965	11 15 11.5	37° 33.2'	121° 38.9'	R(7.0)	3.6
19	07/30/1965	18 13 30.8	37° 38.6'	121° 53.1'	R(7.0)	2.9
20	08/19/1966	11 04 05.7	37° 39.3'	121° 41.4'	R(7.0)	2.5
21	02/10/1966	14 19 47.3	37° 49.3'	121° 45.3'	R(2.7)	3.3
22	02/10/1966	14 21 08.5	37° 48.1'	121° 47.3'	R(2.0)	3.9
23	07/17/1966	02 33 25.9	37° 39.9'	121° 36.1'	R(7.3)	3.6
24	07/19/1966	12 57 41.4	37° 38.8'	121° 37.4'	R(5.2)	3.3
25	08/29/1966	17 30 11.8	37° 48.3'	121° 59.8'	R(7.0)	3.0
26	09/29/1966	17 32 43.0	37° 48.1'	121° 58.9'	R(5.0)	2.6
27	03/11/1968	23 18 36.0	37° 26.9'	121° 33.6'	R(6.8)	3.2
28B	12/29/1969	19 31 11.4	37° 46.1'	121° 42.8'	R(7.0)	3.8
30	01/05/1970	17 31 12.4	37° 31.3'	121° 50.3'	R(6.7)	3.3
30	01/06/1970	08 34 55.8	37° 32.6'	121° 51.2'	R(7.0)	2.8
31	02/28/1970	02 05 00.2	37° 47.9'	121° 56.7'	R(2.0)	2.9
32	08/06/1971	09 38 39.9	37° 45.5'	121° 48.6'	R(5.0)	2.7
33	05/30/1972	02 54 46.4	37° 33.6'	121° 48.2'	R(5.0)	2.6
34	06/29/1972	05 05 54.0	37° 53.5'	121° 39.7'	R(4.7)	2.5
35	10/07/1973	17 45 12.3	37° 33.8'	121° 51.3'	R(5.4)	2.7
36C	08/31/1976	12 29 28.8	37° 37.9'	121° 40.4'	R(6.9)	3.3
37D	06/21/1977	02 43 06.9	37° 38.2'	121° 39.6'	R(9.8)	4.6
38	06/28/1977	12 46 40.4	37° 38.5'	121° 40.3'	R(9.2)	3.2

master event technique. The June 1977 ($M_L = 4.6$) earthquake was selected as the master event to determine seismographic station time adjustments to be applied to the data from the University of California, Berkeley, network. Unfortunately, two earlier stations, Fresno and Palo Alto, had been discontinued. The remaining stations, Berkeley and Mt. Hamilton (Lick), had time adjustments of ≥ 0.1 sec. Thus, this relocation method proved unsatisfactory due to the incomplete station adjustments and was not applied to the data set. The earthquake relocations were determined from the raw data obtained from the University of California, Berkeley, and USGS (NCER) data files. Particular seismograms were reread for the 1943 sequence. The hypocenters were determined by a least-squares program by Bolt and Turcotte (1963). Both P and S wave phases were used. The velocity model of Byerly for central California was adopted. This model is:

Depth (km)		
0	$V_p = 5.6$ km/sec	$V_s = 3.3$ km/sec
12	$V_p = 6.6$ km/sec	$V_s = 3.8$ km/sec
30	$V_p = 7.8$ km/sec	$V_s = 4.5$ km/sec

The relocated epicenters are shown in Figure 21 and listed in Table 4. The magnitude range is from 2.5 to 4.9. The accuracy of the epicenters varies from approximately ± 5 km for 1940 to 1968 and ± 3 km for 1969 to 1977. Also, the depth control parameters are poorly determined for the earlier events.

The results for an earthquake of a magnitude equal to or greater than 4 are:

1. The 1943 sequence, which included five events of magnitude between 4.0 and 4.2, shows an east-west alignment in the central Livermore Valley region.
2. The 1946 event, with a magnitude of 4.6, appears to have been associated with the northwest-striking Greenville fault.

FIGURE 21 SEISMICITY OF THE LIVERMORE VALLEY AREA, 1943 TO 1977, SHOWING FAULT PLANE SOLUTIONS

3. The 1958 earthquake, with a magnitude of 4.2, is associated with the Calaveras fault.
4. The 1977 event, with a magnitude of 4.6, is located along the northern section of the Tesla fault system.
5. The additional earthquakes are generally associated with the Calaveras fault and the various faults to the north and south of the Livermore Valley.

Fault Plane Solutions. The fault plane solutions were determined for a few selected events. The method used a linear velocity gradient technique to approximate the Byerly velocity model. The first motions of the P waves are plotted on an equal-area, lower-hemisphere projection. The individual solutions are shown in Figure 22a through 22d. The focal mechanisms have been plotted in Figure 21. The results indicate mainly strike-slip faulting with a northeast-southwest compressional stress system. Unfortunately, the earlier events have fewer first motion data, and the focal mechanism interpretation is more ambiguous. The June 1977 event clearly indicates a reverse motion on the northwest-striking Tesla fault system. The dip angle of this selected fault plane solution is 56°NE (Figure 22d); however, examination of the figure indicates that, although the dip is most likely northeast, its value is not well constrained and could be more nearly vertical.

Surface geologic mapping shows the Tesla fault to be dipping southwest. However, the 1977 event occurred at an estimated focal depth of 9.7 km (6.0 mi), and the possibility exists that the dip angle associated with this reverse thrust fault may well be a function of depth, in which case there is not necessarily a conflict between the two interpretations.

Probabilistic Assessment of the Exposure to Strong Motion

In order to describe the seismic exposure of the Sandia site, a probabilistic calculation was conducted to estimate the rate of exceedance as a function of peak horizontal ground acceleration at the site. Two basic sets of information are required for such a calculation. The first is a description of earthquakes in space, time, and magnitude. The second set of information consists of an empirical relation expressing the dependence of peak ground acceleration on earthquake magnitude and distance to the earthquake rupture.

06/28/65
M = 3.6

LAT = $37^{\circ} 33.2'$
LONG = $121^{\circ} 38.9'$
DEPTH = 7.0 km

FIGURE 22a FAULT PLANE SOLUTION FOR MAGNITUDE 3.6 EVENT 6/28/65

12/29/69
M = 3.8

LAT = $37^{\circ} 46.1'$
LONG = $121^{\circ} 42.8'$
DEPTH = 7.0 km

FIGURE 22b FAULT PLANE SOLUTION FOR MAGNITUDE 3.8 EVENT 12/29/69

FIGURE 22c FAULT PLANE SOLUTION FOR MAGNITUDE 3.3 EVENT 8/31/76

06/21/77
M = 4.6

LAT = $37^{\circ} 38.2'$
LONG = $121^{\circ} 39.6'$
DEPTH = 9.8 km

FIGURE 22d FAULT PLANE SOLUTION FOR MAGNITUDE 4.6 EVENT 6/21/77

If the Las Positas fault were to be substantiated by future studies as a structure capable of generating moderate earthquakes, it would not have any significant effect on the results of this statistical analysis.

The seismicity data used in the calculation are summarized in Table 5. The main constraint used in constructing the seismicity model was the historic record of large earthquakes that have occurred within about 100 km (60 mi) of the site since the 1830s. Large earthquakes (magnitude 6 or more) have occurred in this region at the rate of approximately one per decade. The seismicity of the region has been documented extensively by Townley and Allen (1938), Tocher (1958), and Bolt and Miller (1975). The major part of the regional seismic activity that could produce strong motion at the site is concentrated along well-recognized active faults of the San Andreas fault system. Although large earthquakes (magnitude 6 or more) in northern California usually occur along recognized active faults, events of small and moderate magnitude (magnitude 5 or less) are distributed more diffusely and cannot always be clearly associated with mapped active faults: this applies to the seismicity of the Livermore Valley region as registered over the past century. Presumably, the apparently diffuse seismicity is concentrated along numerous minor active faults in a manner that is not presently determined.

In the calculation, the seismicity of the Livermore Valley region was represented schematically by two seismic sources: an area source and a line source representing a minor active fault whose trace passes within close range of the site. For all of the seismic sources, it was assumed that earthquake occurrence is randomly distributed in space and time.

In Table 5, parameters A and B are coefficients in the recurrence relation

$$\log N = A - BM$$

where N is the number of events per year of magnitude M or greater. The aggregate B value for all sources combined is approximately 0.9. For the San Andreas fault, the value given for A applies to a 500-km (300-mi) segment rather than the entire length of the fault.

TABLE 5
SEISMICITY MODEL

Earthquake Source	Recurrence Parameters		Maximum Magnitude	Shortest Distance To Site (km)	Radius (km)
	A	B			
San Andreas Fault	3.65	0.8	8-1/4	55	--
Hayward Fault	4.33	1.0	7-1/2	30	--
Calaveras Fault	4.15	1.0	7-1/2	20	--
Antioch Fault	3.55	1.0	6-1/2	20	--
Concord Fault	3.55	1.0	6-1/2	30	--
Livermore Valley Area Source	3.30	1.0	5-3/4	--	20
Possible Nearby Active Fault	2.40	1.0	5-3/4	0.1	--

The relation between peak ground acceleration and earthquake magnitude and distance that was employed in the calculation was based on the work of Gutenberg and Richter (1956). The Gutenberg-Richter relation was amended for small distances in accordance with recent observations indicating that peak ground accelerations in the near field depend less strongly on magnitude than in the far field (see Hanks and Johnson, 1976). Relatively high local peak accelerations may be experienced during earthquakes of moderate size. Peak ground acceleration was treated in the calculation as a log-normally distributed quantity, with a geometric standard deviation of approximately two.

The results of the calculation are illustrated in Figure 23, which shows the probability of exceedance in any period of 30 years versus peak horizontal ground acceleration at the site. The calculation indicates that the local seismic activity in the Livermore Valley region accounts for roughly one half of the overall rate of occurrence of strong ground motion. For peak ground accelerations of a few percent of the gravitational acceleration, this conclusion is supported (although not confirmed) by the accelerograph data recorded in the Livermore Valley since mid-1943. Most of the accelerograms to date have been recorded during local earthquakes, and, in particular, aftershocks of the 1943 Livermore earthquake sequence and the Livermore earthquake of June 22, 1977 (USGS, 1977). However, the Livermore accelerograph record is too brief to afford a meaningful check of this calculation.

From Figure 23 we find that, at the median-plus-one-standard-deviation level of confidence (84.2%), the most severe acceleration experienced at the site in any 30-year interval should not exceed 0.25g. At the median-plus-two-standard-deviation level of confidence (97.7%), the largest acceleration in 30 years should not exceed about 0.5g. The probabilities of exceedance of these two levels are 15.8% and 2.3%, and are usually characterized as "small" and "negligible," respectively.

Finite probabilities of large accelerations ($>0.5g$) in this calculation reflect the assumption that peak acceleration is a lognormally distributed quantity. Because the actual distribution is not well determined, it is

FIGURE 23 PEAK HORIZONTAL GROUND ACCELERATION, PROBABILITY OF EXCEEDANCE, SANDIA TRITIUM BUILDING

difficult to attach great confidence to the estimated probabilities of exceedance of large accelerations. Thus, it is not appropriate to use this calculation to estimate the maximum potential ground acceleration.

The response spectrum shape chosen for the analysis of the Tritium Building is that developed by Newmark-Blume-Kapur. A statistical procedure was used for deriving response spectra based on a mean-plus-one-standard-deviation statistical level for all spectra examined. At this confidence level these response spectra should not be exceeded 85% of the time. The response spectra shown in Figure 24 are for 2% and 5% critical damping and are normalized to a peak ground acceleration of 1.0g.

Assessment of the Maximum Potential Ground Motion Amplitude

Evaluation Based on Earthquake Source Theory and Observational Data. As noted above, a probabilistic calculation is not informative as to the maximum potential ground motion. A deterministic approach to this question is also problematical, first because the Livermore Valley seismicity is not well understood, and second because not enough is known about earthquake sources in general, and about elastic wave propagation in crustal geologic formations in particular, to permit strictly quantitative calculations of strong ground motion. The following assessment of the maximum potential ground motion at the site is based on available strong ground motion observations, and where these are lacking (in the near field of major earthquakes), on some results of earthquake source theory.

According to the seismicity model given in Table 5, events of magnitude up to 5-3/4 may occur within close range of the site. Large earthquakes (magnitude 6 or more) are confined to major active faults that pass within 20 km (12 mi) of the site. Major events of magnitude 7 to 7 1/2 may occur on the Calaveras and Hayward faults, which pass within about 20 km (12 mi) and 30 km (18 mi), respectively. Earthquake magnitudes given in Table 5 refer to the Richter magnitude scale, which was developed to compare the sizes of earthquakes in California. For this scale, magnitude is determined from the logarithm of the maximum response of standard torsion displacement seismographs (natural period of 0.8 sec; 0.8% critical damping), which record an earthquake at epicentral distances within 600 km (370 mi). Definition of

FIGURE 24 MEDIAN-LEVEL DESIGN RESPONSE SPECTRA FOR 2% AND 5% CRITICAL DAMPING, SANDIA TRITIUM BUILDING (SPECTRA ARE NORMALIZED TO PEAK ACCELERATION OF 1.0g)

the size of an earthquake in terms of the local Richter magnitude scale is not directly descriptive of the maximum ground accelerations to be expected within close range of the event.

While far-field ground displacement scales logarithmically with magnitude (by definition of the magnitude scale), high-frequency ground acceleration in the near field does not appear to scale in the same way. For example, Hanks and Johnson (1976) reported acceleration to be "essentially independent of magnitude" in the magnitude range from 4.5 to 7.1 for a set of accelerograph data recorded at source-site distances within 10 km (6 mi). The data set used in this study, consisting of 40 accelerograms obtained from events of magnitude between 3.2 and 7.1, is biased toward unusually large accelerations for the smaller-magnitude events. A more complete set of accelerograph data might indicate more magnitude dependence of near-field acceleration than indicated by this study, but it does appear that the magnitude dependence is rather weak.

The results of recent studies in which the dislocation theory of earthquake sources is adopted suggest an explanation for the apparent weak dependence of near-field acceleration on magnitude. It is found that magnitude is closely related to the seismic moment, which is given as the product of the rupture surface area, the average dislocation amplitude, and the rigidity of the medium wherein elastic strain energy is stored prior to seismic release. Kanamori and Anderson (1975) analyzed the source parameters of 41 large shallow-focus earthquakes and found that the static stress drop, which is proportional to the ratio of the average dislocation to the rupture dimension, is practically independent of seismic moment. Kanamori and Anderson found that the static stress drop depends more on the tectonic environment than on seismic moment, and gave average figures of 30 bars for interplate earthquakes occurring along, or parallel to, major plate boundaries, and 100 bars for intraplate earthquakes (1 bar = 15 psi). The 1952 Kern County and 1971 San Fernando earthquakes were classified in the latter category, while earthquakes of the San Andreas fault, and parallel or subparallel faults such as the Hayward and Calaveras, fall in the first category.

If it is presumed that the dynamic stresses that determine near-field acceleration amplitudes during faulting are proportional to the static stress

drop, then the weak dependence of near-field acceleration on magnitude can be seen as a reflection of the moment (or, equivalently, magnitude) independence of static stress drop. This model is undoubtedly a simplification of reality: for example, the Earth's crust is not a homogeneous elastic medium, and actual rupture surfaces may have considerable geometric complexity, so dynamic stresses may vary considerably along the length of a rupture. The similarity of static stress drops for events differing vastly in moment suggests that the same physical processes are involved in small and large earthquake ruptures.

