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**SEISMIC ANALYSIS
LIVERMORE POOL TYPE REACTOR FACILITY,
LAWRENCE LIVERMORE LABORATORY**

R. C. Murray

F. J. Tokarz

November 15, 1973

Prepared for U.S. Atomic Energy Commission under contract No. W-7405-Eng-48

**LAWRENCE
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SEISMIC ANALYSIS

LIVERMORE POOL TYPE REACTOR FACILITY, LAWRENCE LIVERMORE LABORATORY

Abstract

A seismic evaluation of the Livermore Pool Type Reactor Facility located at the Lawrence Livermore Laboratory was conducted as part of a safety analysis being prepared for the Atomic Energy Commission. The safe shutdown earthquake for the Lawrence Livermore Laboratory site is defined, along with a brief discussion of its basis. The method

of analysis and results are summarized for all critical components.

We conclude that the biological shield and containment building should withstand the safe shutdown earthquake without failures. The crane and heat exchanger could fail under worst-case loading conditions. Recommendations for modifying the crane and heat exchanger support are presented.

Introduction

Safety Analysis Reports (SAR) are required by the U.S. Atomic Energy Commission as part of applications for construction permits and operating licenses for all nuclear power plants. The principal purpose of these reports is to inform the Commission of the nature of the facility and plans for its use. The Commission requires that the information provided be of sufficient detail to determine if the facility can be built and operated safely. The SAR is the principal document whereby the applicant provides this information.

Although the Livermore Pool Type Reactor (LPTR) is a nuclear reactor used for research, not a nuclear power reactor, the Commission has requested that the

Lawrence Livermore Laboratory prepare and submit an SAR. Such a report is in preparation. One aspect of this effort involves an seismic investigation of the LPTR facility. This report summarizes the seismic investigation.

In February 1972, the Regulatory Staff of the Commission issued a standard format for SARs.¹ This document identifies the principal detailed information that is required by the staff for its evaluation. (Previously the Commission specified the required information in general terms.) The standard format requires that each facility meet the seismic criteria defined in Appendix A of 10 CFR Part 100 entitled "Seismic and Geologic Siting Criteria for Nuclear Power Plants."

This Appendix describes the investigations required to obtain the geologic and seismic data necessary to determine site suitability and provide reasonable assurance that the facility can be constructed and operated at the site without undue risk to the health and safety of the public. Procedures are described for determining the quantitative vibratory ground motion levels for design basis and to what extent the facility need be designed for surface faulting.

When investigating possible vibratory effects, Appendix A requires that two levels of earthquake motions be considered, a Safe Shutdown Earthquake (SSE) and an Operating Basis Earthquake (OBE). The SSE is defined as the largest possible earthquake at the site. It produces the maximum vibratory ground motion that the structures, systems, and components needed for safe operation must survive, intact and functional. These structures, systems, and components are those necessary to ensure: (1) the integrity of the reactor coolant pressure boundary, (2) the capability to shut down the reactor and maintain it in a safe shut-down condition, or (3) the capability to prevent or mitigate the consequences of accidents which could result in potential off-site exposures. The OBE is defined as the largest probable earthquake at the site. It produces the vibratory motion that structures, systems and components needed for power generation must survive intact and operational.

This report considers the possible effects of vibratory ground motion produced by a SSE on the LPTR facility. The SSE used is based on the information in Refs. 2 and 3. The discussion in these

references of possible surface faulting at the Lawrence Livermore Laboratory site will not be repeated here. The LPTR facility was not investigated for a possible OBE.

All structural analyses employed the response spectra approach, using the natural periods, mode shapes and damping ratios of the structure. This approach yields conservative response estimates. Damping values equal to 2% of critical viscous damping were used for all analysis. The same damping values were used for all materials and modes of vibration. The 2% value provides a conservative estimate of the actual damping in the structures when stresses approach the yield point. In the dynamic analysis the root mean square procedure was used for combining the modal response of the reactor shield and containment building as shown below:

$$V_{\max} = \sqrt{V_1^2 + V_2^2 + \dots + V_n^2}$$

where: $V_1, V_2, \dots, V_n$ = modal response quantities (i.e., displacements, accelerations, stresses) $V_{\max}$ = maximum value of quantity.

Yield and ultimate stresses of the construction materials were used for comparison with calculated stresses. Displacement criteria were based on physical interference between adjacent components.

The contents of this report include:

- A brief description of the LPTR facility
- A definition and discussion of the basis for the SSE

- A discussion of the possibility of liquefaction and the effects of soil-structure interaction
- A brief discussion of the method of analysis and results for the reactor shield, the containment building, the overhead bridge crane, the reactor top structure, and the heat exchanger support structure.
- Conclusions regarding the structural integrity of the LPTR facility when exposed to ground motions defined by the SSE.

