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Title: Results of RLUOB Seismic Study - With Updated Conclusions

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memorandum

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Subject: Results of RLUOB Seismic Study – With Updated Conclusions

Purpose

The purpose of this memorandum is to present the results of the reanalysis of the Radiological Laboratory Utility Office Building (RLUOB) to the current seismic hazard at TA-55. Conclusions and recommendations follow on page 5.

Revision 1 of this document updates the conclusions and recommendations based on additional study relating to the noted deficiencies.

Background

The RLUOB was constructed as part of the Chemistry and Metallurgy Research Replacement (CMRR) project at Los Alamos National Laboratory (LANL). Structural design of the facility was performed by Carter-Burgess under contract to Austin Commercial Contractors, LP. The facility design was started in November of 2005 and construction documents were issued in the spring of 2007. Construction of the facility started in the spring of 2007 and was substantially completed by 2010.

Some of the primary design parameters that form the basis for the Code of Record are:

1. Structure categorized as PC-2 per DOE Standard 1020-1994.
2. LANL ESM, Chapter 5, Section II (Rev. 2/9/04): $S_{DS} = 0.54g$, $S_{D1} = 0.26g$.
3. IBC 2003/ASCE 7-02 Seismic Use Group III; Importance Factor = 1.5.
4. Steel design per AISC Manual of Steel Construction, LRFD, 3rd Ed.
5. Seismic Steel design according to AISC 341-05[1] and AISC 358-05.
6. Concrete Design according American Concrete Institute (ACI) 318-02.

Los Alamos National Security (LANS) is considering increasing the amount of hazardous material that may be processed or stored within RLUOB. Additionally, the seismic hazard at Los Alamos has been

[1] A variance was granted for the use of American Institute of Steel Construction (AISC) 341-05 and AISC 358-05 so that the prequalified connections in 358 could be used.

redefined in two probabilistic seismic hazard reports[2] [3]. The results of the later seismic hazard report indicate that the design basis ground motion is higher than previously used.

Because of programmatic needs and an increase in the seismic hazard, LANS commissioned a seismic study of RLUOB to determine if the structure could meet the current seismic requirements for both a Seismic Design Category 1, and a Seismic Design Category 2 structure. DOE-STD-1020-2012, DOE-STD-1189, and ANS Standard 2.26 provide a definition of Seismic Design Categories. This letter report presents the results of that study.

Description of the Structure

The RLUOB facility consists of two separate buildings[4]; a laboratory/office building referred to as the MAIN building and a utility building referred to as the CUB (Central Utility Building). These two structures share spread footings on column line F between grids 2 and 8 (see Figure 1), but are otherwise structurally isolated from one another.

The Main building footprint is approximately 40,500 square feet. It consists of a basement and four floors. The Main building is buried up to the first floor on the south, east, and north sides. Along the west side, where the CUB is located, there is no retained soil from grid 2 to grid 8. Between grid 8 and 13 the structure is buried up to the second floor. Area E (4,000 square feet) extends from the west wall between grids 8.8 and 11 to grid H.3. The base of Area E is the same elevation as the first floor. The roof of Area E is approximately six feet below the second floor top-of-slab elevation and supports a soil load up to that level.

The structure is made up of two different gravity/lateral force resisting systems. Below the second floor the structure is supported by reinforced concrete gravity columns with special reinforced concrete shear walls and reinforced concrete floor slabs. Spread footings support the concrete columns. Post-tensioned beams support the second floor slab over the laboratory spaces. Above the second floor, the structure consists of special steel moment frames with light-weight concrete on metal deck composite floor slabs and metal roof deck diaphragms. The following structural features create challenges for structural analysis:

1. A large atrium interrupts the diaphragms above the second floor.
2. Multiple, stepped, roof levels.
3. A mezzanine level in the basement. The mezzanine is a steel frame with light-weight concrete composite flooring supported by the concrete columns.
4. An overhead service carrier (OSC) installed over the laboratory spaces. The OSC is made up of steel framing supported by the concrete columns and by hanger assemblies attached to the second floor.