Three pertinent conclusions can be drawn from the foregoing considerations. First, moderate earthquakes occurring in the Livermore Valley may produce locally severe ground accelerations, although accelerations recorded during such events in the Livermore Valley since instruments were first installed in 1943 have not exceeded 0.1g. Second, although no accelerograph data have been recorded to date within 20 or 30 km (12 to 18 mi) of earthquakes as large as the magnitude $7\frac{1}{2}$ events postulated for the Hayward and Calaveras faults, it is most probable that peak ground accelerations would not differ appreciably from those observed for events one magnitude unit smaller in the same distance range. Third, the appropriate accelerograph data to consider in assessing the maximum potential ground acceleration at the site are data recorded in the near field of earthquakes of the San Andreas fault and associated subparallel subsidiary faults. Data for three such events are listed in Table 6. Our best estimates of the largest accelerations that would occur at the site due to the earthquake sources listed in Table 5 are 0.5g (horizontal) and 0.3g (vertical).

Evaluation Based on Fault Characteristics and Observational Data. An alternative procedure for estimating maximum potential earthquake motions at a site based on capability of the causative fault is in use for evaluation of nuclear power plant sites and is also used in general engineering practice.

In this method, the fault capable of producing the greatest motion at the site is first determined. Then the maximum potential earthquake for the fault is placed on the fault at its nearest point to the site. The resulting ground motions at the site are calculated as a function of magnitude and distance

TABLE 6
NEAR-FIELD ACCELEROGRAPH DATA FOR STRIKE-
SLIP EARTHQUAKES OF THE SAN ANDREAS FAULT SYSTEM

Event	Date	Magnitude	Recording Station	Closest Distance to Fault (km)	Peak Ground Acceleration (cm/sec ²)		
					Horizontal	Horizontal	Vertical
Long Beach	3/10/33	6.3	Long Beach	5	193	156	280
Imperial Valley	5/18/40	7.1	El Centro	10.8	342	210	206
Parkfield	6/28/66	5.6	Cholame-Shandon 2	0.1	480	--	202
Parkfield	6/28/66	5.6	Cholame-Shandon 5	5.5	425	348	117
Parkfield	6/28/66	5.6	Cholame-Shandon 8	9.6	270	233	78
Parkfield	6/28/66	5.6	Temblor	10.6	341	264	130

(site properties are sometimes also taken into account). In this case, site properties are average and are not given special consideration. The basic step in this procedure is to assign a maximum potential earthquake to each fault of interest. This can be done by referring to the historic earthquake record or by estimating the earthquake magnitude based on total length of the fault, length of surface rupture, or other factors such as amount of displacement for the event if that should be known from geologic studies of past earthquakes on the fault. Such analysis has been carried out for a nearby facility (Blums, 1972). Our current findings tend to substantiate our earlier evaluation, which placed major emphasis on capability of the Tesla fault. The following discussion has been adapted, with minor modifications, from the 1972 Blume evaluation.

Most of the known faults near Sandia and the Tritium Building are limited in their total length and are therefore not potential locations for the maximum event; one fault is not considered because it is inactive. The lesser faults are Strand No. 1 of the Tesla (inactive), Strand No. 3 of the Tesla, the Corral Hollow fault, the Ramp Thrust fault, and the Doutherty fault.

The two larger faults, which will govern for considerations of vibratory motion, are the Tesla, passing near the site, and the Carnegie fault, also passing northeast. A third fault, called the Greenville, ends in the hills east of the Carnegie fault and extends northwestward from there. Huey (1948) states that the Greenville fault does not join the Carnegie and that its possible connection with other faults to the northwest is very doubtful. He estimates displacement on the Greenville to be about 100 to 300 ft in the vertical direction. The horizontal component is unknown but is probably not large. Therefore, it is concluded that the fault is not a major structure. The Carnegie fault is about 16 mi long from a point at its junction with the Tesla fault to an estimated point extending 4 mi across the alluvium of Livermore Valley. Its surface trace is about 1½ mi from the Tritium Building.

The Tesla fault is the largest fault with the nearest location to Sandia, and evidence of its recent activity is more conclusive than in the case of the other faults. The Tesla fault strikes S80°E and continues to the

southeast a distance of 11 mi from Sandia, where it intersects a major fault trending $N45^{\circ}W$. This is considered to be the southeasterly end of the Tesla fault because it is highly unlikely that two faults such as these intersecting at an angle of 35° would act as one fault in case of surface rupture or fault displacement. The northwesterly limit of the Tesla fault is not known but appears to be well beyond the area that was investigated as part of this study. A conservative estimate for the northwesterly extent of the Tesla fault is 5 mi across Livermore Valley beyond Sandia. The total length of the Tesla fault is therefore estimated to be approximately 16 mi.

The reported Las Positas fault passes very close to the Tritium Building, as does the Tesla fault, and its total length is about 10 mi. If the Las Positas fault were to be substantiated by future studies as a structure capable of generating moderate earthquakes, the maximum credible earthquake based on this length and the resulting ground motion at the site would be less than is estimated for the Tesla fault.

In order to estimate the probable magnitude of an earthquake that might occur on the Tesla fault, it is necessary to estimate the length of a surface rupture that could occur on such a fault. The distance over which the Tesla fault has controlled the course of Arroyo Seco Creek is about 4 mi. This might be considered indicative of the length of surface rupture for a single displacement. It would be unreasonable to presume that surface rupture might take place along the entire length of the fault simultaneously. A surface rupture length of 6 mi is considered to be a conservative estimate because it equals more than one-third of the total fault length. Data published by Bonilla (1967), relating length of surface rupture on the main fault to earthquake magnitude, show that an earthquake magnitude of 5.7 would be a conservative estimate for a 6-mi length of surface rupture on a reverse fault.

This event of magnitude 5.7 is selected as the maximum potential earthquake. It is located on the Tesla fault at its nearest point to the Tritium Building. Allowing for an 80° south dip on the Tesla fault and a focal depth of 5 mi for the earthquake, the epicentral distance would be about 2 mi from the Tritium Building. Because this is a very close earthquake, it is

necessary to estimate the peak ground acceleration by comparison with available records of peak acceleration at very small distances from the epicenter. A publication by Boore and Page (1972) reviews data relating to accelerations near faults that have moved during moderate-sized earthquakes.

For an earthquake magnitude of 5.7 and epicentral distance of 2 mi, a conservative estimate of peak ground acceleration based on the data of Boore and Page is 0.5g, a value that is in good agreement with other methods of analysis described above.

SUMMARY AND CONCLUSIONS

Seismic and geologic studies have provided ground motion parameters for use in earthquake resistive design and analysis of the Tritium Building, as well as evaluating possible fault rupture hazard. The studies have also evaluated a linear feature, the Las Positas lineament, originally identified by Blume (1972) as a possible fault and more recently described in a USGS Open File Report (Herd, 1977). Work consisted of review of existing data, and of acquisition and analysis of new data from field exploration and field examinations. It included a compilation of available aerial photographs and imagery, accompanied by an examination of this data to determine prominent lineaments relevant to the study. A gravity survey was conducted in the area to delineate possible density contrasts that might be related to fault displacements. Magnetic and resistivity surveys were also conducted to determine any possible magnetic or resistivity anomalies that might be helpful in identifying and locating fault traces. Also, seismic refraction techniques were tried, but these did not appear to be promising for existing conditions and for the particular requirement of this project. Seismologic studies which were carried out included a review of the seismic history of the region, consideration of the seismotectonic regime in the area, a probabilistic assessment of exposure to strong ground motion, and an assessment of the maximum potential ground motion amplitude. The hypocentral parameters of nearly 40 earthquakes selected from the historic records were redetermined, and fault mechanisms were determined for many of these events as an aid in defining the seismotectonic regime. A final phase of the field studies for this project consisted of trenching along the probable locations of the Tesla fault and the Las Positas lineament in the vicinity of the Tritium Building.

Based upon the geologic studies conducted during this survey, it is concluded that the northwesterly striking Tesla fault is the best documented active fault in the vicinity of Sandia. It is also our opinion that field geologic evidence is not sufficiently conclusive to confirm the existence of the Las Positas fault.

Study of lineaments from available photography and imagery showed that the Las Positas is the most prominent linear passing near the Tritium Building. Lineaments related to the main strand and one minor strand of the Tesla fault were also observed.

The gravity survey revealed a significant residual anomaly coinciding with the location of the Las Positas lineament. It appears that this trough-like anomaly shown in contours of the Bouguer gravity data is related to relatively shallow density contrasts within the Livermore gravels. This anomaly could be produced by a northwesterly dipping contact surface between low-density silt beds and high-density gravel beds of the Livermore gravels. On the basis of results of a regional gravity survey by the USGS and of the URS/Blume survey, we conclude that the most straightforward, consistent view of the gravity data in the southern Livermore Valley does not include faulting along the Las Positas trace. The gravity data obtained during this survey are compatible with the presence of the northwesterly trending Tesla fault, but are not conclusive in indicating its presence. The magnetic and resistivity surveys provided evidence that there are no significant magnetic or resistivity anomaly in the area covered, but they are not diagnostic as to the presence or absence of faulting.

Seismotectonic studies of historic events in the region did not provide evidence of active fault locations, but they did show that a sequence of small earthquakes which took place in 1943, oriented in an east-westerly direction, could be related to an east-westerly fault such as the reported Las Positas fault. However, the epicenters were located near the center of Livermore Valley, well north of the Sandia area. The studies also reveal that a 1946 event with a magnitude of 4.6 appears to have been associated with the Greenville fault, and a 1958 earthquake with a magnitude of 4.2 was associated with the Calaveras fault. An earthquake on June 21, 1977, which had a magnitude of 4.6 and was felt in the Livermore area, was located along the Tesla fault system.

The probabilistic assessment of strong ground motion at the site shows that at the median-plus-one-standard-deviation level of confidence (84.2%), the most severe acceleration experienced at the site in any 30-year interval

should not exceed 0.25g. At the median-plus-two-standard-deviation level of confidence (97.7%), the largest acceleration in 30 years should not exceed about 0.5g. The probabilities of exceedance of these two levels are 15.8% and 2.3%, respectively. The full ranges of probability and ground motion are presented in a curve showing probability of exceedance versus peak horizontal acceleration.

In assessing strong ground motions due to the maximum potential earthquake, it is shown that recent studies cast serious doubt on past procedures of scaling near-field peak ground accelerations by earthquake magnitude. It further appears that the stress drop accompanying earthquakes provides a better method of estimating near-field acceleration amplitudes. The maximum peak horizontal ground acceleration is estimated to be 0.5g based upon stress drop considerations. This is in good agreement with the same value estimated by methods currently in use and prescribed by the NRC for nuclear power plants.

Evidence of faulting revealed in the trenches was conflicting and, although in part suggestive of a fault along the trace indicated by Herd (1977), it is not sufficiently conclusive to substantiate the presence of a fault.

No evidence was found by trenching or during the surface geologic examinations or in any other phase of the investigation to indicate a possible hazard to the facility due to surface fault rupture along an active fault.

To summarize results of the various investigative techniques relevant to the reported Las Positas fault (see Table 7), the structure could not be substantiated as an active fault. Although it is considered possible that a minor fault might be present along the Las Positas trace, existing evidence available for review, as well as new evidence developed during this study, can be explained by a simple primary (depositional) juxtaposition of contrasting lithologic units within the Livermore gravels which strike parallel to the lineament and dip to the northwest. Some additional investigations have been proposed which would provide more evidence concerning the significance of this feature.

TABLE 7
SUMMARY OF PREVIOUS LAS POSITAS FAULT EVIDENCE

Las Positas Trace	Lineations	Trenching	Deep Drilling	Gravity	Magnetics	Resistivity	Seismicity
	+	+	0	-	-	0	-

Key:

- + Positive correlation but not conclusive.
- 0 Evidence weighs neither for nor against.
- Evidence against presence of fault but not conclusive.

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APPENDIX A

EXPLORATORY TRENCH LOGS

FIGURE A.2
GEOLOGIC CROSS SECTION OF
EXPLORATORY TRENCH T2

FIGURE A.3 GEOLOGIC CROSS SECTION OF EXPLORATORY TRENCH T3

FIGURE A.7 GEOLOGIC CROSS SECTION OF EXPLORATORY TRENCH T7

FIGURE A.8 GEOLOGIC CROSS SECTION OF EXPLORATORY TRENCH TB

FIGURE A.9 GEOLOGIC CROSS SECTION OF EXPLORATORY TRENCH T9

FIGURE A.10 GEOLOGIC CROSS SECTION OF EXPLORATORY TRENCH T10

APPENDIX B

GRAVITY DATA

GRAVITY DATA

The Bouguer anomalies for both survey periods were calculated using an effective Bouguer density of 2.2 gm/cm^3 . Terrain effects were found to be negligible, except in the vicinity of Arroyo Seco on the south O-Line and on the K-Line.

Pages I-B.2 to I-B.14 of this appendix are from the November 1977 survey, and give station identification code (1-6), coordinates, elevation (both in feet), Bouguer gravity at mean sea level, the Bouguer Anomaly, and the Residual Anomaly (all in milligals).

Pages I-B.15 to I-B.19 list the station, elevation (in feet), raw gravity from the gravity meter, Bouguer Anomaly (in milligals) relative to the smallest elevation for each line, and the Residual Anomaly (in milligals).