Description of LPTR

The LPTR is a tank-type thermal research reactor. The light-water moderated and cooled reactor core generates a peak flux approaching 10^{14} n/cm²·sec at the present operating power of 3MW. Fuel used in the reactor is enriched with ²³⁵U.

The reactor facility is a versatile and flexible tool for the various research programs at the Laboratory. Six horizontal beam tubes, two thermal columns and two pneumatic tubes plus a number of irradiation ports and in-core facilities provide access to the flux. Figure 1 shows a section through the reactor.

The LPTR was designed, constructed and tested by the Foster Wheeler Corporation. Construction began in October 1956 and was completed April 1958. From 1958 to 1960 the reactor was operated at 1 MW. Modification of the containment building equipment to allow increased operating power began in 1960. In January 1967 the maximum power level was increased to 3 MW. Reference 4 presents the final design details and modifications up to 1967, a description of the reactor, related mechanical and electric control systems and a discussion of operating procedures and associated hazards.

Figure 2 shows a cross-section through the reactor building identifying those structures considered in the analysis. These include the containment building, the reactor shield, reactor top structure, the bridge crane and the heat exchanger. A brief description of each follows:

CONTAINMENT BUILDING

The reactor is housed in a controlled-leakage steel building designed to contain radioactive effluents under all conditions. The building is 80 ft in diameter and 52 ft high. The building is a cylindrical shell of welded steel plates 1/4 in. thick reinforced with stiffener rings and has a domed self-supporting roof of 5/16 in. steel plates. Large bars buried in a massive reinforced concrete ring wall anchor the base of the structure.

The containment building was designed and tested for an internal pressure of 2 psi. In addition, the design considered a 12 lb/ft² wind loading and lateral earthquake loading per the 1952 Uniform Building Code with a seismic coefficient of 0.2 (i.e., base shear equals 0.2 times the total seismic weight).

Fig. 1. Livermore pool type reactor, N-S sectional view.

REACTOR SHIELD

The reactor shield consists of an aluminum tank $3/8$ in. thick, 79 in. diameter, and 26.5 ft deep surrounded by a concrete biological shield as shown

in Fig. 1. Magnetite concrete with a unit weight of 225 lb/ft^3 surrounds the fuel elements while standard concrete fills the upper shield section. The top of the reactor is enclosed by a $1/4$ -in. thick removable working platform. The plates

Fig. 2. LPTR reactor building, E-W section facing north.

rest on cantilevered wide-flange beams which support the control rod actuator housing (top structure).

The shield was also designed to comply with the 1952 Uniform Building Code, using a seismic coefficient of 0.133 and a wind loading of 20 lb/ft^2 normal to vertical surfaces.

OVERHEAD BRIDGE CRANE

A 15-ton traveling bridge crane services the main area of the reactor facility. The crane has a maximum hook height of 37 ft clearing the highest pooltop structure by about 4 ft. The crane trolley and hoist have variable-speed motors, with a low-speed range and magnetic brakes.

The crane support structure is a welded and bolted structural steel frame.

Design loads for the crane structure were dead load, increased 25% for vertical impact due to crane operations, and lateral loads from either crane accelerations or from seismic considerations (UBC, Zone 3, seismic coefficient = 0.2).

REACTOR TOP STRUCTURE

The reactor top structure houses the control rod actuators and also covers the top of the reactor pool. The structure is cantilevered from the concrete reactor shield. The top structure was designed according to the 1952 Uniform Building Code. Seismic loads were not considered.

HEAT EXCHANGER

The heat exchanger is part of the reactor primary coolant system and is

located in the equipment pit. The heat exchanger is supported by a rigid frame fabricated from angles and channels.

Safe Shutdown Earthquake

The safe shutdown earthquake (SSE) is defined as the largest possible earthquake for the site. This is the earthquake that safety related reactor components must tolerate and still remain functional. Determination of the SSE and its local ground motion requires investigations into the site geology, seismicity, and tectonics. The SSE used for this investigation is based on a postulated 5.7 magnitude earthquake located on the Tesla fault at a 2.5 mile epicentral distance to

the Laboratory. The SSE gives an estimated peak horizontal acceleration of 0.5 g and a vertical acceleration of 0.33 g at ground level. The frequency content of the SSE is defined by the response spectra shown in Fig. 3. The SSE used is taken from Ref. 2, a project to develop seismic criteria for a new plutonium facility located 1400 ft from the LPTR site. The investigation included gravity, magnetic, geodetic, hydrologic, and seismic refraction surveys.

Reference 2 concerns itself primarily with an investigation of faulting in the vicinity of the Laboratory. An earlier investigation³ developed seismic design criteria without particular reference to the reactor siting criteria of the AEC. The seismic criteria presented in Ref. 3 were based on postulated large earthquakes located on more distant faults to the west of the Laboratory. These faults and their relation to the LLF site are shown in Fig. 4, with maximum postulated earthquakes listed in Table 1.

