Jessie H. Roberson, Vice Chairman
Sean Sullivan
Daniel J. Santos

DEFENSE NUCLEAR FACILITIES SAFETY BOARD

Washington, DC 20004-2901

February 4, 2015

Mr. Steven C. Erhart
Manager, NNSA Production Office
U.S. Department of Energy
P.O. Box 2050, Mail Stop 8009
Oak Ridge, Tennessee 37831

Dear Mr. Erhart:

The enclosed report is provided for your information and use as the National Nuclear Security Administration (NNSA) decides on a path forward for long-term mission work at the Y-12 National Security Complex. Building 9204-2E and the 9215 Complex have known structural performance deficiencies and do not meet modern structural design requirements. These deficiencies result in an increased potential for structural collapse and release of radiological material following certain seismic events. NNSA accepted this risk for near-term mission work with the intent of replacing the capabilities in these facilities with the planned Uranium Processing Facility (UPF). However, following an evaluation of alternative approaches for the UPF project in early 2014, NNSA removed the capabilities of Building 9204-2E and the 9215 Complex from the UPF project scope.

The Defense Nuclear Facilities Safety Board's staff has reviewed the structural performance of Building 9204-2E and the 9215 Complex to refine our understanding of the facilities' deficiencies. The report of this review is enclosed for your information and use as you and your staff re-evaluate these facilities for possible lifetime extension and mission capability additions.

Sincerely,

Jessie H. Roberson
Vice Chairman

Enclosure

c: Ms. Madelyn Creedon
Mr. Joe Olencz

DEFENSE NUCLEAR FACILITIES SAFETY BOARD

Staff Issue Report

November 14, 2014

MEMORANDUM FOR: S. A. Stokes, Technical Director

COPIES: Board Members

FROM: D. Andersen and B. Caleca

SUBJECT: Structural Evaluations of the 9215 Complex and Building 9204-2E at the Y-12 National Security Complex

This report documents a review by members of the Defense Nuclear Facilities Safety Board's (Board) staff of structural calculations for natural phenomena hazards (NPH) at the 9215 Complex and Building 9204-2E at the Y-12 National Security Complex (Y-12). The review focused on the structural calculations and drawings to identify gaps between these facilities' designs and modern seismic design practices.

Background. The National Nuclear Security Administration (NNSA) and Babcock & Wilcox Technical Services Y-12, LLC (B&W)[1], as part of an overall aging management strategy, conducted Facility Risk Reviews (FRRs) in 2007 and 2012 to assess the overall condition of Building 9204-2E and the 9215 Complex. The primary objective of the FRRs was to identify the highest priority repairs and facility modifications that would be necessary to ensure continued safe operations in these facilities until their capabilities could be redeployed in the Uranium Processing Facility (UPF). The 2012 FRRs assumed operations would continue in both facilities until at least 2030. Recent issues with the UPF project's cost and schedule caused NNSA to evaluate alternatives to its overall enriched uranium infrastructure replacement strategy. As a result, NNSA is changing its approach to rely on the deployment of new capabilities in Building 9204-2E and the 9215 Complex to support transition out of Building 9212, which is the highest hazard nuclear facility at Y-12 and is in poorer condition than Building 9204-2E or the 9215 Complex. The newly selected approach also caused the timetable for transitioning operations out of the 9215 Complex and Building 9204-2E to slip to an undetermined date. As such, site personnel have stated their intent to re-evaluate the FRR conclusions in the context of extended lifetime and additional mission capabilities for these facilities. This effort is scheduled for completion in the middle of calendar year 2015.

Deficiencies in Structural Ductility for Building 9204-2E and the 9215 Complex. When structural components deform in a ductile manner, they typically reduce the overall demands on the structure and, if properly designed, concentrate structural damage in preferred ways and enhance structural reliability. Modern codes and standards for building design account

[1] Consolidated Nuclear Security, LLC (CNS) took over as the Y-12 contractor on July 1, 2014.

for ductility and allow for reductions in elastic demands by accounting for the inelastic energy absorption capability of a structural system. Department of Energy (DOE) Standard 1020-2002, *Natural Phenomena Hazards Design and Evaluation Criteria for Department of Energy Facilities*, discusses the importance of ductile design to meet expected performance goals.