STATION	COORDINATES	ELEVATION	BOUSSIER GRAVITY	BOUSSIER ANOMALY	RESIDUAL ANOMALY	
1 north	0. 0.	000.91	070027.70	-00.60	-0.10	J-Line (N → S)
1	50. 0.	001.10	070027.07	-00.61	-0.09	
1	100. 0.	001.05	070027.91	-00.57	-0.10	
1	150. 0.	001.59	070027.06	-00.52	-0.02	A0 = 979,927.88 mgals
1	200. 0.	001.05	070028.00	-00.65	-0.02	AX = .681 x 10 ⁻³ mgals/ft
1	250. 0.	002.10	070028.00	-00.65	-0.05	
1	300. 0.	002.50	070028.05	-00.63	-0.00	rms = .22 mgals
1	350. 0.	002.93	070028.10	-00.58	-0.02	
1	400. 0.	003.20	070028.10	-00.59	.03	
1	450. 0.	003.31	070028.17	-00.51	-0.02	
1	500. 0.	003.25	070028.23	-00.25	.01	
1	550. 0.	003.34	070028.25	-00.22	.00	
1	600. 0.	003.92	070028.32	-00.10	.03	
1	650. 0.	003.91	070028.38	-00.10	.00	
1	700. 0.	004.10	070028.37	-00.10	.02	
1	750. 0.	004.73	070028.40	-00.02	.07	
1	800. 0.	005.07	070028.07	-00.01	.00	
1	850. 0.	005.20	070028.51	-00.07	.00	
1	900. 0.	005.61	070028.97	-00.01	.00	
1	950. 0.	006.19	070028.57	-00.01	.00	
1	1000. 0.	006.60	070028.64	-00.00	.07	
1	1050. 0.	007.47	070028.67	-00.00	.00	
1	1100. 0.	008.07	070028.71	-00.77	.00	
1	1150. 0.	008.53	070028.74	-00.74	.07	
1	1200. 0.	008.84	070028.74	-00.72	.00	
1	1250. 0.	008.91	070028.75	-00.73	.02	
1	1300. 0.	009.22	070028.77	-00.71	.01	
1	1350. 0.	009.58	070028.80	-00.68	-0.00	
1	1400. 0.	010.04	070028.77	-00.71	-0.00	
1	1450. 0.	010.16	070028.81	-00.65	-0.00	
1	1500. 0.	010.35	070028.83	-00.65	-0.00	
1	1550. 0.	010.64	070028.83	-00.65	-0.10	
1	1600. 0.	011.11	070028.80	-00.67	-0.17	
						south

STATION	COORDINATES	ELEVATION	BOUSSIER GRAVITY	BOUSSIER ANOMALY	RESIDUAL ANOMALY
South	0.	0.	979930.01	-00.27	F-line (S → N)
2	50.	0.	979930.21	-00.27	0.10
2	100.	0.	979930.00	-00.20	0.10
2	150.	0.	979930.17	-00.31	0.00
2	200.	0.	979930.17	-00.31	0.03
2	250.	0.	979930.12	-00.30	0.02
2	300.	0.	979930.00	-00.00	0.02
2	350.	0.	979930.00	-00.50	0.07
2	400.	0.	979930.01	-00.07	0.02
2	450.	0.	979930.00	-00.00	0.00
2	500.	0.	979930.00	-00.00	0.00
2	550.	0.	979930.00	-00.00	0.00
2	600.	0.	979930.00	-00.00	0.00
2	650.	0.	979930.00	-00.00	0.00
2	700.	0.	979930.00	-00.00	0.00
2	750.	0.	979930.00	-00.00	0.00
2	800.	0.	979930.00	-00.00	0.00
2	850.	0.	979930.00	-00.00	0.00
2	900.	0.	979930.00	-00.00	0.00
2	950.	0.	979930.00	-00.00	0.00
2	1000.	0.	979930.00	-00.00	0.00
2	1050.	0.	979930.00	-00.00	0.00
2	1100.	0.	979930.00	-00.00	0.00
2	1150.	0.	979930.00	-00.00	0.00
2	1200.	0.	979930.00	-00.00	0.00
2	1250.	0.	979930.00	-00.00	0.00
2	1300.	0.	979930.00	-00.00	0.00
2	1350.	0.	979930.00	-00.00	0.00
2	1400.	0.	979930.00	-00.00	0.00
2	1450.	0.	979930.00	-00.00	0.00
2	1500.	0.	979930.00	-00.00	0.00

AO = 979,930.40 mgals
 AX = -1.02×10^{-2} mgal/ft

1-8.3

2	1650,	0.	649.62	070020.62	-45.66	.04
2	1650,	0.	637.63	070020.76	-45.72	.04
2	1700,	0.	636.64	070020.65	-45.63	-.03
2	1750,	0.	631.61	070020.39	-45.69	-.03
2	1800,	0.	629.90	070020.60	-46.00	-.09
2	1850,	0.	626.69	070020.63	-46.05	-.09
2	1900,	0.	624.55	070020.61	-46.07	-.06
2	1950,	0.	622.42	070020.30	-46.10	-.06
2	2000,	0.	620.56	070020.32	-46.10	-.06
2	2050,	0.	619.27	070020.36	-46.10	-.03
2	2100,	0.	617.73	070020.36	-46.26	-.02
2	2150,	0.	616.60	070020.21	-46.27	-.06
2	2200,	0.	616.02	070020.10	-46.30	-.06
2	2250,	0.	612.36	070020.66	-46.42	-.05
2	2300,	0.	610.99	070020.01	-46.47	-.05
2	2350,	0.	609.61	070027.09	-46.53	-.16
0	2400,	0.	608.19	070027.00	-46.50	-.07

north

STATION	COORDINATES	ELEVATION	BOSUVER GRAVITY	BOSUVER ANOMALY	RESIDUAL ANOMALY	
1 South	0.	0.	709.53	070032.55	-00.13	-0.25
2	50.	0.	709.25	070032.26	-00.22	-0.24
3	100.	0.	709.02	070032.02	-00.20	-0.12
4	150.	0.	708.68	070031.10	-00.32	-0.17
5	200.	0.	708.46	070031.11	-00.37	-0.18
6	250.	0.	708.21	070030.08	-00.46	-0.15
7	300.	0.	707.91	070030.04	-00.44	-0.08
8	350.	0.	707.65	070030.06	-00.52	-0.06
9	400.	0.	707.36	070030.02	-00.56	-0.06
10	450.	0.	707.19	070030.00	-00.50	-0.07
11	500.	0.	706.96	070030.01	-00.67	-0.06
12	550.	0.	706.61	070030.73	-00.78	-0.07
13	600.	0.	706.36	070030.72	-00.76	-0.14
14	650.	0.	706.20	070030.60	-00.79	-0.20
15	700.	0.	706.00	070030.50	-00.89	-0.17
16	750.	0.	705.77	070030.32	-01.16	-0.01
17	800.	0.	705.77	070030.32	-01.16	-0.07
18	850.	0.	705.47	070030.32	-01.16	-0.16
19	900.	0.	705.42	070030.25	-01.23	-0.16
20	950.	0.	705.37	070030.18	-01.30	-0.19
21	1000.	0.	710.07	070030.12	-01.36	-0.21
22	1050.	0.	711.91	070030.07	-01.41	-0.24
23	1100.	0.	713.76	070030.00	-01.50	-0.15
24	1150.	0.	715.70	070030.06	-01.62	-0.24
25	1200.	0.	717.93	070030.75	-01.73	-0.18
26	1250.	0.	720.21	070030.50	-01.90	-0.20
27	1300.	0.	722.59	070030.61	-02.06	-0.01
28	1350.	0.	724.96	070030.20	-02.20	-0.12
29	1400.	0.	727.39	070030.12	-02.36	-0.12
30	1450.	0.	729.84	070030.02	-02.46	-0.18
31	1500.	0.	732.19	070031.03	-02.55	-0.10

S. D-Line (S + N)

AO = 979,934.59 mgals

AX = -0.168×10^{-2} mgal/ft

1	1550.	0.	730.01	070031.00	-02.03	-0.13
2	1600.	0.	737.11	070031.03	-02.05	-0.00
3	1650.	0.	739.31	070031.06	-02.02	-0.00
4	1700.	0.	741.82	070031.70	-02.74	-0.00
5	1750.	0.	743.00	070031.50	-02.00	-0.07
6	1800.	0.	746.35	070031.00	-03.00	-0.10
7	1850.	0.	748.00	070031.33	-03.15	-0.15
8	1900.	0.	751.30	070031.33	-03.05	-0.17
north						

STATION	COORDINATES	ELEVATION	BOUGUER GRAVITY	BOUGUER ANOMALY	RESIDUAL ANOMALY	
1 south 0	0	749.10	070020.00	-04.00	.00	N. D-Line (S + N)
2	50	748.82	070020.50	-04.09	.00	
3	100	748.64	070020.53	-04.95	-.01	AO = 979,929.64 mgals
4	150	748.79	070020.48	-05.00	-.00	AX = $-.102 \times 10^{-2}$ mgals/ft
5	200	737.68	070020.63	-05.05	-.01	
6	250	734.65	070020.42	-05.46	.03	
7	300	731.70	070020.36	-05.12	.03	
8	350	728.69	070020.26	-05.22	-.02	
9	400	725.62	070020.19	-05.29	-.04	
10	450	722.73	070020.21	-05.27	.03	
11	500	719.91	070020.16	-05.31	.02	
12	550	716.87	070020.09	-05.43	-.02	
13	600	713.92	070020.05	-05.43	.02	
14	650	710.85	070020.01	-05.47	.03	
15	700	707.71	070020.06	-05.48	-.04	
16	750	704.70	070020.00	-05.60	-.03	
17	800	701.78	070020.76	-05.73	-.06	
18	850	698.73	070020.67	-05.61	-.10	
19	900	695.73	070020.67	-05.61	-.05	
20	950	692.76	070020.69	-05.63	-.02	
21	1000	689.75	070020.61	-05.67	-.01	
22	1050	686.68	070020.55	-05.93	-.02	
23	1100	683.65	070020.52	-05.95	.01	
24	1150	680.76	070020.50	-05.98	-.11	
25	1200	678.67	070020.40	-05.99	-.08	
north						

STATION	COORDINATES	ELEVATION	BOUSSER GRAVITY	BOUSSER ANOMALY	RESIDUAL ANOMALY	
9 South 9.	0.	723.00	070035.51	-30.97	-0.00	K-line
9	50.	0.	070035.00	-30.00	-0.15	
9	100.	0.	070035.30	-30.10	-0.00	AO = 979,935.60 mgals
9	150.	0.	070035.32	-30.10	-0.00	AX = -132×10^{-2} mgal/ft
9	200.	0.	070035.20	-30.20	-0.07	
9	250.	0.	070035.10	-30.10	-0.03	
9	300.	0.	070035.23	-30.23	-0.03	
9	350.	0.	070035.17	-30.17	-0.00	
9	400.	0.	070035.10	-30.10	-0.00	
9	450.	0.	070035.02	-30.00	-0.01	
9	500.	0.	070030.90	-30.10	-0.00	
9	550.	0.	070030.87	-30.01	-0.00	
9	600.	0.	070030.70	-30.00	-0.01	
9	650.	0.	070030.73	-30.73	-0.00	
9	700.	0.	070030.33	-30.33	-0.00	
9	750.	0.	070030.00	-30.00	-0.00	
9	800.	0.	070030.03	-30.00	-0.10	
9	850.	0.	070030.00	-30.00	-0.02	
9	900.	0.	070030.05	-30.05	-0.15	
9	950.	0.	070030.31	-40.17	-0.03	
9	1000.	0.	070030.21	-40.27	-0.00	
9	1050.	0.	070030.10	-40.10	-0.07	
9	1100.	0.	070030.03	-40.05	-0.11	
9	1150.	0.	070032.90	-40.30	-0.10	
9	1200.	0.	070032.00	-40.07	-0.20	

north

STATION	COORDINATES	ELEVATION	BOUGUER GRAVITY	BOUGUER ANOMALY	RESIDUAL ANOMALY
0	0	0	97927.95	-46.93	.26
0	0	200	97928.09	-46.91	.26
0	0	400	97928.13	-46.93	.26
0	0	600	97928.29	-46.20	.18
0	0	800	97928.37	-46.88	.18
0	0	1000	97928.38	-46.14	.08
0	0	1200	97928.45	-46.08	-.02
0	0	1400	97928.55	-45.93	.08
0	0	1600	97928.77	-45.71	-.03
0	0	1800	97928.97	-45.91	-.05
0	0	2000	97928.76	-45.72	-.28
0	0	2400	97928.92	-45.56	-.25
0	200	200	97928.14	-46.36	.17
0	200	400	97928.39	-46.19	.20
0	200	600	97928.53	-46.18	.12
0	200	800	97928.58	-45.98	.26
0	200	1000	97928.56	-45.92	.09
0	200	1200	97928.66	-45.92	-.03
0	200	1400	97928.64	-45.88	-.07
0	200	1600	97928.70	-45.69	-.05
0	200	1800	97928.97	-45.61	.01
0	200	2000	97929.06	-45.42	-.03
0	200	2200	97929.06	-45.42	-.15
0	200	2400	97929.18	-45.38	-.03
0	400	0	97928.12	-46.36	.11
0	400	200	97928.07	-45.60	.78
0	400	400	97928.41	-46.07	.15
0	400	600	97928.47	-46.01	.09
0	400	800	97928.60	-45.88	.09
0	400	1000	97928.67	-45.81	.08
0	400	1200	97928.69	-45.76	-.06

Average Bouguer Anomaly

= -45.063 mgals

-45.1 mgals

Sandia Grid

BO = $979,927.68$ mgals

BX = $.818 \times 10^{-3}$ mgal/ft

BY = $.618 \times 10^{-3}$ mgal/ft

6	800.	1600.	653.20	070020,60	-45,48	-0,14
6	400.	1600.	653,72	070020,60	-45,48	-0,00
6	400.	1600.	655,97	070020,14	-45,33	,02
6	400.	2000.	657,96	070020,23	-45,25	-0,01
6	400.	2200.	659,68	070020,26	-45,24	-0,13
6	400.	2400.	661,22	070020,27	-45,20	-0,22
6	400.	0.	630,60	070020,20	-46,20	,03
6	800.	200.	660,00	070020,37	-46,11	,07
6	800.	400.	663,20	070020,59	-46,03	,12
6	800.	600.	669,30	070020,76	-45,72	,00
6	800.	1200.	660,00	070020,90	-46,50	,03
6	800.	1400.	660,37	070020,91	-45,47	-0,03
6	800.	1600.	656,45	070020,10	-45,30	,02
6	800.	1800.	657,05	070020,25	-45,23	-0,03
6	800.	2000.	660,30	070020,30	-45,12	-0,05
6	800.	2000.	660,77	070020,07	-45,01	-0,10
6	800.	0.	657,05	070020,31	-46,17	-0,03
6	800.	200.	659,53	070020,60	-46,02	-0,00
6	800.	400.	661,07	070020,67	-45,00	,00
6	800.	600.	660,00	070020,70	-45,00	,00
6	800.	800.	660,72	070020,03	-45,04	,00
6	800.	1000.	658,00	070020,07	-45,50	-0,00
6	800.	1200.	657,11	070020,10	-45,30	,02
6	800.	1400.	655,57	070020,15	-46,32	-0,00
6	800.	1600.	652,26	070020,33	-46,15	,01
6	800.	1800.	651,05	070020,00	-46,00	-0,00
6	800.	2000.	662,15	070020,55	-46,03	-0,03
6	800.	2200.	660,10	070020,06	-46,07	-0,00

0	000.	2000.	007,42	070020,70	-00,70	-0,12
0	1000.	0.	034,25	070020,01	-00,07	-0,00
0	1000.	200.	001,90	070020,01	-00,97	-0,12
0	1000.	000.	003,02	070020,70	-00,70	-0,01
0	1000.	000.	007,07	070020,00	-00,00	,01
0	1000.	000.	001,04	070020,03	-00,04	-0,06
0	1000.	1000.	000,04	070020,07	-00,01	-0,10
0	1000.	1200.	000,00	070020,10	-00,10	-0,00
0	1000.	1000.	000,00	070020,07	-00,10	-0,10
0	1000.	1000.	000,11	070020,00	-00,00	-0,00
0	1000.	1000.	000,00	070020,00	-00,00	-0,00
0	1000.	2000.	001,07	070020,07	-00,01	,13
0	1000.	0000.	000,01	070020,00	-00,00	-0,00
0	1000.	2000.	001,12	070020,00	-00,00	-0,00
0	1000.	0.	001,00	070020,00	-00,00	-0,12
0	1000.	200.	000,07	070020,00	-00,00	-0,17
0	1200.	000.	000,03	070020,07	-00,01	-0,00
0	1200.	000.	000,00	070020,01	-00,07	-0,03
0	1000.	000.	000,00	070020,00	-00,00	-0,11
0	1200.	1000.	000,00	070020,17	-00,11	-0,11
0	1200.	1200.	000,31	070020,30	-00,00	-0,01
0	1200.	1000.	000,30	070020,00	-00,00	-0,00
0	1200.	1000.	000,07	070020,07	-00,01	-0,02
0	1200.	1000.	000,00	070020,12	-00,10	,10
0	1000.	2000.	000,00	070020,00	-00,00	,11
0	1200.	2200.	000,00	070020,07	-00,01	,05
0	1000.	2000.	000,00	070020,10	-00,10	,01
0	1000.	0.	000,01	070020,70	-00,70	-0,10
0	1000.	200.	000,00	070020,01	-00,07	-0,10
0	1000.	000.	000,00	070020,00	-00,00	-0,00
0	1000.	000.	001,10	070020,10	-00,10	-0,00
0	1000.	000.	007,21	070020,10	-00,10	-0,17