Fig. 3. Safe shutdown earthquake (SSE) spectra. For vertical component multiply acceleration scale by 2/3. For operating basis earthquake (OBE) multiply acceleration scale by 1/2.

Table 1. Maximum postulated distant earthquakes, Ref. 3.

Fault	Magnitude	Epicentral distance (mi)
San Andreas	8.3	35
Hayward	7.5	17
Calaveras	7.0	10

Fig. 4. Important faults and earthquake epicenters located within 60 mi of L.P.T.R.

The estimated peak ground acceleration from any of these earthquakes was 0.4 g at the Laboratory site.

Another report⁵ recommends a higher level of horizontal ground acceleration of 0.8 g and slightly different design

spectra. However, after a careful review of both works the Laboratory has decided to use the 0.5 g SSE as

recommended in Ref. 2 for the evaluation and design of the Laboratory's critical facilities.⁶

Liquefaction Potential

The development of the SSE criteria assumes the subsurface soil conditions at the Laboratory site to be stable. This section presents arguments to justify this assumption.

Liquefaction describes the condition of soil instability that can occur in saturated cohesionless materials subjected to vibrations. During vibration the soil tends to compact, increasing the pore-water pressure. The increased water pressure causes an upward flow that liquifies the sand above. This can make structures founded on the site settle or tilt.

Liquefaction occurs in fine sands located below the ground water table. The sands must be loose and able to compact. Relative density (D_r) is a measure of the sand's state of compaction. A sand with a D_r of 100% is at its densest. Also, in order to liquify, there must be a path through the sand for the upward flow of water. Without low D_r and a path for water flow, no potential liquefaction problem exists.

The subsurface soil conditions at the ELL site are summarized in Fig. 5 (Ref. 7). The only saturated material is the Livermore formation. Since this

Fig. 5. Generalized soil properties at Livermore site.

material is coarse and has a relative density of 100%, it will not compact under vibration. Even if it were able to compact, the 40-ft clay blanket above it would prevent the upward flow of pore water needed for liquefaction. Thus neither of the conditions for liquefaction exist.

A worst case examination was conducted, using the soil profile shown in Fig. 6. This profile meets the necessary conditions for liquefaction. Then we analyzed this profile using a procedure developed by Seed and Idriss.⁸

The procedure is based on the concept that shear stress developed in a soil

Fig. 6. Assumed worst case soil profile at LPTR site.

$a_{\max}$ = ground acceleration

γ = unit weight of soil

h = depth

$$(\tau_{\max})_{\text{rigid}} = \left(\frac{\gamma h}{g}\right) a_{\max}$$

g = acceleration of gravity

Fig. 7. Rigid soil column subjected to ground acceleration.

deposit are due to an upward-propagating shear wave. First, maximum stresses induced by the earthquake are calculated based on a rigid soil column as shown in Fig. 7. The shear stress is then corrected to account for the deformable nature of the soil by introducing the stress reduction factor r_d . Laboratory tests show the average shear stress to be $\approx 65\%$ of the maximum shear stress. Hence the average shear stress induced in the soil deposit during an earthquake is

$$(\tau_{av})_{\text{deformable}} = 0.65 \left(\frac{\gamma h}{g} a_{\max}\right) r_d \quad (1)$$

Next, the stresses needed to cause liquefaction must be calculated. Cyclic loading triaxial compression test data was used to obtain a laboratory stress ratio at liquefaction as a function of the grain size of the sand. These tests were all conducted at a relative density of 50%. The laboratory values are then corrected for relative density (D_r) and related to field data with a correction factor C_r .

The stress ratio in the field is then given by the equation

$$\left(\frac{\tau}{\sigma_o'}\right) = (\text{stress ratio})_{\text{lab}} C_r \frac{D_r}{50}$$

where

τ = shear stress needed for liquefaction
 σ_o' = initial effective overburden pressure,

Thus the stresses needed to cause liquefaction may be estimated by

$$\tau = \sigma_o' (\text{stress ratio})_{\text{lab}} C_r \frac{D_r}{50} \quad (2)$$

Shear stresses are then evaluated as a function of depth from Eqs. (1) and (2) and plotted. If an overlap occurs in the

Fig. 8. Evaluation of soil liquefaction potential (worst case) with ground water table at 100 ft.

plot a potential liquefaction problem exists at the site. These stresses are plotted in Fig. 8. Since there is no overlap, no potential for liquefaction exists.

Based on the current knowledge of the soil conditions and the location of the ground water table at the LLL site, liquefaction is not a problem.

Soil-Structure Interaction Effects

The structural analyses to be discussed in the following sections are based on an analytical model that assumes a fixed base condition at ground level. The purpose of this section is to determine whether including the soil beneath the facility affects the calculated response of the reactor shield.