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[2] URS Corporation, "Update of the Probabilistic Seismic Hazard Analysis and Development of Seismic Design Ground Motions at the Los Alamos National Laboratory," for Los Alamos National Laboratory, 15 May 2007.
[3] URS Corporation, "Update of the Probabilistic Seismic Hazard Analysis and Development of CMRR Design Ground Motions, Los Alamos National Laboratory, New Mexico," for Los Alamos National Laboratory, 4 December 2009.
[4] Refer to Figure 1 for a plan view of the structure

5. A re-entrant corner irregularity created by Area C. The area also has a curved roof.
6. Stepped foundation walls.
7. Steel moment frames terminating at the second floor diaphragm.

Analysis

A modified seismic margins approach was used to determine the seismic capability of the structure. Using a seismic margins approach, the seismic performance of the structure may be determined. The seismic performance can then be compared to target performance goals in DOE-STD-1020. The use of alternate methods, such as the seismic margins approach is permitted in both DOE-STD-1020 and in the 2009 International Building Code (by incorporation Section 12.6 of ASCE 7-05). The performance goals for Seismic Design Category 1 and Seismic Design Category 2 structures are provided in ASCE-43, "Seismic Design Criteria for Structures, Systems and Components in Nuclear Facilities" and are reproduced in Table 1.

The seismic margins approach follows the Conservative Deterministic Failure Margins methodology of EPRI NP-6041 SL5. In this approach, best estimates of non-seismic demand are combined with slightly conservative estimates of seismic demand, conditioned on a review level earthquake occurring. In the reanalysis of RLUOB, the review level earthquake was equal to the 2,500 year ground motion or a PC-3 earthquake. Given the non-seismic and seismic demand, the margin on capacity above the review level earthquake may be determined. The results are expressed in terms of the ground motion at which there is a High Confidence of a Low Probability of Failure (HCLPF). The HCLPF represents the ground acceleration at which there is 95% confidence of less than a 5% likelihood of failure. The HCLPF value can be mapped to the performance goals shown in Table. The HCLPF values corresponding to SDC-1 and SDC-2 performance goals are as follows.

- HCLPF values greater than 0.22g correspond to achieving performance goals for SDC-1
- HCLPF values greater than 0.40g correspond to achieving performance goals for SDC-2

Results

HCLPF values were computed for elements in the lateral force resisting system for two different limit states defined in ASCE 43-05. Limit State A represents large permanent distortion, but precludes collapse and instability. Limit State B represents moderate distortion and generally repairable damage. The limit states represent the limiting acceptable deformation, displacement, or stress that the structure may experience during or following an earthquake and still perform its safety function. The safety analysis performed in support of this study indicated that the offsite consequences of a seismic initiated accident sequence were low enough to support an SDC1 classification. Since the building was not credited with mitigating a release Limit State A is acceptable.

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[5] Electric Power Research Institute, "A Methodology for Assessment of Nuclear Power Plant Seismic Margin," EPRI NP-6041-SL, August, 1991.

Table 2 presents the results. The results demonstrate that the HCLPF exceeds 0.25g for both limit states A and B for all components except the metal roof deck diaphragm attachment and in-plane shear capacity in Area C. The results also show that the majority of elements meet the performance requirements for SDC 2; the exceptions include two steel moment frame baseplates and the metal roof deck on level 4 for Limit State B.

A detailed discussion of the results according to the summaries shown in Table 2 follows.

- **Metal Roof Deck Diaphragm Attachment to Frames**

  The attachment of the metal roof deck to the moment frame beams was shown to be inadequate at three locations in Area C and at one location on level 4. Only the attachment to frame 17 failed to meet SDC1A performance goals. The other attachments noted met SDC1A criteria, but fell short of SDC2B criteria.

  A subsequent analysis was performed to investigate whether a viable alternate load path exists to transfer inertial forces from the Area C deck into lateral force resisting elements. It was found that the beams and connections in the saddleback area along grids 4, 5, and 6 are adequate to transfer the inertial deck loads into the columns on grid B which in turn transmit the loads into the main level 3 diaphragm. It was also verified that the columns on grid A can tolerate the displacements, and subsequent loading, associated with this alternate lateral load support configuration.