6	1000.	1000.	661.69	979929.36	-45.11	-.08
6	1000.	1200.	666.16	979929.56	-44.96	-.07
6	1000.	1400.	669.29	979929.71	-44.76	-.07
6	1000.	1600.	676.85	979929.88	-44.55	.11
6	1000.	1800.	686.19	979930.11	-44.36	.17
6	1000.	2000.	693.67	979930.23	-44.25	.16
6	1000.	2200.	691.93	979930.20	-44.20	.09
6	1000.	2400.	687.88	979930.36	-44.16	-.01
6	1000.	0.	684.91	979929.89	-44.00	-.18
6	1000.	200.	686.87	979929.96	-44.50	-.14
6	1000.	400.	689.60	979929.16	-44.30	-.06
6	1000.	600.	693.90	979929.33	-44.19	-.04
6	1000.	800.	691.72	979929.41	-44.07	-.08
6	1000.	1000.	689.62	979929.56	-44.92	-.05
6	1000.	1200.	690.69	979929.78	-44.76	-.04
6	1000.	1400.	681.19	979930.00	-44.60	.15
6	1000.	1600.	691.62	979930.11	-44.37	.18
6	1000.	1800.	691.27	979930.22	-44.16	.21
6	1000.	2000.	692.47	979930.39	-44.06	.17
6	1000.	2200.	689.67	979930.46	-44.82	.11
6	1000.	2400.	678.21	979930.59	-44.66	.06
6	1000.	0.	680.10	979929.67	-44.41	-.09
6	1000.	200.	685.75	979929.13	-44.35	-.16
6	1000.	400.	689.51	979929.33	-44.19	-.08
6	1000.	600.	693.57	979929.52	-44.05	-.09
6	1000.	800.	686.16	979929.69	-44.70	.04
6	1000.	1000.	689.99	979929.78	-44.70	.06
6	1000.	1200.	689.69	979929.90	-44.66	.10
6	1000.	1400.	706.93	979929.89	-44.59	-1.93
6	1000.	1600.	714.13	979930.31	-44.17	.16
6	1000.	1800.	696.58	979930.48	-44.09	.21
6	1000.	2000.	696.14	979930.59	-44.00	.19
6	1000.	2200.	693.70	979930.62	-43.89	.19
6	1000.	2400.	673.69	979930.66	-43.70	.05

6	2000.	0.	659,54	070020,16	-45,32	-.16
6	2000.	600.	670,32	070020,60	-44,80	-.83
6	2000.	600.	681,61	070020,77	-44,71	-.88
6	2000.	600.	686,46	070020,87	-44,61	-.90
6	2000.	1200.	692,70	070030,83	-44,25	-.17
6	2000.	1400.	698,02	070030,33	-44,14	-.15
6	2000.	1600.	710,32	070030,42	-44,66	-.11
6	2000.	1800.	715,81	070030,57	-43,95	-.10
6	2000.	2000.	701,80	070030,70	-43,70	-.10
6	2000.	2200.	694,34	070030,60	-43,60	-.20
6	2000.	2400.	683,36	070030,61	-43,67	-.31
6	2000.	0.	680,30	070020,36	-43,12	-.12
6	2200.	300.	707,90	070020,52	-42,96	-.50
6	2200.	600.	711,00	070020,79	-42,60	-.80
6	2200.	600.	719,26	070020,90	-42,25	-.80
6	2200.	600.	724,40	070030,87	-42,41	-.90
6	2200.	1000.	723,96	070030,19	-42,33	-.95
6	2200.	1200.	721,87	070030,10	-42,33	-.90
6	2200.	1400.	717,15	070030,32	-42,66	-.87
6	2200.	1600.	710,40	070030,60	-43,02	-.10
6	2200.	1800.	720,36	070030,80	-43,00	-.95
6	2200.	2000.	717,50	070030,96	-43,00	-.16
6	2200.	2200.	696,93	070030,90	-43,40	-.15
6	2200.	2400.	693,69	070031,87	-43,41	-.10
6	2400.	0.	734,16	070020,96	-44,92	-.60
6	2400.	300.	754,15	070020,69	-44,70	-.80
6	2400.	600.	766,72	070030,80	-44,40	-.10
6	2400.	600.	756,32	070020,95	-44,55	-.85
6	2400.	1000.	738,70	070030,33	-44,15	-.87
6	2400.	1200.	760,40	070030,54	-43,90	-.15
6	2400.	1400.	734,07	070030,60	-43,67	-.17
6	2400.	1600.	724,51	070031,09	-43,30	-.33

6	2400.	2400.	710,00	070031,14	-03,14	,26
6	2400.	2200.	710,00	070031,20	-03,20	,10
6	2400.	2400.	712,00	070031,27	-03,27	,10

L-Line 3/14/78 1057 + 1158

Station	Elevation	Raw Gravity	Bouguer Anomaly	Residual Anomaly
L-00	659.11	3,514.24	0.00	+ .01
L-20	660.15	3,514.17	-0.001	- .02
L-40	661.09	3,514.14	+ .03	0.0
L-60	662.09	3,514.11	+ .07	+ .02
L-80	663.11	3,514.04	+ .06	- .01
L-100	664.63	3,514.02	+ .15	+ .06
L-120	665.15	3,513.93	+ .09	- .03
L-140	666.05	3,513.91	+ .06	- .08
L-160	666.89	3,513.90	+ .17	+ .02
L-180	667.95	3,513.86	+ .20	+ .03
L-200	669.09	3,513.80	+ .22	+ .02
L-220	670.24	3,513.70	+ .20	- .02
L-240	671.38	3,513.67	+ .24	0.0
L-260	672.82	3,513.57	+ .24	- .02
L-280	674.48	3,513.52	+ .30	+ .02
L-300	676.55	3,513.40	+ .31	+ .01
L-320	678.92	3,513.28	+ .35	+ .03
L-340	681.75	3,513.05	+ .31	- .04
L-360	684.64	3,512.93	+ .38	+ .02
L-380	687.72	3,512.78	+ .43	+ .05
L-400	690.77	3,512.55	+ .40	- .01
L-00	659.09	3,514.22		

Note:

Instrument Correction Negligible

M-Line 3/10/78 0925 + 1142

Station	Elevation	Raw Gravity	Bouguer Anomaly	Residual Anomaly
M-00	659.68	3,514.53	0.00	+ .01
M-20	661.28	3,514.42	-0.00	- .01
M-40	662.53	3,514.36	+ .03	+ .01
M-60	663.60	3,514.33	+ .06	+ .02
M-80	664.34	3,514.25	+ .04	- .02
M-100	664.89	3,514.28	+ .09	+ .02
M-120	665.33	3,514.23	+ .08	- .01
M-140	665.75	3,514.23	+ .10	- .01
M-160	666.26	3,514.23	+ .14	+ .02
M-180	666.79	3,514.22	+ .16	+ .02
M-200	667.34	3,514.17	+ .16	0.0
M-220	668.26	3,514.11	+ .15	- .03
M-240	668.88	3,514.13	+ .22	+ .03
M-260	669.35	3,514.08	+ .19	- .02
M-280	669.50	3,514.08	+ .21	- .02
M-300	669.63	3,514.10	+ .23	- .01
M-320	669.75	3,514.14	+ .29	+ .03
M-340	669.85	3,514.15	+ .29	+ .01
M-100	664.91	3,514.20		

Note:

Tidal Correction = +.005 mgal/station

N-Line 3/10/78 1216 + 1427

Station	Elevation	Raw Gravity	Bouguer Anomaly	Residual Anomaly
N-00	663.45	3,514.11	0.00	-.04
N-20	663.74	3,514.14	+.06	0.0
N-40	664.09	3,514.15	+.08	0.0
N-60	664.38	3,514.13	+.09	-.01
N-80	664.63	3,514.14	+.11	-.01
N-100	664.90	3,514.18	+.18	+.03
N-120	665.34	3,514.14	+.15	-.02
N-140	665.89	3,514.14	+.20	+.01
N-160	666.02	3,514.16	+.22	+.01
N-180	666.20	3,514.15	+.23	0.0
N-200	667.03	3,514.18	+.31	+.06
N-220	668.39	3,514.07	+.30	+.03
N-240	669.15	3,514.02	+.29	-.01
N-260	669.38	3,514.04	+.33	+.01
N-280	669.75	3,514.02	+.33	-.01
N-300	670.06	3,514.00	+.34	-.02
N-320	670.12	3,514.05	+.38	0.0
N-340	669.80	3,514.10	+.42	+.02
N-360	669.71	3,514.11	+.41	-.01
N-380	669.72	3,514.11	+.41	-.04
M-100	664.92	3,514.08		

Note:

Instrument Correction = +0.006 mgal/station

O-Line 3/10/78 1441 + 1608

Station	Elevation	Raw Gravity	Bouguer Anomaly	Residual Anomaly
0-00	660.55	3,514.26	0.00	+ .03
0-20	660.67	3,514.30	+ .04	+ .04
0-40	660.81	3,514.27	+ .03	+ .01
0-60	661.04	3,514.30	+ .06	+ .02
0-80	661.28	3,514.26	+ .05	- .02
0-100	661.56	3,514.25	+ .05	- .04
0-120	661.94	3,514.29	+ .12	0.0
0-140	662.37	3,514.28	+ .13	- .01
0-160	662.81	3,514.28	+ .17	+ .01
0-180	663.49	3,514.22	+ .14	- .05
0-200	664.15	3,514.19	+ .17	- .04
0-220	664.87	3,514.18	+ .20	- .06
0-240	665.69	3,514.20	+ .28	+ .02
0-260	666.63	3,514.18	+ .31	+ .03
0-280	667.64	3,514.12	+ .33	+ .02
0-300	668.81	3,514.02	+ .30	- .03
0-320	669.78	3,514.01	+ .36	0.0
0-340	670.95	3,513.95	+ .37	- .01
0-360	672.29	3,513.90	+ .42	+ .02
0-380	673.70	3,513.80	+ .40	- .03
0-400	675.13	3,513.74	+ .44	- .01
0-420	676.43	3,513.75	+ .53	+ .05
0-440	677.51	3,513.69	+ .55	+ .05
0-460	678.29	3,513.62	+ .52	- .01
0-480	679.09	3,513.62	+ .59	+ .04
0-500	679.90	3,513.55	+ .56	- .01
0-520	680.72	3,513.51	+ .58	- .02
0-540	681.71	3,513.47	+ .60	- .02
0-560	682.82	3,513.46	+ .67	+ .02
0-580	684.08	3,513.39	+ .68	+ .01
0-600	685.72	3,513.26	+ .66	- .03
M-100	664.90	3,514.20		

Note:

Instrument Correction = -.004 mgal/station

I-8.18

P-Line 3/14/78 1224 + 1431

Station	Elevation	Raw Gravity	Bouguer Anomaly	Residual Anomaly
P-00	666.19	3,513.89	0.0	.14
P-20	666.46	3,513.75	-.13	-.02
P-40	666.75	3,513.78	-.08	0.0
P-60	666.64	3,513.79	-.08	-.03
P-80	667.29	3,513.75	-.08	-.05
P-100	668.50	3,513.71	-.04	-.04
P-120	670.01	3,513.60	-.05	-.08
P-140	671.62	3,513.59	+.07	+.01
P-160	673.96	3,513.44	+.08	-.01
P-180	677.55	3,513.26	+.13	+.02
P-200	680.27	3,513.07	+.12	-.02
P-220	682.02	3,513.04	+.21	+.04
P-240	683.31	3,512.97	+.22	+.02
P-260	684.41	3,512.90	+.22	0.0
P-280	685.30	3,512.90	+.28	+.03
P-300	685.64	3,512.86	+.27	-.01
BM-117		3,515.14		

Note:

Instrument Correction = +.01 mgal/station

PART II

STRUCTURAL INVESTIGATIONS OF THE
TRITIUM RESEARCH FACILITY

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1. INTRODUCTION

This report summarizes the results of the structural investigation of the Tritium Research Facility, Building 968, at the Sandia Laboratories' Livermore site, including the steel stack and those components and systems within the facility that are important to its safe operation.

Building 968, originally named the Aerostatics Laboratory Building, consists of two separate structures: the laboratory building and a small building known as the equipment room, which is located 22 ft. north of the laboratory building and is connected by beams that support a canopy.

The components and systems to be included in this investigation were agreed upon after a site visit by personnel of URS/John A. Blume & Associates, Engineers (URS/Blume), accompanied by Sandia personnel, on December 14, 1976. These items are as follows:

1. Piping and supports for the gas purification system supply and return (GPSS and GPSR) systems and the vacuum effluent recovery system (VERS)
2. Roof holding tank supports and connecting piping
3. Glove boxes and connecting piping
4. Exhaust duct system
5. Decontamination equipment supports (skids) and connecting piping

An additional item of work in the investigation of the laboratory building was the request by Sandia to study the effect on the structural strength of the building of enlarging the door opening on the south wall of Room 115A, where the decontamination skids are housed.

The general objective of the work reported here was to determine the ability of the existing structures and systems to withstand the seismic forces predicted from the geological investigations contained in the first part of this report. In addition, Building 968 and the stack were analyzed for compliance with the lateral force provisions of the 1976 Uniform Building Code (UBC). Remedial concepts or schemes for strengthening the structures and systems were developed. The structures and systems were then analyzed in their condition as modified according to the remedial schemes.

Each structure or system was investigated with methods that included mathematical modeling and computer analysis, as well as conventional hand calculations. In each analysis, the strength or capacity was calculated and compared with the expected demand of the postulated ground motion and the UBC.

2. SEISMIC CONSIDERATIONS AND STRUCTURAL EVALUATION CRITERIA

2.1 Seismic Considerations

The Tritium Research Laboratory was evaluated for compliance with the 1976 UBC. Concurrently, geologic and seismicity investigations were performed to determine the seismic exposure of the site (see Part I of this report). From these investigations, probability curves were developed that indicate the probability of exceeding any level of ground motion up to the maximum credible level for a given period of time. Parametric structural evaluations were then performed by dynamic analysis of a mathematical structural model of the facility. These analyses were performed for a level of ground motion with a reasonable probability of being exceeded. The maximum ground acceleration of 0.275g* selected for these analyses represents a 84% probability of not being exceeded in 30 years or 75% in 50 years. The spectral shape selected for the analyses was that developed by URS/Blume from a study of 38 earthquake records. The structures were analyzed for the mean or median spectrum. A comparison of structural responses of the existing structures to the 1976 UBC and the spectra for the postulated ground motion is given in a following section.

Chapter 0531 of the DOE manual covering the safety of nonreactor nuclear facilities (which would include a tritium facility) refers to application of DOE Appendix 6301 for structural design. Appendix 6301 is part of the DOE Facilities General Design Criteria. Part 1, C covers the Basic Structural Design. It states that earthquake loading criteria shall be in accordance with the UBC. Appendix 6301 also states that "facilities, such as those for radioactive material processing and storage, may require application of a dynamic analysis in determining structural requirements for earthquake loading." This requirement which is similar to that in UBC Section 2312(1), means that, for "irregular or unusual" structures, the force distribution to the resisting element should be established by dynamic analysis performed for the base shear required by the code. (Sandia has determined that the 1976 UBC earthquake requirements constitute the appropriate seismic criteria for this facility.) Thus compliance with the 1976 UBC would satisfy the requirements of DOE Appendix 6301. Our comparisons indicate that the loads applied via dynamic analysis for site-specific criteria are greater than those derived from the UBC. The site-specific criteria used with the dynamic analysis are considered to be a more realistic appraisal of the seismic exposure and structural response. This results in a more conservative approach than would be provided by UBC.