The designs of the 9215 Complex and Building 9204-2E do not include the ductile design concepts that are used in modern structural design, and thus lack seismic margin to collapse compared to a contemporary structure designed to the same demands. This weakness was acknowledged by B&W engineers, who used full elastic demands to qualify non-ductile structural components. Should seismic demands exceed the elastic capacities of certain structural members, undesirable failure modes may be triggered such as column or joint failures that can rapidly lead to progressive collapse.

9215 Complex Structures. The 9215 Complex consists of Buildings 9215, 9998, 9215A, 9811-2, 9996, and A-2 Wing of Building 9212. Because A-2 Wing is part of Building 9212, and Building 9996 neighbors Building 9212, they are not part of the scope of this review. In addition, because Building 9215A[2] and Building 9811-2[3] are minor structures, they were also excluded from this review. In its review of the 9215 Complex, the review team focused on the three main structures: Building 9215 M Wing, Building 9215 O and P wings, and Building 9998. These buildings were constructed in the 1950s.

Lateral stiffness for Buildings 9215 and 9998 is provided by steel bracing members, mostly made of single- and double-angle steel braces, plus architectural masonry hollow clay tile (HCT) infilled walls. The masonry walls are not credited for structural strength, but their stiffness is considered in the seismic analysis and changes the structural response of the building. During a design basis earthquake (DBE), many of these infill walls would be severely damaged; however, some would survive and continue to contribute to the lateral stiffness of the building. B&W performed extensive testing and analysis to demonstrate the survivability of these walls in response to seismic events for different wall configurations, penetrations, and offsets from the framing systems [1].

Structural Performance of the 9215 Complex. In the latest structural evaluation for the 9215 Complex, performed in 2005 [2], B&W analysts used a site-specific seismic spectrum along with a 50% reduction in the return period of the seismic hazard to calculate seismic demands. Both the site-specific analysis and return period reduction are allowed in DOE Standard 1020-2002 for existing Performance Category (PC)-2 structures. The spectrum was then additionally reduced by an R factor to account for inelastic behavior in the structure consistent with DOE standards for PC-2 facilities. The motivation for re-performing the structural analysis in 2005 was that the site-specific hazard response spectrum was lower than the code-based hazard spectrum used in the 2002 analysis of the 9215 Complex. The new spectrum, developed from the 2003 Probabilistic Seismic Hazard Analysis, resulted in reduced demands for the frequencies of interest for this structure (below 9 Hz).

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[2] Building 9215A is a maintenance annex constructed of unreinforced masonry on the east side of Building 9215.
[3] Building 9811-2 is a tanker truck bay covered by steel siding on the west side of Building 9215.

The analyses performed for the 9215 Complex were limited to linear elastic methods. To account for the behavior of masonry infill walls, 10% damping was selected based on research performed on infill-type structures, and an R factor of 3 was selected. Because R factors are based on 5% damped structural response, an adjusted R factor of 2.4 was used in conjunction with 10% damping for the response spectrum analysis of the structure; this approach is consistent with the intent of American Society of Civil Engineers (ASCE) code provisions for design. Based on B&W's analysis, the demands on the 9215 complex structural elements exceeded capacities in a number of locations, making the structure unable to meet PC-2 demands for an existing facility. According to the analysis, the extent of overstressed structural elements in the 9215 Complex includes, but is not limited to, the following:

- Several undersized braced frame connections.
- Several axially overstressed columns.
- Several axially overstressed vertical braced frame members.
- Loss of infilled HCT walls from in-plane and out-of-plane motions.
- A subgrade concrete wall that resists soil pressures.