Soil-structure interaction generally affects both the horizontal and vertical motions at the base of the structure. One effect of the underlying soil will probably be to reduce the shield stresses, mainly due to the high damping provided by the soil. A more important aspect is the possible introduction of rocking and translation of the structure, a topic that will occupy the remainder of this section.

The reactor shield was assumed to be rigid and the effect of the soil was modeled as a translational and torsional spring. The values of the spring constant were determined from results derived for

elastic half-spaces.⁹ The following equations were used for the translational and torsional spring constants:

$$K_x = 2(1+\nu)G\beta_x \sqrt{BL} = \text{Translational spring constant} \quad (3)$$

$$K_\phi = \frac{G}{1-\nu}\beta_\phi BL^2 = \text{Torsional spring constant} \quad (4)$$

where:

G = Shear modulus of soil

ν = Poisson's ratio of soil

B = Effective foundation width
(1 to applied force)

L = Effective foundation length
(1 to applied force)

β_x, β_ϕ = Parameter determined as a function of L/B . (Values of β_x and β_ϕ are shown in Fig. 9.)

Soil properties used to determine the spring constant are $G = 5 \times 10^6$ lb/ft², obtainable from the site soil properties, and an assumed Poisson's ratio of $\nu = 0.1$.

The effect of earthquake loading was modeled by placing a lateral load at the center of gravity of the shield. The magnitude of the lateral load was determined from $F = Ma = 0.5 Mg = 0.5 W$, where

$a = 0.5 g$ = peak acceleration from SSE spectra for calculated period of fixed base model ($T = 0.020$ sec.)

Fig. 9. Coefficients β_x and β_ϕ for rectangular footing (from Ref. 9).

g = acceleration of gravity

M = mass of shield

$W = Mg$ = weight of shield.

A static analysis of the model shown in Fig. 10 results in expressions for the maximum rocking and maximum translational displacement of

$$\Delta_{\text{max}}^{\text{rocking}} = \frac{(0.5W)(H)(h+t)}{K_{\phi}} \quad (5)$$

$$\Delta_{\text{max}}^{\text{translational}} = \frac{0.5W}{K_x} \quad (6)$$

Estimates of maximum displacements due to rocking and translation in the North-South and East-West directions were conducted. Results appear in Table 2. Since these displacements were within acceptable limits no further analysis was necessary.

Estimates of the fundamental period of vibration of the reactor shield with rocking and translation were made by applying Dunkerley's Formula^{10,11}

$$\tau = \sqrt{T_{fb}^2 + T_t^2 + T_r^2} \quad (7)$$

where

T = Fundamental period of shield with soil-structure interaction

T_{fb} = Fundamental period of fixed base model

T_t = Fundamental period due to ground translation alone

T_r = Fundamental period due to ground rocking alone

The periods of ground translation and rocking can be expressed as¹⁰

$$T_t = 2\pi \sqrt{\frac{W}{g} \cdot \frac{1}{K_x}} \quad (8)$$

Fig. 10. Model of reactor shield for estimating rocking and translational displacement.

Table 2. Estimates of maximum lateral displacement of reactor shield due to rocking and translation.

Earthquake direction	Effective foundation dimensions for rocking ^a		Effective foundation dimensions for translation ^b		β_{ϕ}	K_{ϕ} (ft-lb/radian)		K_x (lb/ft)	$\Delta_{\text{max}}^{\text{rocking}}$ (in.)	$\Delta_{\text{max}}^{\text{translation}}$ (in.)
	B(ft)	L(ft)	B(ft)	L(ft)		β_x				
North-South	30	20	43	30	0.45	45.0×10^9	1.0	6.0×10^8	0.044	0.008
East-West	20	30	30	45	0.55	82.5×10^9	1.0	6.0×10^8	0.025	0.008

^a Actual concrete pad dimensions.

^b 1.5 times actual concrete pad dimensions. These dimensions were increased due to the confinement provided by adjacent foundations of other structures within the containment building.

$$T_r = 2\pi \sqrt{\frac{W}{g} \cdot \frac{1}{82.8 K_x}} \quad (9)$$

where:

$$\frac{W}{g} = \text{mass of reactor shield}$$

The calculated period with ground interaction was $T = 0.045$ sec. Examination

of the SSE spectra shows that this increase in fundamental period causes no dynamic amplification. This justifies the assumption of peak ground acceleration acting on the reactor shield for rocking and translational displacements and the use of a fixed based model for seismic analysis.

Reactor Shield Analysis and Results

The reactor shield, shown in Fig. 1, was analyzed using the finite element method. The analysis was performed using the computer code, GHOSH.^{12,13} The shield was modeled as an axisymmetric structure about its weak axis. Figure 11 shows the finite element mesh used. Both the standard and heavy concrete were represented as solid elements. The aluminum pool tank was idealized with shell elements. The shield was considered rigidly connected to the foundation. The materials properties used in the analysis^{14,15} appear in Table 3.