*This value was used for the dynamic analyses in this part of the report. The value of 0.25g (Part I) was derived from studies of all data subsequently available. However, results of analyses would not be materially different if the latter value were used.

2.2 Structural Evaluation Criteria

Each structural system was evaluated in its existing condition and in its modified conditions according to the remedial concepts developed in Section 5.4 to determine its capacity with respect to the dynamic analysis and to the 1976 UBC seismic forces. Two evaluation criteria were used. For the UBC analysis, the generally accepted allowable stress values were used in determining structural element capacities. The allowable stress values for the different materials come from the various material codes (see 3 below). For the median spectrum analysis, "ultimate" stress values were used in evaluating structural element capacities and were derived from the various material codes and information from the literature (see 3 below).

Other aspects of the structural evaluation criteria are summarized as follows:

1. Combined Forces: The vertical load forces in general do not stress the lateral-load-resisting-system elements significantly. Therefore, it was not necessary to combine horizontal earthquake forces with vertical loads except for the case of shear walls investigated for overturning. In this case, it was sufficiently conservative to assume that 90% of the dead load acts to resist overturning.

In investigating vertical-load-carrying elements, the calculated vertical dynamic forces were combined with actual dead-load forces (and actual live-load forces where applicable) wherever that would lead to significantly different results.

2. Combined Earthquake Components: The horizontal ground motion was considered applicable in any horizontal direction. The worst-condition direction was chosen for each structural system.

1. the building system, because the lateral-load-resisting system is distinct from the vertical-load-resisting system, it was generally not necessary to combine the horizontal and vertical earthquake components.

In the other structural systems, the same general approach was taken. Only when combining the two components would result in significantly different conclusions were the vertical and horizontal components combined.

The combination process used was the square-root-of-the-sum-of-the-squares (SRSS) procedure.

3. Allowable Stress and Ultimate Stress Criteria: The allowable stress values for the structural steel components of the structures were determined by the specifications of the 1971 American Institute of Steel Construction code. The same values for concrete components were determined by section 8.10 of the American Concrete Institute Code 318-71. The values for gypsum concrete components were determined using the UBC Standard No. 24-12, Part II. Various other papers and manufacturers' reports in the literature were also consulted in evaluating the gypsum concrete. The "ultimate" stresses used here are not the failure stresses, but are values of allowable stresses

increased by an appropriate factor. For steel members the ultimate capacity would correspond to the load at yield stress. Ultimate capacities for other materials would have the same approximate relation to their failure load as the ratio of yield to failure stresses for steel.

3. BUILDING AND SITE SURVEYS

In the course of the investigations, several surveys of the Tritium Research Facility were conducted to familiarize the personnel working on the study with the facility, to identify the vital structural systems, to verify structural configurations and details as given on construction drawings, and to obtain information, such as weights and dimensions, not shown on the drawings. Dead load and live load assumptions were verified where possible. In particular, field investigations to determine as-built conditions were necessary in the study of the laboratory building precast wall construction. Also, surveys of the skid support structure were required because the as-built conditions do not necessarily conform to the construction drawings. A survey of nonvital systems was also made to determine where a failure of these systems could damage those systems that are important. Additional surveys were required to determine the feasibility of structural modifications to the systems. The results of these surveys are presented in the sections covering the various structural systems.

4. ANALYSIS PROCEDURES

In the first phase of the studies, the buildings and the steel stack were analyzed for their adequacy with respect to the lateral forces prescribed in the 1976 UBC. These structures had been originally designed under the 1970 UBC, which had lower requirements for earthquake forces. This procedure is essentially a static load analysis.

In the next phase, dynamic analyses were done for these structures. The results of these analyses were then used to determine whether strengthening was required. Subsequently, remedial concepts were developed, and the structures were analyzed again.

In the case of the vital systems, especially the piping, no bracing or other lateral-force restraints had been designed for the original construction. For these systems, remedial concepts were developed and then preliminary analyses were made for the modified conditions.

Piping connected to the various systems takes on numerous configurations and was therefore not analyzed individually. Calculations determined allowable unsupported spans for various sizes and types of pipe. Typical support locations were recommended, and typical support details were shown schematically because they are not required to develop large loads.

4.1 UBC Analysis

The UBC analysis consisted of a check of seismic forces as prescribed in Section 2312 of the 1976 UBC. Seismic requirements for buildings are based on the formula:

$$V = ZIKSN$$

where:

V = total lateral force at the base of the structure that the structure must be capable of resisting (base shear)

Z = zone factor of seismic risk, which is specified as 1.0 for Livermore

I = occupancy importance factor, varying from 1.0 to 1.5 depending on the type of occupancy

K = horizontal force factor, specified as 1.33 for buildings with a box system, which the laboratory building has

C = coefficient that is a function of the period and therefore the stiffness of the building (stiffer buildings have higher values of C)

S = coefficient for site-structure resonance. S varies between 1.0 and 1.5 depending on the relationship between the periods of the site and the structure. The product of CS need not be greater than 0.14 and is the value used here. (See further discussion in Section 5.3.1)

W = dead load of the structure

For parts of a structure, including equipment -- such as the vital systems included in this report -- a formula similar to the base shear formula is prescribed. The lateral force F_p that the system must be capable of resisting is calculated as follows:

$$F_p = ZIC_pSW_p$$

in which the symbols are the same as for the base shear formula except that the subscript p denotes those values for the part of the structure rather than the whole structure.

The UBC allows the use of stresses one-third greater than allowable values when a structure is subjected to wind or seismic forces.

4.2 Dynamic Analysis

Forces under postulated earthquake conditions were estimated using dynamic analysis procedures. Both elastic and inelastic response were considered in system evaluation, but only computer routines using elastic response were used for analysis. The analyses were conducted for the postulated earthquakes using the response spectrum modal superposition technique. All modes having frequencies less than 33 Hz were included. In systems where hand calculations were appropriate, only the first dominant mode of response was considered.

Properties of each analysis model were selected to closely approximate the mass and stiffness distribution of the actual structural system. All major structural elements that contributed lateral stiffness were considered. Masses were calculated from construction drawings whenever possible. For systems with a large amount of piping or other equipment, Sandia furnished the required mass data. Member stiffness properties of moment of inertia, shear area, and cross-sectional area were determined for the various elements of the structural systems. These properties, along with the appropriate dimensions and elastic moduli, were then used as input stiffness data for the various computer and hand calculations. Values of the modulus of elasticity and shear modulus used in the analyses are discussed in the individual systems sections as are damping, inelastic response, and ductility considerations.

4.3 Computer Programs

Several computer programs were used for the analysis. These were TABS, SAP IV, and SAP I. In these analyses and in hand calculations, possible effects of soil-structure interaction were assumed to be negligible. These computer programs are all capable of three-dimensional analysis. However, this capability was not required here because the structures in this study were simple enough to be analyzed as two-dimensional models.

TABS is a computer program developed at the University of California for the linear structural analysis of frame and shear wall buildings subjected to both static loads and earthquake loadings. Each building is modeled by a system of independent frame and shear wall elements interconnected by rigid floor diaphragms. Column bending, axial, and shearing deformations are considered, and beam and girder bending and shearing deformations are also included. Shear panels can be considered with individual frames. Finite column and beam widths can be included.

SAP IV is a finite-element structural analysis program for the static and dynamic response of linear two- or three-dimensional systems. The program is written to analyze structures that are idealized by combinations of structural element types and has been modified and improved by URS/Blume. There is practically no restriction on the number of elements, the number of load cases, and the bandwidth of the equations to be solved. In a dynamic analysis, the options are (1) frequency calculations only, (2) frequency calculations followed by response history analysis, (3) frequency calculations followed by response spectrum analysis, and (4) response history analysis using step-by-step direct integration.

5. LABORATORY BUILDING AND EQUIPMENT ROOM

5.1 Description

The laboratory building (hereafter called the lab building) is a one-story structure with concrete tilt-up wall panels and a poured gypsum concrete roof deck supported on structural steel framing (see Figure 1). Plan dimensions are approximately 80 ft. by 180 ft., with two lines of steel columns in the interior at the sides of the central corridor, which runs the entire length of the building. Interior height averages 18 ft. to the underside of the roof deck. Exterior columns are concrete, poured in place, and tie the tilt-up panels together. Footings occur only under columns. Wall panels span between and are supported by the column footings. The gypsum roof deck is 3 in. thick, reinforced with welded wire fabric and poured on 1-in. form boards laid on bulb tees at 24-in. spacing. The roof deck is connected to the walls by 5/8-in. threaded rods screwed into inserts cast into the wall panels. The protruding end of the rod is embedded in the gypsum. The steel roof girders run in the north-south (transverse) direction and at the exterior walls are connected through clip angles to a steel plate anchored to the concrete columns. These lines of girders hold the wall panels together to prevent separation from the roof deck. In the longitudinal direction, the steel beams are similarly anchored but to the wall panels because they do not line up with the concrete columns. The floor is a 6-in. slab on grade and is tied to the walls and columns by reinforcing bars doweled out from the walls and columns.

The equipment room is a much smaller building, 34 by 40 ft. in plan, but of similar construction. The main difference is in the roof deck, which is a 6-in. concrete slab instead of gypsum. The reason for the heavier roof is that it supports equipment over most of its area. The main supply and exhaust ducts with their fans, motors, filter banks, sound traps, etc., are supported on concrete curbs on top of the roof. Another difference is the structural steel columns at each corner, which support the steel roof beams directly. These columns are placed just inside the corner concrete columns. Another pair of steel columns is located at the center of the north and south walls. These two are 4-in. pipe columns and are embedded in the concrete columns. The interior space of the equipment room is clear of columns and has a height of 12 ft. 5 in. to the slab soffit. The floor of this building is 1 ft. lower than the lab building floor. The two buildings are connected by steel beams at the level of the equipment room roof. These beams support a metal deck canopy, which covers the space between the two buildings. This space is designated as the service area (see Figure 2). In both buildings, the roof deck is the diaphragm, and the tilt-up panels are the shear walls for resisting seismic forces.

In the lab building, the construction drawings show that the horizontal wall bars, projecting into the poured column from one side, are wrapped with paper. The bars projecting from the wall on the other side are not wrapped.

This was done to prevent concrete bond along one side of each panel, apparently to reduce shrinkage and temperature cracking in the wall. The drawings are unclear where the slip joints occur, but their existence and locations were generally verified by site surveys. Examination of the wall panels indicated that the joints on one side of the columns on the north and south walls have opened up. On the east and west walls, opened joints were also observed, but only on the corridor side of the columns on lines D and E. From this it is reasoned that wrapping on the reinforcing bars occurs only on the small center panel on the east and west walls.

Construction drawings and photographs taken at the time of error indicate that the horizontal reinforcing in the wall beams, formed in the top of the panels, is not wrapped at the columns. Site surveys, however, have revealed cracks along panel edge lines extending up through the wall, indicating that wraps might be present. The drawings show that the laps of these reinforcing bars occur at the columns, are approximately 12 in. long, and are not sufficient to develop the bars in tension, even if unwrapped. Because of this discontinuity of the wall panels, the steel ledger angle at the junction of the roof and wall was designed to act as the diaphragm chord. This angle is spliced by full-penetration welding to obtain the continuity required for chord action.

5.2 Analysis of the Buildings

The lab building and the equipment room were designed originally as separate structures. However, as mentioned in Section 5.1, the beams connect the roof of the equipment room to the lab building columns at a point 6-1/2 ft. below its roof deck. Any tendency for differential movement between the two buildings in the north-south direction will force the lab building columns to act in bending on a 6-1/2-ft. cantilever. Structural distress is possible if differential movements are large. However, the buildings are analyzed as separate structures, and, where movements are sufficiently large to cause distress, remedial measures, such as modifying the connection to allow slip, will be recommended.

5.2.1 Lab Building. It is apparent from the shape of the building that the critical direction for earthquake motions is the north-south direction. Earthquake motions in other horizontal directions generate much smaller stresses. Consequently, the bulk of the study was based on a north-south earthquake. The critical element of this system is the gypsum concrete diaphragm. For the system in its existing condition, the diaphragm experiences stresses exceeding its design capacity at a relatively low level of peak ground acceleration.

5.2.2 Equipment Room. Because of its smaller size and its concrete roof diaphragm, the lateral-force-resisting capacity of this building is much higher than that of the lab building. The equipment room, when analyzed as a separate building, is capable of resisting a base shear in the order of 0.7g.

No remedial concepts are required except when considering this building as linked with the lab building in the same structural system.

5.3 Analysis and Results for Laboratory Building

5.3.1 UBC Analysis. A description of the different variables in the 1976 UBC earthquake base shear formula $V = ZIKCSW$ can be found in Section 4.1. The building period, T , is calculated from the code formula $T = 0.5h_n/D = .11$ sec. This yields a value of C larger than the maximum required by the code. Similarly, using geological data collected for Part I of this report, the calculated value of $S = 1.4$. The product of CS , thus calculated, is larger than the maximum value required by the code; therefore, the maximum value of $CS = 0.14$ is used. Other variables of the base shear formula take on the following values:

$$Z = 1.0 \text{ and}$$

$$K = 1.33$$

The factor I is specified as 1.0 for all buildings except that 1.25 is used for assembly buildings for over 300 persons and 1.5 for essential facilities. Essential facilities are defined as those "...buildings which must be safe and usable for emergency purposes after an earthquake in order to preserve the health and safety of the general public." Sandia has made an interpretation of this code section and considers the value of $I = 1.0$ as the appropriate value for this facility.

The calculated value of the base shear becomes

$$V = (1)(1.0)(1.33)(0.14)W = 0.186W$$

For the calculated value of the dead load $W = 1104$ kips,

$$V = 206 \text{ kips}$$

This value is about 29% higher than the original design base shear of 160 kips; hence, some overstress may be expected. Calculating the forces, or demand, imposed on the various structural elements of the building by the prescribed base shear in the north-south direction and comparing these with the capacity of the elements as constructed produces the results in Table 1.

The ratios in the last column of Table 1 indicate adequacy or inadequacy of the building as compared with the requirements of the 1976 UBC. Ratios greater than one indicate roughly the overstress that occurs for that structural element. Ratios less than one show that those elements are stressed at

TABLE 1
UBC ANALYSIS, V = 206 KIPS

Structural Element and Loading Condition	Demand of 1976 UBC	Capacity Based on Allowable Stresses	*Demand/Capacity Ratio
Diaphragm shear	1.33 kip/ft	1.26 kip/ft	1.06
Diaphragm chord force	58.9 kips	45.6 kips	1.29
Diaphragm-to-wall connection	1.33 kip/ft	1.48 kip/ft	0.90
Sliding of shear wall	133 kips	168 kips	0.79
Shear wall overturning	2,178 kip-ft	4,090 kip-ft	0.53
Shear on walls	31 psi	69 psi	0.45

*Ratios greater than 1.0 indicate overstress.

levels below the allowable maximum. As expected, overstress occurs in the diaphragm system. This overstress may not necessarily result in failure because there are factors of safety built into the allowable values. The original design, it should be noted, is adequate for the criteria under which the building was designed, the 1970 UBC.

An area of concern not noted in the table is the canopy connection between the lab building and the equipment room. The maximum displacement of the lab building relative to the equipment room calculated for the 1976 UBC forces is 0.24 in. This connection would probably experience some overstress, most likely in bending on the lab building column. To avoid this overstress, it would be necessary to modify the beam connection to allow some movement. Other structural elements and stresses not included in the table are generally not as significant as those listed.