- **In-Plane Shear in Metal Roof Decks**

  The initial results showed several locations in the Area C and Level 4 roof decks that fell short of the SDC2B performance goal. Additional study showed that the Level 4 roof deck issues were not concerns for overall stability since the metal roof deck is adjacent to a large section of composite floor deck. This floor deck is more than adequate to transfer the diaphragm forces to the supporting frames. At the onset of yielding in the metal roof decks the loads will transfer to the composite floor deck and then into the frames. No modifications are required for this issue.

  The alternate load path evaluation discussed for Area C applies to this condition.

- **Moment Frame Beam Bracing**

  Several frame beam-bracing details were suspected to be inadequate to allow the beams to develop their full plastic hinge capacity. After evaluating multiple bracing configurations, it was determined that several braces could not develop the loads stipulated by AISC 341-05.

  To address this issue it was decided to determine HCLPF values for the beams assuming that deficient braces were not present and to reduce the inelastic energy dissipation factor, F_μ, to those corresponding to an intermediate moment frame. Unbraced beam lengths are directly related to the beam capacity; therefore, impact the HCLPF calculation. The HCLPF values computed with these changes show the frame members meet both SDC1A and SDC2B criteria without any need for modifications.

- **Concrete Pilaster at Grid F/10**

  The preliminary results showed one concrete pilaster failed SDC2B performance goals. The west wall contains the pilaster in a location where the wall retains two stories of earth. The preliminary model considered different reduction factors on wall and pilaster elements which created an artificial stiffness disparity between the elements which act as one. To correct this inconsistency, the stiffness reduction factors used to simulate cracked concrete were adjusted so that the pilasters and surrounding wall elements used the same values. Following this change, the loads were more evenly distributed between the walls and the pilasters and all the HCLPF values met SDC2B performance.

- **Uplift Potential in Footings**

  Preliminary results showed uplift at some of the exterior footings. This was largely due to 1) not including soil load to resist uplift, and 2) using a response spectra analysis technique which is overly conservative in some cases.

  The modal results for the column bases were evaluated and it was found that only a few modes at a few columns produce uplift. This was consistent with the overturning analysis. For SDC1/2 demands, uplift is not a concern.

Conclusions and Recommendation

The results demonstrate that the structure will meet the seismic performance goals in DOE-STD-1020-2012 for SDC1 Limit State A without any modification to the structure. The results also show that a majority of elements meet the performance requirements for SDC 2 for limit state B.

Table 1 – Target Performance Goals for Seismic Design Categories (Ref. ASCE 43-05)

Seismic Design Category (SDC)	Target Performance Goal (P_F)
1	($<1 \times 10^{-3}$)*
2	($<4 \times 10^{-4}$)*
3	$\sim 1 \times 10^{-4}$
4	$\sim 4 \times 10^{-5}$
5	$\sim 1 \times 10^{-5}$

Notes:

* Working Group's assessment of performance goals approximately achieved by building codes.

Table 2: RLUOB Seismic Margin Study HCLPF Values

Component	Computed HCLPF Values		Comment
	Limit State A	Limit State B	
Spread Footings	0.4g	0.4g	
Concrete Columns	PMM: 0.80g Shear: 0.43g	PMM: 0.70g Shear: 0.43g	
Shear Walls	In-Plane: > 1g Out-of-plane:> 1g	In-Plane: >1g Out-of-plane: >1g	
Level 1 & 2 Concrete Diaphragms	1.43g	1.25g	
Steel Moment Frame Members	0.52g	0.41g	
Steel Moment FrameBeam to Column Connections	3.66g	2.44g	
Steel Moment Frame Base Plates	0.45g	0.30g	There are two baseplate locations with HCLPF values less than 0.40g for LS B
Non-Moment Frame Steel Members	0.76g	0.60g	
Steel Collector Connections	0.65g	0.51g	
Level 3 and 4 Composite Floor Diaphragms	Frame Attachment: 0.84g In-Plane Shear: 1.02g	Frame Attachment: 0.74g In-Plane Shear: 0.89g	
Metal Roof Deck Diaphragms	Frame Attachment: 0.17g In-Plane Shear: 0.22g	Frame Attachment: 0.15g In-Plane Shear: 0.19g	Load Path concerns near Frame 17. Original letter showed values based on recommended modifications. The current values assume no modifications.