The model was subjected to dead load, and horizontal and vertical components of the SSE earthquake loading. Hydro-

static pressure and hydrodynamic effect of the water in the pool tank were neglected.

The ten lowest fundamental modes of vibration were used for horizontal response. Only five modes were needed to calculate the vertical response. Table 4 gives the first three periods of vibration for both horizontal and vertical responses.

The results from both the dead load and the earthquake loads are plotted in Figs. 12-17. Figures 12 and 13 show maximum calculated horizontal and vertical displacements of 0.0075 and 0.005 in., respectively, at the top of the shield. Figures 14 and 15 show the shield in its maximum deflected position (magnified 4000 times) for both the

Table 3. Material properties used for shield analysis.

Material	Modulus of elasticity ^a (10 ⁶ psi)	Poisson's ratio	Unit weight (lb/ft ³)	Mass density $\left(\frac{10^{-4} \text{ lb-sec}^2}{\text{in.}^4}\right)$
Standard concrete	3.30	0.17	150	2.25
Heavy concrete	5.00	0.20	225	3.37
Aluminum	10.6	0.33	—	2.59

^aTypical design values were used for the modulus of concrete. These values were not increased to simulate improvement of properties with age.

Fig. 11. Computer model of LPTR shield and aluminum tank.

Table 4. Lowest vibration periods of reactor shield.

Mode	Period sec
Horizontal	
1	0.0200
2	0.0091
3	0.0048
Vertical	
1	0.0073
2	0.0038
3	0.0033

Fig. 12. Lateral displacement at outer surface of LPTR shield, horizontal and vertical SSE.

horizontal and vertical component of the SSE loading. Figures 16 and 17 give the variation of the calculated maximum stresses with height in the reactor shield. A maximum vertical compressive stress of 158 psi was calculated at the interface between standard and heavy concrete. This is well below the 3000 psi compressive strength of the concrete.

The maximum shear stress in the shield is 50 psi. (See Fig. 17.) The allowable shear stress for concrete having a compressive strength of 3000 psi is 60 psi.¹⁶

Tensile stress were assumed carried solely by the steel reinforcement. Details of the reinforcement in the shield appear in Fig. 18. Regions of large

Fig. 13. Vertical displacement at outer surface of LPTR shield, horizontal and vertical SSE.

vertical stress occur at the connection to the foundation and at the standard concrete-heavy concrete interface. An overlap of reinforcement in these areas provides for these stress concentrations. The maximum tensile steel stresses throughout the shield are always below ultimate values for A-15 hard grade billet steel. (Yield stress $\approx$ 50 ksi, ultimate stress 80 ksi.) Typical values of the steel stresses are also shown in Fig. 18.

The displacements resulting from the earthquake motions are very small. The concrete alone has adequate strength to carry the shear and compressive stresses. No yielding of the horizontal steel and some yielding of the vertical steel is

expected to result from the combined deadload and earthquake loading.

The vertical stress distributions at the reactor shield foundation due to dead load and earthquake loading are shown in Fig. 19. The distributions due to the dead load and the vertical earthquake component are axisymmetric; the distribution due to the horizontal component

Fig. 14. Deformed shape of LPTR shield and aluminum tank, using rms approximation, resulting from horizontal component of SSE. Displacement amplification, $\times 4000$. (Dots indicate un-stressed position.)

Fig. 15. Deformed shape of LPTR shield and aluminum tank, using rms approximation, resulting from vertical component of SSE. Displacement amplification, $\times 4000$.

Fig. 16. Maximum normal stress at each elevation of LPTR shield due to combined dead load and horizontal and vertical components of SSE (rms approximation).

Fig. 17. Maximum shear stress at each elevation of LPTR shield due to combined dead load and horizontal and vertical components of SSE (rms approximation).

Properties of A15-50 reinforcing steel

- $\sigma_{\text{yield}} = 50 \text{ ksi}$
- $\sigma_{\text{ultimate}} = 80 \text{ ksi}$

Fig. 13. Reactor shield reinforcement details and calculated steel stresses.

varies as the cosine of the angle between the earthquake direction and the position of interest.

Figure 20 shows the maximum combined compressive stress distribution. These maximum values assume that both earthquake components combined with the dead load to give the largest compressive values. A maximum compressive stress of 200 psi was obtained. This is well below 3000 psi, the crushing strength of the concrete.

The maximum earthquake and dead load stresses were also combined in such a way to produce the maximum tension stresses at the reactor shield base. (See Fig. 21.)

An array of No. 6 bars distributed over its base ties the shield to the foundation. Twenty-two bars are evenly spaced at a 43-in. radius and 68 at a 96-in. radius. These bars are assumed to carry all of the tensile stress at the base.