5.3.2 Dynamic Analysis and Results. For this analysis, the median response spectrum with 5% of critical damping is used. Because the roof deck is relatively thin and composed of gypsum concrete, it is a relatively flexible diaphragm with a fairly large fundamental period. This situation can lead to large amplifications of earthquake motions and forces that are not adequately accounted for in the 1976 UBC. To evaluate these amplifications, it is necessary to determine the dynamic characteristics of the structural system considering the diaphragm deflection. The amplifications are most serious when the earthquake motions are in the north-south direction, where the diaphragm span is 180 ft.

The wall beam, as explained in Section 5.1, does not contribute to the diaphragm stiffness for motions in the north-south direction. The east and west walls of the lab building are assumed fixed because their stiffnesses are much higher than the diaphragm's stiffness. The gypsum diaphragm acts as a simply supported beam spanning the 180 ft. between the lab building east and west walls. The mass of the lab building tributary to the roof diaphragm level was assumed to act uniformly along the 180-ft. length. The building system in this case is sufficiently simple that hand calculations were adequate to get the dominant, first-mode response. The calculated first-mode period is $T_1 = 0.28$ sec. For the median response spectrum, with 5% of critical damping, a spectral acceleration of 0.60g at the roof results, and the base shear becomes 545 kips. The diaphragm shear is 273 kips, and the diaphragm chord force is 156 kips. Again, for these high forces, some distress would be expected. The results for the important elements are shown in Table 2. For this analysis, the demand of the calculated dynamic forces is compared with the "ultimate" capacity of the structural elements. The "ultimate" capacity is comparable to the allowable capacity with a factor of safety applied (see Section 2.2).

The ratios in the last column of Table 2 indicate the adequacy or inadequacy of the structural elements, based on ultimate strength criteria, as compared with the demand of the median response spectrum. Ratios greater than 1.0 indicate the overstress that occurs for an element and loading condition. Comparing the values in the last column with the corresponding values in Table 1, it is seen that the degree of overstress predicted by the median response spectrum is higher than that predicted by the 1976 UBC.

TABLE 2
DYNAMIC ANALYSIS, V = 545 KIPS

Structural Element and Loading Condition	Demand of Dynamic Analysis	"Ultimate" Capacity	*Demand/Capacity Ratio
Diaphragm shear	3.4 kip/ft	2.5 kip/ft	1.36
Diaphragm chord force	155.7 kips	57 kips	2.73
Diaphragm-to-wall connection	3.4 kip/ft	2.5 kip/ft	1.35
Sliding of shear wall	317 kips	264 kips	1.20
Shear wall overturning	5,770 kip-ft	4,090 kip-ft	1.41
Shear on walls	81 psi	126 psi	0.65

*Ratios greater than 1.0 indicate overstress.

As was noted in the evaluation of the building for the 1976 UBC forces, large displacements of the lab building relative to the equipment room can result in distress in the two lab building columns because of the load delivered by the canopy beam connection between the two buildings. This relative displacement predicted by using the median response spectrum forces is 0.57 in. and is large enough to cause severe distress in the lab building column to which the beam between buildings is connected. Modification of the connection is necessary.

Other structural elements not included in Table 2 are generally not as important as those that are listed.

5.4 Remedial Concepts

5.4.1 General. The results of the analyses of the lab building in the preceding section show that the structure, in its existing condition, is deficient in certain respects when evaluated against the requirements of both the 1976 UBC and a dynamic analysis using the median response spectrum. The gypsum roof deck, acting as a diaphragm to resist seismic forces, is found to have inadequate capacity. In addition, there are problems with sliding and overturning of the shear walls for forces predicted from the dynamic analysis.

A number of remedial concepts that would increase the level of resistance of the building to lateral forces have been developed and analyzed. In these remedial measures, the structural system is modified in various ways to increase the capacity of the structural system to withstand earthquake forces in the north-south direction. The effects of the remedial measures on the building's structural capacity have been evaluated using the 1976 UBC and the median response spectrum. In using the UBC, the specified base shear was distributed to the shear walls in the same proportions as obtained from a dynamic analysis of the building. The remedial concepts are described for each scheme or modified configuration of the structural system in the following section.

5.4.2 Description of Remedial Concepts

Scheme 1 (see Figure 3):

In this scheme, the lab building and the equipment room are linked together and analyzed as a single structural system. This uses the shear walls of the equipment room to help support the lab building. To determine the stiffness of the shear walls, the east and west walls of the lab building and the east and west walls of the equipment room were individually analyzed, using the TABS computer program, to determine their structural stiffness as shear elements. The linkage between the two buildings occurs on the lines of the east and west walls of the equipment room along column lines 4 and 6. In practical terms, a structural steel member connects the end of the lab building roof girders to the equipment room at its roof level. It will be necessary to add splice plates on the top flange of the girders on these two

PLAN AT ROOF
SCHEME 1

lines to achieve continuity for the members as collectors. A dynamic analysis was then made for a north-south earthquake using the SAP IV computer program. The lab building roof is modeled as a continuous beam on four spring supports, representing two shear walls in the equipment room and two shear walls in the lab building. The geometric and stiffness properties of the elements accurately model the existing structural characteristics, assuming two links are provided. Of particular importance is the modeling of the gypsum roof diaphragm of the lab building so that the deformation of that element is accounted for. Neglecting this characteristic would result in a total base shear that is unrealistically low and shear distribution to the walls that would not be correct. The equipment room roof diaphragm is a 6-in. concrete slab and can be assumed as rigid. The masses used in the SAP IV lumped-mass model are the same as for the existing condition, except that the masses of the east and west walls of the lab building are lumped at the nodes where those spring elements attach to the diaphragm. Also, the mass of the equipment room is lumped at the two nodes where the two spring elements representing the equipment room walls attach to the lab building.

Scheme 2A (see Figure 4):

This scheme is similar to Scheme 1 except that a new shear wall element is added in the lab building on column line 8. This interior shear wall replaces the existing gypsum-board partition on that column line on the north side of the corridor. The mass contributed by the new wall was added to the computer model, and the stiffness of the spring representing this new wall was determined in the same fashion as were the stiffnesses for the other concrete walls. In this scheme, splice plates in the roof girders will be required on the line of the new shear wall (line 8) as well as on lines 4 and 6 to obtain effective collector numbers.

Scheme 2B (see Figure 5):

This scheme is similar to, and is a variation of, Scheme 2A, the main difference being the degree of structural stiffness. The link between the two buildings is accomplished by extending the east and west walls of the equipment room to abut the north wall of the lab building. The thickness of these two walls will be increased to 14 in. It is anticipated that the small door and lower opening on the east wall will be closed up to obtain the necessary stiffness. A door opening will have to be provided for access to the service area between the buildings. Sandia personnel have indicated that this enclosing of the service area will not affect its function. Remedial work will also be done to the diaphragm system of the lab building by strengthening the chord member at the north and south sides of the roof. This can be accomplished by removing the concrete at the top of the column and splicing the wall beam bars by welding. These bars will then be effective as part of the diaphragm chord. The new shear wall in the lab building will be 12 in. thick and will have a footing. The wall concrete can be placed by shotcreting methods. Splice plates on the roof girders are the same as in scheme 2A.

In developing Schemes 2A and 2B, a number of other variations were explored. For example, the thickness of the extension of the equipment room walls could vary; the extension could be to the north as well as to the south. The

PLAN AT ROOF
SCHEME 2A

PLAN AT ROOF
SCHEME 2B

thickness of the interior shear wall in the lab building can be varied. The strengthening of the diaphragm chord could also be accomplished by exposing the ledge angle and welding on a new steel section. By adjusting the thickness and dimensions of walls, the shear distribution to the five shear walls can be varied. Similarly, diaphragm shears and chord stresses will change. In general, the more stiffening that is done, the higher the capacities that can be obtained.

Scheme 3 (see Figure 6):

In this scheme, the lab building is assumed to act independently of the equipment room. A 3-in. structural lightweight concrete deck is added on top of the gypsum deck on the lab building roof. The edges of the concrete are thickened and reinforced to act as part of the diaphragm chord. For stiffness calculation, the existing gypsum deck is neglected. The mass of the gypsum is, however, included, as is the mass of the new lightweight concrete deck, in the total mass calculations for the computer model. No interior shear walls are required in this scheme. The roof framing system has been studied, and all the structural members were found to be adequate for carrying the new load with the exception of the 21-in. girders. These girders can be strengthened relatively easily by exposing the top flange (without penetrating the roof form boards) and adding welded stud anchors to effect composite action with the new concrete deck (see Figure 7). No disruption will occur on the inside of the building because shoring will not be necessary, and the work can be accomplished without penetrating the existing roof. It will be necessary, however, to raise the main exhaust duct on the roof to allow the placement of concrete underneath. The east and west walls require remedial work to increase their resistance to sliding and overturning.

Scheme 4 (see Figure 8):

This scheme requires the reorganization of the rooms within the lab building so that four new interior shear panels can be located on column lines 4 and 7 on both sides of the corridor. The feasibility of the scheme has not been established as far as laboratory operations are concerned. If construction cost alone is considered, this is an economical scheme.

Various arrangements of shear walls were considered, such as using only two panels on either column line 5 or 6. Because this would also require drastic rearrangement of the laboratory space, it was not investigated in great detail. Another variation with only one additional wall was studied but was not considered feasible because the increase in capacity over the existing condition is minimal.

5.4.3 Results of Analyses and Evaluations. The results of the analyses of the remedial schemes are shown in Table 3, which presents the pertinent structural response data when subjected to two different force levels:

1. Median response spectrum
2. 1976 UBC

SECTION A-A

ROOF GIRDER MODIFICATIONS
SCHEME 3

PLAN AT ROOF
SCHEME 4

TABLE 3
RESULTS OF ANALYSES OF REMEDIAL CONCEPTS

Scheme	Seismic Loading*	Period of First Mode In N-S Direction (sec)	Maximum Deflection (in.)	Maximum Base Shear (kips)	Maximum Diaphragm Shear (kips)	Maximum Diaphragm Chord Force (kips)	Shear on Wall Elements Due to N-S Earthquake (kips)				
							Laboratory Building West Wall	Equipment Room West Wall	Equipment Room East Wall	Interior Wall Line 8	Laboratory Building East Wall
Existing Laboratory Building	1	0.28	0.57	545	273	155.7	273	-	-	-	273
	2		0.24	206	103	58.9	103	-	-	-	103
1	1	0.146	0.17	515	117	49.3	54	190	151	-	121
	2		0.09	272	61	26.0	29	80	100	-	63
2a	1	0.110	0.09	568	100	26.7	48	185	140	125	70
	2		0.05	273	52	13.9	23	89	67	65	33
2b	1	0.073	0.04	422	67	16.8	76	106	119	60	60
	2		0.03	311	43	10.8	49	82	92	49	39
3	1	0.060	0.07	594	297	168.7	297	-	-	-	297
	2		0.04	277	119	63.1	138	-	-	-	138
4	1	0.078	0.03	430	69	13.2	67	west interior wall	east interior wall	-	71
	2		0.02	237	39	7.3	37	160	132	-	39
								88	73		

*1 = Median response spectrum (5% of critical damping and 0.275g peak ground acceleration)

2 = 1975 UBC

It is seen in Table 3 that the remedial measures all have the effect of stiffening the lateral-load-resisting system as indicated by the decreasing periods. Remedial Schemes 1, 2A, 2B, and 4 accomplish this by adding new shear walls and/or stiffening existing ones, whereas Remedial Scheme 3 relies on stiffening the diaphragm without adding new walls. The results of this stiffening, as indicated in the dynamic analyses, are decreases in the first mode periods and smaller amplification of ground motions. Consistent with these results are the decreased base shears shown in some schemes and decreased deflections in each scheme. The UBC analyses do not reflect the same results because they do not sufficiently account for the dynamic characteristics of the structure. The total base shear is seen to increase in each scheme because of the increased building weight included in the remedial concepts.

Table 3 also presents the forces and moments on the most critical elements of the system (last seven columns). Comparable data from the analysis of the building in its existing condition are shown for comparison. It should be noted that maximum displacements, forces, and moments do not always occur at the same place in the structure for the various remedial schemes.

Table 4 compares the earthquake demands on critical elements (Table 3) with structural capacities of those elements for the various remedial schemes. The demand/capacity ratios of these elements are shown. UBC forces are compared to capacities based on allowable stress criteria. Dynamic forces are compared to capacities based on "ultimate" criteria (see Section 2.2). Structural elements not listed in the table are less critical. The ratios for the existing condition case are included for comparison. Demand/capacity ratios greater than 1.0 do not necessarily render a scheme unfeasible, but they do point out possible problem areas. Adjustments in the stiffness of certain elements and more precise calculations will bring the ratios closer to 1.0.

Scheme 1: The lab building is rigidly linked to the equipment room. There are no additional shear walls and no strengthening of the diaphragm system. At \$20,000 (estimated construction cost), this is the least expensive of the remedial schemes. It is seen that the diaphragm forces are brought down to tolerable limits with the reduction of the diaphragm spans. This scheme is adequate for UBC forces. For the median spectrum analysis there is one wall which appears to be inadequate for overturning. However, this is a strictly theoretical condition. It is highly improbable that an overturning failure could occur, especially in a one-story structure, due to the transient and vibratory nature of earthquake forces.

Schemes 2A and 2B: In these two schemes, an interior shear wall is added along column line 8, and the building is linked to the equipment room as in Scheme 1, except for additional remedial work that strengthens the diaphragm chord and shear walls. These schemes further strengthen the building system so that it is adequate for both the UBC and dynamic analyses with the exception in the latter case of overturning on one shear wall in Scheme 2A. This condition can be remedied by making adjustments in the modified wall stiffnesses to vary the shear distribution and reduce forces to acceptable levels. It should be noted that a wide range of alternative strengthening schemes linking the two buildings is possible. In general, more strengthening is obtainable at more

TABLE 4
DEMAND/CAPACITY RATIOS FOR CRITICAL ELEMENTS OF REMEDIAL CONCEPTS

Scheme	Estimated Construction Cost	Seismic Loading*	Demand/Capacity Ratio					
			Laboratory Building				Equipment Room	
			Diaphragm Shear	Diaphragm Chord Force	Shear Wall Sliding	Shear Wall Overturning	Shear Wall Sliding	Shear Wall Overturning
Existing Laboratory Building	-	1	1.36	2.73	1.20	1.41	-	-
		2	1.06	1.29	0.79	0.53	-	-
1	\$20,000	1	0.59	0.87	0.63	0.53	0.99	1.20
		2	0.61	0.56	0.55	0.28	0.88	0.63
2a	\$33,000	1	0.50	0.47	0.43	0.31	0.96	1.23
		2	0.52	0.30	0.38	0.15	0.84	0.59
2b	\$70,000	1	0.29	0.29	0.46	0.39	0.56	0.27
		2	0.43	0.23	0.47	0.25	0.67	0.18
3	\$110,000	1	0.28	0.27	1.29	1.52	-	-
		2	0.19	0.19	0.89	0.61	-	-
4	\$60,000	1	0.35	0.23	0.44	0.35	-	-
		2	0.40	0.15	0.41	0.19	-	-

*1 = Median response spectrum (5% of critical damping, peak ground acceleration = 0.275g)

2 = 1976 UBC

expense. The advantage of the more expensive Scheme 2B is that lateral displacements of the roof and problems of shear wall sliding and overturning are minimized. Smaller roof displacement may reduce the cost of remedial work on the vital systems because lower seismic forces are generated on those systems. The estimated costs are \$33,000 and \$70,000 respectively.

Scheme 3: This scheme assumes a new reinforced, lightweight structural concrete deck with strengthened chord members. The lab building is not linked to the equipment room, and no interior shear walls are added.