The 9215 Complex structures are credited in the safety analysis as safety-significant. The 2005 analysis estimates that under a site-specific earthquake of approximately 0.12g peak ground acceleration, significantly below the PC-2 existing facility seismic demand, the 9215 Complex structures will have reached a damage state where progressive collapse of the structure is likely, damaging or destroying many if not all areas of the structure as a result. The safety analysis accepts this performance in the short term by formally crediting the structure to survive a 100-year return period earthquake (with an estimated site-specific demand of 0.05g), making structural collapse of the facility an unlikely, rather than anticipated hazard.

Review Team Concerns with the Structural Performance of the 9215 Complex. In addition to the aforementioned deficiencies where elements are overstressed by the PC-2 existing facility seismic demand, there are a number of non-ductile features within the structure that limit the ability of the 9215 Complex to withstand large inelastic demands for seismic hazards beyond the PC-2 existing facility seismic demand. Additionally, because the 9215 Complex is made up of several different structures with dissimilar layouts, there is the possibility for undesirable torsional motions due to this irregularity, which can result in unevenly distributed forces during a seismic event.

Bracing Connections—Modern design practices preferentially concentrate nonlinear deformations in braces before the remainder of the structure undergoes nonlinear behavior, i.e., it is desirable that the ultimate stress in tension of a brace is well beyond the yield strength of the member but less than the yield strength of the connection. Gusset plate connections are now designed such that the ultimate axial load of a brace can be transferred through the connection to prevent brittle failure modes such as block shear and bolt shear. Special Concentric Braced Frames (SCBF), which are the only braced frame system permitted under the International Building Code in high-seismic regions in the United States for structures that are high-hazard or high-occupancy, require this type of design approach. If a structure is permitted to be designed as an Ordinary Concentric Braced Frame (OCBF), and the connections cannot be made to develop the full tension force of a yielding brace, elastic load capacities would be permitted. The lack of lateral system ductility in such a structure would not be considered desirable when compared to best practices such as those outlined by the National Earthquake Hazards Reduction Program.

The current bracing connections of the 9215 Complex largely control the braced frame capacities, with failures in bolt or rivet shear predicted prior to yielding of the brace members in tension. Block shear typically does not control over bolt shear in these connections, but there are a number of cases where block shear capacities are also well below the brace yield capacity. B&W staff identified these features in structural analyses of the 9215 Complex; however, they only identified connections that fail under a PC-2 existing facility seismic load as candidates for retrofitting in the current analysis of record. The site has not performed retrofits because its priorities for facility improvements have been made based on the assumption that the 9215 Complex would be replaced by UPF in the near future. If the site's priorities were to change and NNSA elected to meet the detailing criteria for modern SCBF systems as a risk-reduction initiative, it is likely that every gusset plate of the lateral bracing system would need to be replaced with plates conforming to current code requirements.

Slenderness of Bracing—Slenderness limits on OCBF and SCBF brace members currently in structural steel codes did not exist when the 9215 Complex was constructed. A significant number of braces have slenderness ratios that exceed current code minimums for OCBF and SCBF systems. This could result in brace members that buckle in compression during NPH loadings. It is typical for OCBF systems to have braces with a higher buckling capacity. The slenderness of 9215 Complex braces is thus another design feature that limits its inelastic capacity significantly compared to modern structures of similar elastic strength.

B&W engineers acknowledged this noncompliance with modern codes and standards, and justified not replacing bracing members by considering the braces as tension-only structural components, which would not be credited to resist any compressive loads. The review team believes this is a reasonable approximation of existing brace behavior. Currently, ASCE codes do not allow the primary lateral system of a new structure to be designed using tension-only bracing due to their limited structural ductility. Meeting modern code requirements would require replacing a large number of brace members in the 9215 Complex. In turn, associated brace connections would need to be enlarged to accommodate the larger braces.