In order to simplify the calculation and still arrive at a reasonable evaluation

Fig. 19. Axial stress distribution at base of LPTR shield due to dead load and SSE components.

of the ability of these bars to resist tensile stresses, the average stress distribution shown in Fig. 21 was assumed. The assumption that this stress distribution is axisymmetric yielded tensile stresses of 55 and 65 ksi on the inner and outer circles of bars. These bars have a yield strength of 50 ksi and an ultimate strength of 80 ksi.

The ability of these bars to develop sufficient bond was also checked. The results indicated a maximum bond strength of 923 psi with a maximum bond stress

of 900 psi.¹⁶ Since there is adequate bond some yielding of the bars would take place if both earthquake components combined in this worst case situation.

The maximum shear stress in the concrete at the base is 34 psi from Fig. 17. The base shear force was computed on the assumption that this shear force is distributed equally over all 90 bars. This results in a shear stress of 29.5 ksi in each bar. This value is slightly greater than the shear yield stress ($\tau_{\text{yield}} = 50 \text{ ksi}/\sqrt{3} = 29 \text{ ksi}$). Thus some yielding of the bars is expected.

Fig. 20. Maximum compressive stress distribution at LPTR shield base, i.e., sum of horizontal, vertical, and dead loads.

Fig. 21. Maximum tension stress distribution at LPTR shield base, i. e., horizontal plus vertical minus dead load.

Other means of transferring the shear load at the base include mechanical interference due to the key and friction between

the foundation and shield. Since these were not considered in the analysis, the above results are conservative.

Containment Building Analysis and Results

We conducted a linear elastic analysis of the containment building using the finite element method. The building was represented as the axisymmetric structure shown in Fig. 22 and subjected to dead load, 2 psi internal pressure (which could result from atmospheric conditions), and horizontal and vertical components of the SSE. The computer program GHOSH^{12,13} was used.

A finite element mesh consisting of shell elements was used to model the

steel structure. Solid elements connected to the shell at five locations simulated the effect of stiffeners in the actual structure. The mass density of the steel was increased to include the effect of insulation weight. The building was considered rigidly connected to its foundation.

Shell thicknesses were varied to model the different thickness of the shell and roof. Lap joints were included by increasing the thickness at the joints.

Fig. 22. Computer model of LPTR containment building.

Fig. 23. Deformed shape of LPTR containment building, 2 psi internal pressure. Displacement amplification, $\times 100$. (Dots indicate unstressed position.)

The compression ring located at the roof connection was included as a thicker shell, locally stiffening the shell-roof connection.

Ten modes of vibration were used to determine both horizontal and vertical response to the earthquake loading. The first three periods appear in Table 5.

Figure 23 shows the displaced shape of the structure, amplified 100 times, when subjected to dead load and 2 psi internal pressure. Figures 24 and 25 show horizontal and vertical displacements

Fig. 24. Lateral component of displacement of LPTR containment building under 2 psi internal pressure.

due to the pressure loading. Maximum horizontal and vertical displacements are 0.090 in. and 0.075 in. respectively.

Stresses in the shell and roof for the pressure loading are well within allowable design values for ASTM A-283 steel plates. Compare the design values of yield strength (33 ksi) and ultimate strength (55 ksi) with the calculated stresses shown

Table 5. Containment building lowest periods of vibration.

Mode	Period (sec)
Horizontal 1	0.055
2	0.042
3	0.041
Vertical 1	0.065
2	0.040
3	0.032

in Table 6. Furthermore, the structure has been tested to an internal pressure of 2 psi.

Figures 26 and 27 show the displaced shape for the horizontal and vertical components of the SSE. These shapes have been amplified by factors of 320 and 600 respectively.

Figures 28 and 29 show the horizontal and vertical displacements combined for both components of the SSE. A maximum lateral displacement of 0.135 in. and a maximum axial displacement of 0.156 in. were calculated.

Stresses induced by the earthquake components are approximately 10% of the stresses resulting from the pressure loading. No problems are expected in the containment building.

Table 6. Shell and roof stresses.

Altitude (in.)	Hoop stress (ksi)			Axial stress (ksi)		
	Static loads ^a	Earthquake		Static loads ^a	Earthquake	
		Horizontal	Vertical		Horizontal	Vertical
Shell						
45	3.88	0.025	0	2.00	0.500	0.165
130	3.88	0.050	0	1.98	0.425	0.165
214	3.86	0.065	0	1.96	0.365	0.150
299	3.86	0.090	0	1.94	0.290	0.150
385	3.89	0.100	0	1.88	0.240	0.150
Roof						
476	7.47	0.150	0.450	5.43	0.515	0.390
535	6.38	0.540	0.415	5.93	0.525	0.525
597	5.44	0.390	0.475	3.18	0.225	0.290
621	2.84	0.525	0.375	1.45	1.390	0.475

^a Dead load plus 2 psi internal pressure.