At \$110,000 (estimated construction cost), this is the most expensive of the remedial concepts. It can be seen that the capacity of the diaphragm is greatly improved. Displacements of the roof are held to a minimum. However, the overturning forces on the east and west walls of the building are high for the case of the dynamic analysis. Preliminary studies have indicated that these problems can be surmounted by additional remedial work on the walls, such as thickening or adding concrete at the footings. An allowance has been made in the cost estimate for this work.

Scheme 4: This scheme assumes two new lines of interior shear walls, dividing the 180-ft. length of the lab building into three 60-ft. spans. No strengthening of the chord member is necessary, and the equipment room is isolated from the lab building.

This scheme obtains a fairly high level of strengthening with relatively low cost. The estimated construction cost of \$60,000 is less than that of Scheme 2B. This scheme represents an ideal structure, in that it does not rely on interaction with the equipment room, and the increased number of shear walls results in a better distribution of shear and thus avoids the problems of large sliding and overturning forces. As noted in Table 3, the east and west walls of the lab building are more than adequate to resist sliding and overturning without modifications.

Because this scheme requires substantial cost for rearrangement of the space inside the building, it was not initially considered. This interior, non-structural work is not included in the cost estimate. A discussion with Sandia personnel indicated that this question of feasibility should not preclude an investigation of this type of scheme.

6. STEEL STACK

6.1 Description

This 100-ft.-high stack functions as the main exhaust stack for the facility. The stack is fabricated from 1/4-in. plate, and its diameter varies from 10 ft. 8 in. at the base, to 4 ft. 8 in. at the top. It is connected by 24 1-7/16-in.-diameter anchor bolts to a circular reinforced concrete foundation of 20 ft. diameter. The stack is shown in elevation in Figure 9. The breach in the stack is a 4-ft. by 7-1/2-in.-high rectangular opening, reinforced by various steel plates. The main exhaust stack is connected at this opening by bolts.

6.2 UBC Analysis and Results

A description of the different variables in the 1976 UBC earthquake base shear formula $V = ZIKCSW$ can be found in Section 4.1. The exhaust stack stands by itself, not connected to any building except by the relatively flexible sheet metal exhaust duct extending from the equipment room roof. As such, the variables in the force formula take on the following values:

$Z = 1.0$, the same value as for the buildings;

$CS = 0.14$, its maximum value, will also be used because the values of C and S calculated according to the code will yield larger results.

$K = 2.0$, applies to structures other than buildings. This value is higher than the K for buildings because the stack has less natural damping and generally fewer redundant structural elements than buildings.

$I = 1.0$

$W = 27$ kips, the dead load recalculated using the actual dimensions

Substituting in the code formula the base shear

$$V = 0.28 W = 7.56 \text{ kips}$$

The force distribution over the height of the structure is specified by the code formula:

STEEL STACK

$$F_x = \frac{(V-F_t)W_x h_x}{\sum w_i h_i}$$

In this case, F_t = zero because the calculated period $T = 0.10 < 0.7$ sec. This force distribution is essentially triangular over the height of the stack, so the resulting moment at the base of the stack (at the top of the concrete foundation) is:

$$M = 7.56 \text{ kips} \times 66.7 \text{ ft.} = 504 \text{ ft.-kips}$$

The stack was originally designed under the 1970 UBC which specified a smaller base shear coefficient. However, a conservative value for the weight and a larger height were used in the original calculations which resulted in values of base shear and overturning moments that are not too much different from the values calculated here for the 1976 UBC. Consequently, a check of the stresses from the new loading shows that the stack is adequate for the 1976 UBC earthquake loads.

Table 5 compares the earthquake demand of the 1976 UBC to the structural capacity for the most critical elements or conditions of loading. Allowable capacities are based on allowable stresses with a one-third increase where applicable.

The values in the last column of Table 5 indicate that the level of structural stresses and loading conditions is low compared to the maximum allowable values. Because all the values are less than 1, the stack is adequate in all respects for the 1976 UBC load.

Concrete and reinforcing steel stresses in the foundation are not listed, but calculations show that stresses are generally less than one-third of the allowable.

6.3 Dynamic Analysis and Results

The dynamic analysis was performed using the TABS computer program. A model with masses lumped at ten points along the height of the structure was used. The program was run, using the median spectrum (see Section 2) for 2% of critical damping, to generate the lateral forces. In the program, the participation of the first five modes was included, and the results (moments, shears, etc.) are the combined effect of all the modes. The SRSS method is used in the combination.

Seismic forces from the dynamic analysis are:

TABLE 5
EXHAUST STACK UBC ANALYSIS

Structural Element and Loading Condition	1976 UBC Demand	Allowable Capacity	*Demand/ Capacity Ratio
Anchor bolt tension	7.7 kips	25.7 kips	0.30
Shell stress, combined compression and flexure	2.1 ksi	12.0 ksi	0.18
Weld on anchor bolt bracket	4.5 ksi	29 ksi	0.16
Moment corresponding to maximum soil pressure	549 kip-ft	2,068 kip-ft	0.27
Sliding of foundation	57 kips	118 kips	0.48
Overturning	549 kip-ft	2,655 kip-ft	0.21

*Ratios greater than 1.0 indicate overstress.

Base shear $V = 9.61$ kips

Moment at bottom of stack
(top of foundation) $M = 619$ kip-ft.

It should be noted that the above values are larger than the corresponding values from the UBC analysis (Section 6.2). Table 6 lists values of demand for the dynamic analysis using the median spectrum compared with the "ultimate" value of the structural elements or load conditions.

Because the base shear and overturning from the dynamic analysis using the median spectrum are higher than the corresponding values for the UBC, the demands on the former case are also higher. However, the capacities in the above table are calculated on the basis of ultimate strength. Consequently, the demand/capacity ratios for the dynamic analysis and the UBC analysis are in the same order of magnitude.

6.4 Remedial Concepts

The results from the analysis in Sections 6.2 and 6.3 indicate that there would be no remedial work required on the stack and its foundation.

An area that may require remedial work is the connection of the exhaust duct to the stack. If the deflection of the stack at this point is larger than the maximum deflection that the connection can undergo without distress, it would be necessary to modify the connection. See Section 7.4 for recommendations concerning this connection.

TABLE 6
EXHAUST STACK DYNAMIC ANALYSIS

Structural Element and Loading Condition	Demand of Median Spectrum	Ultimate Capacity	*Demand/Capacity Ratio
Anchor bolt tension	9.4 kips	32 kips	0.29
Shell stress, combined compression and flexure	2.7 ksi	16.2 ksi	0.17
Weld on anchor bolt bracket	5.5 ksi	36 ksi	0.15
Moment corresponding to maximum soil pressure	676 kip-ft	3,102 kips	0.22
Sliding of foundation	83 kips	208 kips	0.40
Overturning	676 kip-ft	2,655 kip-ft	0.25

*Ratios greater than 1.0 indicate overstress.

7. VITAL SYSTEMS

The important systems included in this report are listed in Section 1.

Evaluation was made for each system against both the 1976 UBC and the median spectrum with 2% of critical damping.

UAC forces for the systems were determined by the formula described in Section 4.1:

$$F_p = ZIC_pSV_p$$

Values of $Z = 1.0$ and $I = 1.0$ were used, the same values as in the base shear formula for the building.

$C_p = 0.5$ is the appropriate value for systems necessary for continued operation of essential facilities.

$S = 1.0$ is believed to be sufficient in this situation because the calculated F_p becomes

$$F_p = 1.0 \times 1.0 \times 0.5 \times 1.0W_p = 0.5W_p$$

which means that seismic forces will be 0.5g. In some instances, this is larger than the calculated dynamic forces for the system in the existing condition.

For dynamic analysis, the same median spectrum as used for the buildings was used, except that 2% of critical damping instead of 5% was used. The procedure used in the analyses of the systems depended on the structural characteristics of the system or part of the system. These can generally be divided into two groups. The first group includes those items which are inherently stable and are capable, in their existing condition, of resisting fairly high lateral forces with minor modifications. These items include the glove boxes and the exhaust ducts on the roof. The second group includes those items which do not have very substantial lateral bracing. These items are the support systems for all of the piping included in this report, the roof holding tank supports, and the decontamination skids. For the pipe and duct supports and the holding tank supports, lateral bracing apparently was not provided in the original design. The skids are not currently in accordance with the construction drawings, which show some lateral bracing. For the items in the second group, it is not practical to make an analysis in the existing condition. A remedial scheme of lateral bracing was developed, and

the system was then analyzed to determine the adequacy of the bracing scheme. Further modifications were generally not considered necessary because the relatively small masses of the systems did not generate high lateral forces.

7.1 GPSS, GPSR, and VERS Piping

These piping systems considered together can readily be divided into two separate areas of study. The first is the corridor pipe rack system, including the supporting elements of the pipe rack. In this study, it is necessary to consider all elements supported by the same pipe rack (supply ducts, water pipes, etc.) because they influence the behavior of the rack. The second area of study is the GPSS, GPSR, and VERS piping and supports within the laboratory rooms.

7.1.1 Corridor Pipe Rack System. Thorough examination of construction drawings and on-site investigations have shown that, for lateral earthquake motions in any direction, the corridor pipe rack system is essentially unrestrained. A transverse section through the corridor showing the pertinent structural details is shown in Figure 10. This figure also shows a recommended bracing scheme. A section taken at one of the intermediate pipe rack supports would be essentially the same except that the connection of the 5/8-in. hanger rods to the roof system is made through a channel clamp rather than a beam clamp. Because the system is laterally unrestrained, it will experience large displacements from lateral forces with associated pounding between the various building elements. Therefore, the effort on this piping and support system focused on developing a bracing scheme that would restrain the pipe rack system and prevent large displacements or distress for the required lateral forces. Thus, the vital piping will not be damaged by a failure of the pipe rack. Also, the vital piping itself was investigated for stresses and displacements, with the recommended bracing on the supporting system.

In the dynamic analysis of the system with the assumed bracing, the following were considered:

- Possible amplification of the peak ground accelerations due to the building framing system, which supports the corridor pipe rack system

- Possible amplification of the peak ground accelerations due to the new bracing system itself and to the pipes

- Possible overstress in structural elements restraining the pipe rack system (either laterally or vertically)

The recommendation for this area of piping is the bracing scheme shown in Figure 10. In addition to the details shown, the following general recommendations should be followed:

- At the column lines, all pipes on the unistrut rack, including those that are not designated as part of the vital systems, should be attached to the rack to preclude sliding along the rack and impacting against each other.

BRACING OF PIPE RACK IN CORRIDOR

The type and size of the existing clamps attaching the 5/8-in. hanger rods to the channel at the roof should be determined by field inspection and checked for the calculated dead plus seismic load of 1.26 kips. If the manufacturers' maximum allowable load for the clamp is inadequate the clamp should be replaced by one that is capable of carrying this load.

The bracing shown in Figure 10 is a concept. Not all the details shown may be possible because of interference from existing equipment, piping, etc. However, this concept will still be valid even if braces are moved to different positions and different sizes and shapes of members are used.

The analysis of the developed bracing system indicates that it is capable of resisting forces larger than the demand of either the UBC or the median spectrum.

7.1.2 Piping and Supports in Laboratory Rooms. As in the case of the corridor pipe rack system, the piping in the laboratory rooms is also essentially laterally unrestrained, with the exception of its connection to the glove boxes and to the piping in the corridor. The GPSS and GPSR piping, upon entering the rooms from the corridor, reduces from 6-in. stainless steel to 4-in. copper. Because there are so many different pipe configurations within the rooms, it was not practicable to define every support detail. Rather, general guidelines are recommended for typical cases. These recommendations take into account the dynamic characteristics of the pipe.

None of the straight runs of pipe within the rooms is very long. But where they occur, the maximum laterally unsupported length for the GPSS and GPSR lines should be limited to 15 ft. For the VERS lines, this maximum should be 10 ft. In addition, where changes in direction occur, bracing should be installed to restrain lateral motion. By the simple expedient of modifying selected existing supports to provide lateral restraint, the system can be brought up to the capacity required by the UBC and the median spectrum. Such a detail is shown in Figure 11.

By using the suggested relatively short unsupported lengths, dynamic amplifications due to the pipes themselves will be small. Hence, lateral demand will essentially be limited to the peak acceleration of the roof plus whatever amplification is introduced by the lateral bracing chosen. Amplifications will be small if bracing schemes such as shown in Figure 11 are carried out along with other recommendations given here as to their maximum spacing.

The connections of the GPSS, GPSR, and VERS piping to the glove boxes may require the installation of flexible connections. The deflections of the glove box in any direction are expected to be small. Consequently, the demand on these connections in their existing conditions depends on the relative deflections of the roof diaphragm, the glove box, and the proposed lateral bracing for the piping. If the displacement of the roof is limited to the level described by Remedial Scheme 2B in Table 3, flexible connections should not be necessary.

NOTES:

1. MAX. SPACING FOR LATERAL BRACING
GPSJ, GPSR PIPING — 15'-0"
VERS PIPING — 10'-0"
2. LOCATE BRACE AT EACH SIDE OF
HORIZONTAL PIPE BENDS

TYPICAL LATERAL BRACING FOR
PIPE SUPPORTS IN LABORATORY BUILDING

7.2 Roof Holding Tank Supports

The tank-support system consists of a framework of structural steel members raised about 1 ft. off the roof deck. There are three tanks: two 73-in.-diameter tanks, each weighing 1750 lb., and a third smaller tank, weighing 600 lb. The contents are gaseous and have negligible weights.

The tanks are supported by steel plate saddles at each end. These plates are attached to steel channels, which are, in turn, supported by wide-flange sections spanning between jack posts. The jack posts are welded to the roof framing system of the lab building. The roof tank support system with the proposed modifications is shown in Figure 12.

By inspection, the amplification of the roof-level accelerations (referred to as base accelerations) due to the various elements of the roof tank system is significant for dynamic loads. Earthquake motions in either of the two horizontal directions would contribute significant forces because of amplifications.

A dynamic analysis of this structural system was carried out using hand calculations. This analysis indicates that large amplifications of motion could be expected, particularly for motions in the north-south direction. The resulting forces would cause stresses in the saddle channels and connections that would exceed yield stress at a relatively low level (less than 0.1g) of base acceleration.

A relatively simple scheme of bracing (see Figure 12) will eliminate a major part of the amplification and increase the allowable base acceleration from 0.07g to about 0.75g. Even higher capacities are possible by strengthening various connections. This can be easily accomplished by adding connection angles, adding welding, etc. Again, the values of base acceleration refer to the acceleration of the building at the roof level. If capacities are desired in terms of peak ground acceleration, the amplifications due to the lab building system must be accounted for.

7.3 Glove Boxes

These structures are rectangular boxes fabricated from sheet metal with various openings on all sides. They are all supported directly on the floor of the lab building. From the vendor's drawings and field surveys, it was seen that it would not be practicable to analyze the glove box with its contents as part of the same structural system. Of more than a dozen glove boxes, each has a different combination and arrangement of equipment. It was also noted that, in most cases, the equipment does not connect to the boxes except at the base plate. The work was then limited to determining the capacity of the box itself, excluding the equipment within. It is assumed, for the purpose of this study, that the interior equipment is adequately braced so that it will not impact with the sides of the box or introduce significant amplifications of acceleration to the box.