Linear Elastic Modeling—To evaluate the structure's component response more accurately than the results from linear elastic models, B&W engineers used a number of intermediate steps to account for incremental failures or nonlinear behavior outside of the linear elastic dynamic analyses. For brace connections and vertical load resisting elements (typically columns), an overstrength factor was used in conjunction with an elastic analysis. Evaluations of the infilled walls required an iterative process to determine the expected building response consistent with the expected number of walls lost.

One of the limitations of the above modeling approach is that it does not capture the most accurate set of structural deficiencies for the 9215 Complex. If NNSA intends to consider retrofits to the structure or re-evaluate it to better quantify and understand the risk of collapse, more advanced analysis techniques should be pursued to determine building performance and the complete set of areas requiring retrofit. A nonlinear analysis could: capture the effects of connection response, brace overstrength, redistribution of loads after damage, degradation of structural elements with cyclical loads, and the variations of damping in the system; determine which braces might exhibit nonlinear behavior prior to failure; account for asymmetric stiffness and capacity of slender braces; and account for the complex behavior of masonry infilled walls. Additionally, the current In-Structure Response Spectra (ISRS) used to qualify equipment in the 9215 Complex are most likely inappropriate considering the nonlinear behavior of the structure. The ISRS could also be revisited in a new analysis.

Building 9204-2E Structure. Constructed in the late 1960s, Building 9204-2E is a three-story, reinforced concrete moment frame structure. The 1960s-era design for the concrete structure predates modern seismic detailing codes for reinforced concrete moment frames. The building is supported on a basemat foundation, and its first floor is partially submerged below grade. The exterior walls of the structure are predominantly non-structural unreinforced masonry, with the exception of the first floor walls below grade, which are reinforced concrete. The building has several features that are advantageous for seismic performance: the structure has a regular and symmetric layout, resulting in minimal torsion of the building during dynamic loading, and the building column capitals have drop panels that provide additional punching shear strength (i.e., out-of-plane slab strength near the columns). Similar to the 9215 Complex, the structure's frame is infilled by masonry walls, most of which are HCT, but with a few areas where concrete block was used. At one location, siding replaced the masonry infill as a repair to address issues with wall movement due to exposure to the sun. The Building 9204-2E structure is credited in the safety analysis as safety-significant and has been qualified for the PC-2 existing structures seismic demand.

Structural Performance of Building 9204-2E. Although the Building 9204-2E safety basis only requires the facility to be qualified for PC-2 seismic requirements, the latest NPH evaluation performed in 2004 qualified the building to PC-3 existing structure seismic requirements for collapse prevention [3]. Both the PC-2 and PC-3 design criteria are based on the same return period earthquake, though PC-2 designs are allowed to take inelastic reduction factors. For this facility, the use of full elastic demands on critical or non-ductile components in the building structure make PC-2 and PC-3 evaluations nearly identical. The expected damage state of the facility for the existing facility seismic hazard will include damage or loss of some infilled walls and a loss of active or passive confinement as a result of a PC-3 or PC-2 seismic event, but the primary frame structure would be largely intact and undamaged.

The latest B&W seismic analysis utilized a 10% damped spectra, appropriate for a building with masonry infill walls, and a ductility factor of unity, as required for PC-3 analysis. The analysis identified several columns near the x-ray vault of the building that could become overstressed; however, the analysis attributes this prediction to issues arising from modeling thick-walled structural elements. In addition, damage to the connector structure between Buildings 9204-2E and 9204-2 is expected during a DBE, and a number of interior partition walls constructed of unreinforced masonry are at risk of collapse throughout Building 9204-2E. The current analysis for interior partition walls shows many with demand-to-capacity ratios very close to unity, and any uncertainties in the ISRS generated for Building 9204-2E could challenge these nonstructural features. For interior partition walls with inadequate capacity, updated analyses were performed using ISRS results from building analyses with the seismic demands from the latest Probabilistic Seismic Hazard Assessment as input. Where systems and components do not meet PC-2 criteria, efforts were made to remove or isolate material-at-risk to other areas of the facility [4].