Steel Plates (ASTM-A283)

$\sigma_{\text{yield}} = 33 \text{ ksi}$

$\sigma_{\text{ultimate}} = 55 \text{ ksi}$

Fig. 25. Axial component of displacement of LPTR containment building under 2 psi internal pressure.

Fig. 26. Deformed shape of LPTR containment building, using rms approximation, resulting from horizontal component of SSE. Displacement amplification, $\times 320$.

Fig. 27. Deformed shape of LPTR containment building, using rms approximation, resulting from vertical component of SSE. Displacement amplification $\times 600$.

Fig. 28. Lateral component of displacement of LPTR containment building from combined horizontal and vertical components of SSE (rms approximation).

Fig. 29. Axial component of displacement of LPTR containment building from combined horizontal and vertical components of SSE (rms approximation).

Other Category I Structural Analyses and Results

OVERHEAD BRIDGE CRANE SUPPORT STRUCTURE

The first step of the analysis was the determination of maximum hook loading. We considered only weights (masses) that would be lifted to the top of the overhead bridge structure, neglecting masses lifted only slightly from the floor. The maximum expected crane hook load was determined from the actual past loading history. Although the crane has a capacity of 30 kip, past records of crane usage indicate the maximum load raised to the top of the structure is 7 kip. This load occurs three times annually for short durations. Therefore, for the

analysis of the crane support structure a 7-kip live load was selected.

Step 2 was the determination of the worst case combination of hook position and earthquake direction, using the computer code TABS.^{17,18} The analysis indicated two worst case loading conditions, the structural and the nuclear. Case 1 (structural) locates the hook near the one quarter span point with the critical earthquake direction in the plane of the long span truss. Case 2 (nuclear) has the hook at midspan directly over the reactor pool tank. These loading conditions are shown on Fig. 30. Results also indicate that a planar analysis of the crane support structure in the long span direction would

Fig. 30. LPTR bridge crane structure, showing location of worst case live loads.

adequately represent the actual structural behavior.

The remainder of the analysis consisted of a detailed elastic analysis of the long span frame with dead loads plus the 7 kip maximum expected live load. The crane support was modeled as a plane frame. The model used for this analysis is shown in Fig. 31. Properties of A-7 structural steel used in the analysis were $E = 30,000$ ksi, yield stress = 33 ksi, ultimate stress = 60 ksi. Examination of the member connection details suggested semirigid internal joints and pinned column connections¹⁹ at the base.

The frame analysis was conducted with the finite element program²⁰ FRAME2.

An estimate of the lateral stiffness of the frame, K , was determined as 14.9 kip/in. The structure's fundamental period of vibration, T , was calculated by considering the frame as a single-degree-of-freedom system with its mass lumped at the upper level. Spectral accelerations were obtained from Fig. 3 for 2% damping as a function of period. The response spectrum method was then used to determine the maximum lateral displacement.

Results of these calculations for both load cases are collected in Table 7.

Member forces were then determined by conducting a static analysis of the plane frame subjected to the maximum lateral displacement $\Delta_{\max}$ and the loading conditions stated previously.

Fig. 31. Plane model of LPTR crane structure used in analyzing the response to the SSE under the two worst case loading conditions.

Conclusions

Lateral displacements of the structure are acceptable. (See Table 7.) A 5.5-in. clearance between the crane support structure and the containment building walls now exists.

Examination of the member forces for Case 1 indicates the anchor bolts at the pinned base plate connection are overstressed, extreme fiber stresses in the columns exceed ultimate values, and some secondary compression struts are overstressed. AISC allowable stresses in combined tension and shear for A-307 bolts were used to evaluate the anchor bolts.

For Case 2 the anchor bolts exceed the allowable shear stresses and some

secondary struts are overstressed.

Ultimate stresses are not exceeded under this loading condition.

Recommended Modifications

Cover plates were designed that would reduce the stresses in the columns and braces during the earthquake loading. Figure 32 shows the details of the modification needed. A repeat analysis for Case 1 loading showed that the modified structure has a lateral stiffness of 36.1 kip/in. and a fundamental period of 0.27 sec.

At 2% damping a spectral acceleration of 772 in./sec^2 and a maximum lateral displacement of 1.5 in. were used as the earthquake loading conditions. For the

Table 7. Summary of crane calculations.

Loading condition	Dead load plus max. expected load W (kip)	Mass $M = \frac{W}{g}$ ($\frac{\text{kip-sec}^2}{\text{in.}}$)	Period, T (sec)	Spectral acceleration, S_a ($\frac{\text{in.}}{\text{sec}^2}$)	Max. lateral displacement $\Delta_{\text{max}} = \frac{MS_a}{K}$ (in.)
Case 1	$19.64^a + 7 = 26.64$	0.069	0.43	733	3.4
Case 2	$19.64^a + 3.50 = 23.14$	0.060	0.40	772	3.1

^aFrame weight = 14.64 kip, crane hoist and trolley weight = 5.0 kip,
Total dead load = 19.64 kip.

structure modified as shown, all member and anchor bolt stresses were below acceptable values.