Stiffness and masses of the glove box elements were determined from the vendor's drawings and from information furnished by Sandia. The glove box was dynamically

NOTE :

ALL CONSTRUCTION IS EXISTING
UNLESS NOTED AS NEW

HOLDING TANK SUPPORTS ON ROOF

analyzed using the TABS computer program by treating the box as a frame in both the transverse and longitudinal directions. In both directions, the box was subjected to the median response spectrum and to a static force at the top level of the box to simulate a possible loading from the pipes attached at that level. Although the box was found to be quite stiff, there was some limited amplification of the transverse motions. Nevertheless, the structural elements of the box are stressed only to a low level. Even with the more conservative assumption of a 2-kip load at the top of the box, the stresses are still low. The capacity of the box as limited by allowable stress in the shell is large. Results of the analysis show that accelerations of 1.0g can be resisted without overstress. Thus, the UBC forces of 0.5g, which in this case are larger than the forces from the median spectrum, pose no problem of stress in the box. For resistance to sliding of the box on the floor capacities of over 1.0g can be readily achieved through proper design of the anchorage.

Although deformations of the boxes are not large when the boxes are subjected to lateral forces in the order of 1.0g, the probability of maintaining an airtight condition of the box under this loading is not within the scope of this study.

The only remedial measures that may be required for the glove boxes (excluding the contents) are possible installation of flexible couplings on the pipe connections at the top of the boxes and modification of the floor anchorages. Final recommendations must necessarily follow adoption of remedial scheme for the building, which will determine the maximum relative roof deflections. Each glove box must be considered individually for its special piping configuration.

7.4 Exhaust Duct System

This system consists of three main parts: (1) the round ducts, varying from 10 in. to 16 in. in diameter, which start from the glove boxes in the laboratories, run through the roof, and tie on to the main rectangular exhaust duct on the roof; (2) the main exhaust duct on the lab building roof, which starts out as a 36-in. square section and increases by increments to 120 in. by 48 in.; (3) the continuation of this duct over the canopy of the service area, which connects to the system of sound traps, baffles, and fans on the equipment room roof and then to the steel stack.

In the analysis of this system, hand calculations were used because the configuration of each part is relatively simple. The lateral forces imposed on the system are generated by the displacements and accelerations of the buildings at roof level. The capacities of the various supports on the lab building roof were calculated in terms of the accelerations they are capable of resisting. These capacities should be compared against the accelerations at the roof resulting from earthquake effects on the building as modified according to the selected remedial scheme.

The supports for the main rectangular duct in its existing condition are capable of resisting a lateral load of 0.9g at the roof level. The limiting factor is the allowable pullout value of the lag screw that connects the duct through a metal strap to the longitudinal wood sleeper under the edge of the duct. Supports for the round duct are typically a two-legged rigid frame,

18 in. high, fabricated from steel angles welded together. In their existing condition, these supports are capable of resisting a lateral load of 2.0g at the roof level.

The portion of the duct system on the equipment room roof was not analyzed because the connection details were not available. Sandia did not have drawings showing these details. In the field, the anchorages were roofed over and not visible except for the support under the fans. This support appears adequate, but the embedded length of the bolts was not known. The supports in this area are not expected to be a problem because the duct and equipment are supported on a concrete curb on top of the 6-in. roof slab. Anchorages of high capacity can be readily achieved by drilling into the concrete.

Comparisons of the capacities of the duct system calculated here against the URC requirements of 0.5g indicate that the ducts on the lab building roof are adequate. The likelihood is good that the ducts on the equipment room roof will also be adequate because the weights are relatively light. If the median response spectrum for 5% damping and 0.275g peak ground acceleration is applied to the building, the maximum roof acceleration would be 0.63g, and the duct supports will also be adequate. However, it will be necessary to check the relative deflections between the round ducts and main duct to determine whether the connections can withstand the predicted deformation. It is also possible that the joint may lose its airtightness before its structural integrity. This will be a consideration in determining whether stiffening of the supports is required.

At the point where each round duct penetrates the roof is a clamp on the pipe bolted to the roof subpurlin. This connection should be included in the investigation of the duct system when remedial measures for the building are known. Another location where differential deflections may be critical is the point where the main exhaust duct enters the stack. A flexible connection may be required at this location.

7.5 Decontamination Equipment Supports (Skids)

Decontamination equipment is found in Room 115A of the lab building. It is divided into four separate supporting frames commonly referred to as skids. These skids are identified according to the process carried out by the equipment they support. These are the dryer and regeneration skid, catalytic reactor skid, blower skid, and VERS skid.

The coupling of dynamic motions between the various skids is negligible because there is only a single 4-in., thin-walled pipe that spans between the skids. Consequently, the work done here investigates each skid, independent of the others. For all the skids, investigations showed that lateral bracing was minimal and that the existing supports mainly resist vertical loading. Hence, the investigation for each skid was aimed at developing lateral bracing systems that would limit the accelerations of the equipment on the skids.

It was assumed that the numerous pieces of equipment and appurtenances are adequately attached to the supporting framework. Only the most massive pieces of equipment, such as blowers, were investigated for adequate anchorage to the supporting framework. The other equipment was not included in the study. The approximate weights of the equipment on the skids were furnished by Sandia.

It should be emphasized that the bracing schemes presented here are concepts. From on-site surveys, they appear to be feasible, but no detailed measurements were made to verify that there are no interferences with the attached equipment. Variations on these bracing schemes are also feasible and may be preferable due to possible interferences. The concepts shown here should be adequate for either one of the standards used in the other sections of this report. Regardless of the criteria chosen, however, the bracing configurations described here would not change other than in the details. Because the forces to be resisted by the suggested bracing are small, the concepts shown should be adequate for a 1.0g force level.

7.5.1 Dryer and Regeneration Skid. (1) Cross-bracing, as shown in Figures 13 and 14, should be installed. Where equipment interferes, alternate configurations could be used. Preliminary design work shows that standard unistrut members (P5500 or smaller) with typical unistrut or welded connections will probably be adequate. However, due to the irregularity of the existing joints, special connections may be required for some of the new bracing members as well as strengthening of existing connections.

(2) All the cross-bracing done under (1) will not be effective unless each piece of equipment is adequately connected to this supporting framework. By jarring the pieces of equipment by hand, an estimate of their anchorages can be determined.

In particular, anchorages should be provided connecting the elements of the blower to the skid so that displacements in any direction are restrained. Anchorages could be simple "keepers" that butt up against the blower base and attach to the skid base. The anchorages should allow for the proper operation of the isolator pads.

(3) Because of the length of the skid and the relatively low allowable base shear on the existing anchors, two additional pairs of anchor bolts should be installed, spaced along the length of the skid. Anchors should be the same as the existing anchors at the skid corners.

7.5.2 Catalytic Reactor. (1) The gas-gas and water-gas heat exchangers should be attached to the supporting 10-in. channel sections so that they cannot slide in the longitudinal direction. These channel sections should, in turn, be rigidly attached to the supporting framework. Sketches of the suggested strengthening are shown in Figure 15(a), (b), and (c).

(2) An additional pair of anchorages should be provided midway along the length of the skid, capable of carrying shear and uplift.

NOTES:

1. NEW BRACING IS INDICATED BY
 HEAVY LINES — ALL OTHER
 MEMBERS ARE EXISTING

2. EQUIPMENT & PIPING
 ARE NOT SHOWN

NOTES:

1. DIFFERENT TYPES OF BRACING ARE SHOWN TO SUIT VARIOUS EXISTING CONDITIONS. ANY TYPE MAY BE USED AT ANY TRANSVERSE FRAME (AT GRID LINES SHOWN ON PLAN, FIG.)
2. TYPE 1 BRACING IS MOST PREFERRED. TYPE B IS LEAST PREFERRED
3. A TOTAL OF FOUR SETS OF TRANSVERSE BRACING ARE REQUIRED.
4. SEE NOTES ON FIG.

**TYPICAL TRANSVERSE
BRACING, DRYER SKID**

(a) SECTION B-B

(b) Section a-a

(c) SECTION C-C

(d) SECTION D-D

(e) DETAIL 1

CATALYTIC REACTOR SKID SECTIONS

SEE NOTES ON FIG. 16

(a) PLAN OF CATALYTIC REACTOR
SKID AT BASE

NOTES:

1. ALL STRUCTURAL FRAMING, NEW & OLD, IS REPEATED ON WEST SIDE OF C BUT NOT SHOWN.
2. NEW BRACING IS INDICATED BY HEAVY LINES — OTHER MEMBERS ARE EXISTING.
3. EQUIPMENT & PIPING ARE NOT SHOWN

(b) SECTION A-A

(3) Cross-bracing should be provided as seen in Figures 16 and 15(c). The connections can be welded or bolted, and preliminary studies indicate that standard unistrut members will probably be adequate.

7.5.3 Blower Skid. (1) The lateral bracing schemes shown in Figure 17 should be installed on the skid.

(2) It is recommended that the piping be more rigidly attached to the bracing frame. An example of this is seen in Figure 17(a).

(3) Recommendations for anchoring the blowers on the dryer regenerator skid apply to the blowers on the blower skid as well. See Section 7.5.1.

(4) Two new anchors should be installed to attach the skid to the floor slab. The anchor bolts must resist shear and tension.

7.5.4 VERS Skid. (1) Lateral bracing as described by Figures 18, 19, and 20 should be installed. It is noted in those figures that alternative bracing schemes may be required. If bracing marked A is used, then bracing marked B, an alternate, is not required (or vice versa). It is also possible that many existing connections of the supporting framework will need to be strengthened by welding. Preliminary studies indicate that standard unistrut members should be adequate for the new bracing.

Lateral bracing marked C is suggested to restrain the pumps, but any bracing actually used should allow for the proper operation of any vibration mounting of the pumps.

(2) It is recommended that the piping on the skid be more rigidly attached to the frames. This may require new horizontal or vertical members secured to the frame or skid base to which the equipment can be attached.

(3) The anchorage of the skid to the floor is not considered adequate to brace against east-west ground motions. It is suggested that two anchors (of the same type as exist on the north and south sides of the skid) be installed on the east and west sides.

7.5.5 Connecting Piping. Piping that spans between the various skids is limited to a single 4-in. stainless steel pipe. Stresses on this pipe and its connections will arise from two sources. The first source is the relative displacement of the two skids to which it is attached. The second source is the accelerations of the pipe itself under the postulated earthquake.

If adequate stiffening of skids as suggested is carried out, the relative displacements between skids should be small, and, if the pipe is securely attached at each end to the skids, the forces induced should be small. Accelerations of the pipe itself will not induce large forces at the connections if, again, the pipe is adequately braced. In particular, the pipe might require a few more anchors to the skid to prevent torsion at any of the existing bellows.

NOTES:

1. NEW BRACING IS INDICATED BY HEAVY LINES
2. EQUIPMENT & PIPING ARE NOT SHOWN

BLOWER SKID BRACING

ALTERNATE TO
KNEE BRACES
IN SECT. A-A

NEW STUB MEMBER
AT BASE
NEW DIAG. BRACE
BASE TO TOP
OF POST

PLAN OF VERS SKID

NOTES:

1. NEW BRACING IS INDICATED BY HEAVY LINE, OTHER MEMBERS ARE EXISTING
2. EQUIPMENT & PIPING ARE NOT SHOWN

* ALTERNATE TO KNEE BRACES

VERS SKID BRACING

SECTION C-C

SECTION D-D

VERS SKID BRACINGS

8. SUMMARY AND CONCLUSIONS

This study consists of an investigation of the structures and vital systems of the Tritium Research Facility to determine their structural adequacy for the requirements of two standards: the 1976 UBC and the postulated ground motion developed in Part I of this report. The ground motion used takes the form of a median-level response spectrum with a peak ground acceleration of $0.275g$ * and critical damping values of 5% for the buildings and 2% for the stack and vital systems (see Part I of this report). Remedial concepts to bring the structural capacity of the structures and vital systems up to the level of the seismic demands of the UBC and median spectra were developed. Cost estimates for the remedial concepts were made.

The analyses of the buildings showed that the equipment room was quite adequate, but the lab building was deficient for a north-south earthquake because of the low capacity of the gypsum diaphragm system. Five remedial schemes were developed that will provide the necessary increase of structural strength for the two standards mentioned above. These schemes range in cost from \$20,000 to \$110,000 and provide differing degrees of strengthening. Scheme 1 (\$20,000) links the equipment room to the lab building so that the two work together as one structural system. Laboratory operations are not disrupted. Schemes 2A and 2B (\$33,000 and \$70,000, respectively) also tie the buildings together and, in addition, add an interior concrete shear wall inside the lab building, replacing an existing nonstructural partition. There will be some disruption of the laboratory operations when the new wall is constructed. Scheme 2B differs from 2A in that the east and west walls of the equipment room are thickened and extended. Scheme 3 (\$110,000) adds a 3-in. concrete slab to the top of the existing gypsum roof on the lab building to strengthen the diaphragm system. This scheme is the most expensive one, but it provides the best diaphragm. Some disruption of laboratory operations are involved because the main exhaust duct on the roof has to be raised. Scheme 4 (\$60,000) involves adding two lines, of two panels each, of concrete shear walls in the lab building, dividing up the building into three sections of equal length. This is the best scheme when considering only the structural aspects. The disadvantage is that drastic rearrangement of the laboratory rooms is required, and this consideration may override the other advantages.

The actual remedial scheme that may be adopted could be one that combines some features of two or more of the above schemes or may be a variation of one of these.

The lab building is adequate for seismic loads in the east-west direction for the two standards used here, and no remedial work is required. A study was made to evaluate the effect of enlarging the exterior door on

*See footnote, p. II-2.

the south exterior wall of Room 115A. The study shows that the resultant slight decrease in the shear area of the south wall has no significant effect on the structure.

The steel stack was found to be designed with sufficient capacity to meet the requirements of both the UBC and the median spectrum. No remedial work is required.

Piping and supports for the GPSS, GPSR, and VERS systems were not designed for lateral forces. A scheme for bracing the piping and supports in the corridor has been developed (Figure 10), and general recommendations and a typical bracing detail are shown in Figure 11. These bracing schemes will be adequate for the two standards mentioned above and even for higher seismic criteria. As a rough estimate, \$6,000 should cover the work on these piping systems.

The holding tank supports on the roof, as existing, have a rather low structural capacity for lateral loads. This capacity can be substantially increased by adding several bracing members to the supports. This bracing, shown in Figure 12, will increase the structural capacity for the two standards used in this report and also for seismic criteria that would require forces several times higher. The piping connecting to these tanks appears to be adequate for substantial lateral loads. The cost of this bracing work probably should not exceed \$300.

The glove boxes are inherently strong structures and require no remedial work. The contents of the boxes were not included in this study and generally do not affect the boxes. The connecting piping is covered under the paragraphs on piping and supports. The possibility of flexible connections should be investigated when the remedial scheme for the building is adopted.

The exhaust duct system and the support details that are known are adequate for the two standards used above. The support details not shown are also expected to be adequate, judging from the known details. No remedial measures are required, but, when building remedial measures are determined, differential deflections that may affect the airtightness of connections of round ducts to rectangular ducts and the connection at the stack should be investigated.

The four decontamination equipment skids, in their as-built condition, need additional lateral bracing to meet the requirements of the two standards used in this study. Bracing schemes are shown (Figures 13 through 20). In some cases, alternate configurations of bracing are shown when there is a possibility of interference with the equipment and appurtenances mounted on the skid. Installation of the recommended bracing for the four skids is estimated to cost approximately \$3,500.

All the above costs are estimated current construction costs only and do not include costs incurred by Sandia due to disruption of laboratory operations and rearrangement of interior space, equipment, piping, and other additional work of such nature that may be required. If a decision is made to expend funds for this purpose, design drawings showing specific details for all the structures and systems can be prepared for the actual construction work.