As mentioned above, the area where most column damage is predicted after a seismic event is near the x-ray vault, a monolithic concrete structure with relatively high structural stiffness compared to the rest of the Building 9204-2E structure. Relative motion of the building and the vault may lead to overstressing the short concrete columns in shear in this area. B&W justified the adequacy of these short columns using a shear-friction-based shear capacity with equation 11-25 of American Concrete Institute (ACI) 349-01, *Code Requirements for Nuclear Safety-Related Concrete Structures* [5]. While the model uses an approximate height of four feet for these columns, most of that space is in fact taken up by the concrete floor slab, drop panel, and column capital. Only about six inches of clear height is solely the concrete column. The use of a shear-friction capacity may therefore be defensible for this configuration; however, that is not entirely clear or technically justified in the calculation. B&W performed two separate analyses with and without rotational stiffness for these column elements, and the results greatly affected shear values in columns located in the floor above. Any re-examination of the modeling assumptions used for Building 9204-2E should consider a more refined modeling of this region to better predict shear and moment demands.

Review Team Concerns with the Structural Performance of Building 9204-2E. In addition to the potential for structural damage to some concrete columns near the x-ray vault, the review team found that ductile detailing of the 9204-2E facility is lacking, which prevents the structure from adequately performing during large earthquakes beyond the DBE. For a number of areas, especially on upper floors, columns are not stronger than the ultimate moment strength of the adjacent beams. "Strong column, weak beam" behavior is required in modern concrete moment frame structural systems by ACI codes to prevent plastic hinging in concrete columns prior to beam hinging and to enhance structural ductility of the building system.

Furthermore, beam-column joints in modern buildings have very strict requirements for interior shear reinforcement and prohibit splicing of continuous steel reinforcement in the regions closest to the joint. Building 9204-2E's beam-column joints have marginal shear reinforcement and in many cases beam longitudinal reinforcement is spliced directly inside the beam-column joints, which would significantly limit the ductility of the connection. In general, Building 9204-2E does not have shear reinforcement in beams and columns as required by ACI codes to develop the full plastic moment strength of these elements or to confine concrete adequately in seismically-detailed moment frames.

Evaluation of Existing Facilities in DOE Standard 1020. Both the 2002 and 2012 revisions of DOE Standard 1020 contain provisions for the evaluation of existing facilities. Over the life of a facility, seismic hazards are updated and may increase over time, or the facility may undergo degradation or have weaknesses identified. The guidance in DOE Standard 1020 provides criteria expected of existing facilities where conformity to modern codes and standards may not always be possible, and can inform NNSA of the risks for continued operation of existing facilities. DOE Standard 1020-2002 allows a reduction of 50% of the return period for seismic hazards when evaluating existing facilities or systems that are "close to meeting criteria."

In practice, this allowance led to most existing facilities using the 50% reduction in return period as a starting point of analysis before evaluating the gap between new design performance criteria and existing capacities. Chapter 9 of the latest version of the standard, DOE Standard 1020-2012, clarifies that "close to meeting criteria" implies being within 10% of the needed capacity for a design basis NPH. This revision further restricts reductions in seismic hazard response spectra to no more than 20%. However, the reduction of the return period from 2500 years to 1250 years for the Y-12 site reduces the seismic hazard by significantly more than 20% for most frequencies of interest for these facilities (over 40% in some cases). Many structural members of these facilities that were deemed adequate with a reduced seismic hazard still had demand-to-capacity ratios greater than 0.8, including for brittle failure modes such as punching shear.

It appears that the evaluations for the 9215 Complex and Building 9204-2E are both using what the review team believes is an inappropriate level of hazard reduction, especially in light of the clarifying language of DOE Standard 1020-2012. NNSA accepted[4] these conditions based on the then-expected limited remaining operational life of these facilities and mission needs. However, recent changes to the scope of the UPF project change this outlook. New concepts are being considered to extend the use of the 9215 Complex and Building 9204-2E by at least twenty years.