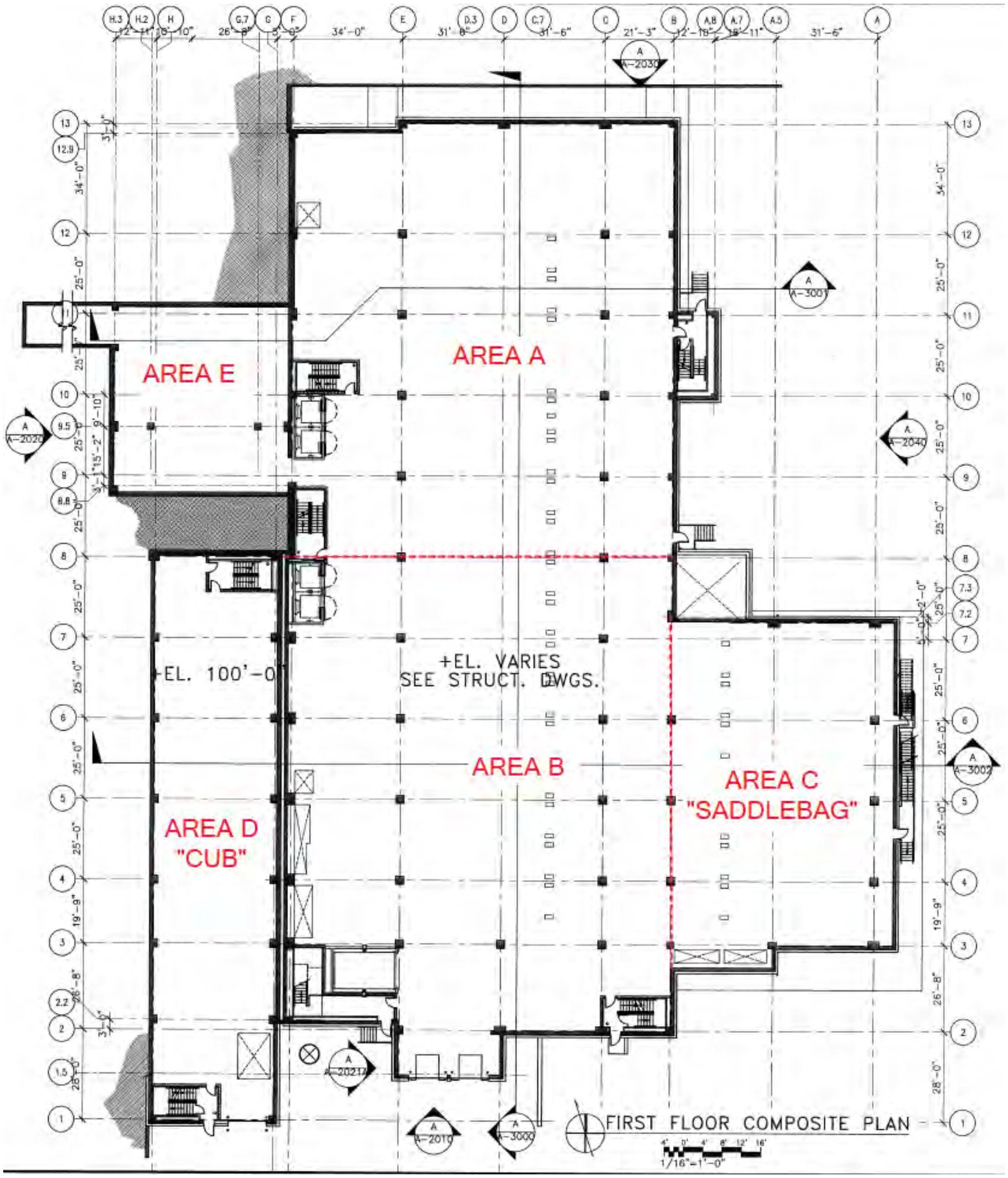

Figure 1 – First Floor Plan View from Sheet A-1150