REACTOR SHIELD TOP STRUCTURE

Figure 33 depicts the reactor shield top structure housing the control rod actuators and covering the top of the reactor pool. The structure is cantilevered from the concrete shield by two 5WF18.5 beams connected to the reactor shield by eight hooked anchor bolts 0.75 in. diam $\times$ 20 in. long embedded in the concrete.

The top structure was modelled as a single-degree-of-freedom system, i.e., a lumped mass cantilevered from the reactor shield. A conservative estimate of 2000 pounds was made for the equipment and housing of the structure. The weight of the cantilever beams was neglected. The spectral response method was used to determine stresses and deflections. It was assumed that the SSE spectrum, transmitted unaltered by the shield, was applicable as a direct input for analysis.

The response of the top structure to both horizontal and vertical components of the SSE was computed. Results of the analysis appear in Table 8.

Table 8. Summary of top structure calculations.

Earthquake loading	Cantilever beam stiffness (kip/in.)	Fundamental period (sec)	Maximum displacement (in.)
North-South	354	0.024	0.0028
East-West	8175	0.005	0.0001
Vertical	71	0.934	0.019

Analysis of the anchor bolt stress indicates they are well below allowable design values. A maximum tension stress of 9.07 ksi and maximum shear stress of 2.115 ksi for a typical anchor bolt was calculated. Allowable design values used for A-307 bolts under combined tension and shear loadings are 14 ksi in tension and 10 ksi in shear.

HEAT EXCHANGER SUPPORT STRUCTURE

An analysis procedure similar to that conducted on the crane support structure was conducted on the heat exchanger support. The support structure was modeled as a pin ended frame with rigid joints. The stiffening effect of the connecting piping and the dead load of the structure were neglected in the analysis. It was felt that a plane frame loaded as

Fig. 32. Suggested modifications to the LPTR crane support structure to improve resistance to lateral earthquake loads.

shown in Fig. 34 would adequately represent the three-dimensional structure. The total heat exchanger load of 10 kip was distributed as shown on each of two frames.

The structure lateral stiffness was determined as 3.5 kip/in. Other calculated values include a mass of $0.013 \text{ kip-sec}^2/\text{in.}$

and fundamental period of 0.38 sec. From Fig. 3 at 2% damping a spectral acceleration of 790 in./sec^2 was determined which gives a corresponding maximum lateral displacement of 2.94 in.

Member forces were then determined by conducting a static analysis of the frame subjected to the heat exchanger dead load

Fig. 33. Reactor shield top structure.

and the lateral displacement due to the earthquake.

Conclusions

Anchor bolt stresses exceed allowable design values and extreme fiber stresses in both beam and columns exceed ultimate values for A-7 structural steel.

The deficiencies in the structure can be corrected by adding cross bracing to the legs to increase the lateral stiffness of the support.

Recommended Modification

An analysis was conducted with cross bracing to determine an adequate member size to reduce stresses to allowable values. The cross bracing was rigidly connected to the support legs and assumed to resist tension forces only (no compression).

Eight 5 in. $\times$ 1/2 in. steel bars welded to the support legs as shown in Fig. 35 are adequate to brace the structure for

Fig. 34. LPTR heat exchanger support structure and plane frame model used to analyze structure's response to the SSE.

the earthquake loading. Calculated values for stiffness, period, spectral acceleration and maximum lateral displacement are 91.74 kip/in., 0.07 sec, 463 in./sec², and 0.065 in. respectively. For this modification all member and bolt stresses are below allowable design values for A-7 structural steel.

Fig. 35. LPTR heat exchanger support structure crossbracing scheme to improve resistance to lateral earthquake loads.

Conclusions

Based on the results of the analyses several conclusions have been reached about the seismic behavior of the LPTR facility if subjected to the earthquake loading defined by the SSE.

Maximum reactor shield displacements during an earthquake are within acceptable values. The concrete alone has adequate strength to carry the shear and compressive stresses induced by the SSE; some yielding of the vertical reinforcing steel is expected. There are adequate mechanisms to transfer the seismic shear to the foundation. No problems are anticipated for the reactor top structure.

The existing crane and heat exchanger support structures are presently inadequate to resist the SSE. Severe damage or collapse of the heat exchanger support and the overhead bridge crane frame is expected if fully loaded at the time of earthquake occurrence. These deficiencies can easily be corrected by adding bracing and cover plates to the effected members.

Earthquake loading will stress the containment building to approximately 10% of the 2 psi design internal pressure loading. The building has been proof tested at this internal pressure loading. No problems are expected in the containment structure.

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