Differences in Evaluation Techniques for DOE Standard 1020-2012—The addition of new processing capabilities to Building 9204-2E or the 9215 Complex (e.g., a major modification associated with installation of uranium electrorefining (ER) or direct electrolytic reduction processes) could drive new structural evaluations of these facilities. DOE Standard 1020-2012 states the following regarding major modifications to existing facilities:

> For major modifications of existing facilities, the design of SSCs [structures, systems, and components] shall be based on the methods and criteria given in this standard for new facilities with the following caveat. On a case-by-case basis, analyses may be performed to evaluate the need to upgrade existing SSCs (including interfacing SSCs) in accordance with these criteria …. The analyses shall be submitted to the DOE for approval … with recommendations and justification for the recommendations.

In a 2013 letter to the NNSA Production Office, B&W indicated that its considers the installation of ER equipment into Building 9215 to be a major modification, but concluded that it would not be in the best interest of NNSA to require significant structural upgrades. B&W supported this conclusion using a risk benefit argument, which is allowed by DOE Standard 1020-2012. B&W's argument was based on the high cost of structural upgrades, the small risk

[4] The current revision of the 9215 Complex Safety Evaluation Report states that "the significant impact on production activities, the line item cost, and the limited useful life does not support the PC-2 seismic upgrades for these facilities" [6].

of a significant seismic event during the period of ER operations, and the significant risk reduction of moving this capability out of Building 9212.

While B&W's conclusion does not necessarily reflect the position of the incumbent contractor, the review team believes it is a strong indication of the stance that CNS will take as the ER deployment project progresses, making it unlikely that a major modification determination alone would drive the contractor to re-analyze the structures in question using the latest version of DOE Standard 1020-2012. However, NNSA's recent decision to reduce the scope of the UPF project has rendered invalid some of the current assumptions[5] associated with the risk of continued operation of these facilities, particularly their remaining operational lifetime. In response to this change, CNS and NNSA are in the process of defining the approach for re-evaluating the facility conditions and risks, and re-prioritizing the risk-mitigation projects needed to continue safe operations for the remaining operational lifetimes of these facilities. The staff review team believes that NNSA and CNS should consider, as part of this re-evaluation, applying the increased seismic loads required by the latest version of DOE Standard 1020 for existing facilities while utilizing more advanced nonlinear analysis techniques, in particular for the 9215 Complex. Such an approach would provide those responsible for re-prioritizing the risk-reduction projects for these facilities a better representation of the risk presented by the 9215 Complex's structural deficiencies.

Conclusion. NNSA presently plans to operate both the 9215 Complex and Building 9204-2E well beyond what was originally predicted during the early stages of the UPF project. In addition, new processing capabilities are being considered for deployment in these existing facilities. With the remaining operational life of these two buildings now approaching the life assumed for new designs, the review team believes that NNSA should consider performing an updated analysis using more accurate nonlinear modeling techniques while applying the requirements of DOE Standard 1020-2012. The current evaluations of the 9215 Complex and Building 9204-2E do not consider the large extension of their operational lifespans and fail to explicitly acknowledge the impact of the lack of structural ductility on each building's design margin, particularly for the 9215 Complex. Site personnel stated their intent to re-evaluate the facility risk reviews in the context of extended lifetime and additional mission capabilities for these facilities. However, the review team learned that this effort is not scheduled for completion until the middle of calendar year 2015 at the earliest.

[5] These assumptions are documented in the NNSA Production Office-approved Safety Evaluation Reports for the Safety Analysis Report for the 9215 Complex and the Safety Analysis Report for Building 9204-2E.

Cited References:

[1] Martin Marietta, *Hollow Clay Tile Wall Program Summary Report*, July 30, 1995.

[2] Babcock & Wilcox Technical Services Y-12, *Site Specific Seismic Evaluation of Buildings 9215 and 9998*, DAC-ST-9215-A025, Rev. 0, October 17, 2005.

[3] Babcock & Wilcox Technical Services Y-12, *Update of the Natural Phenomena Evaluation of Building 9204-2E in Support of the 10CFR830 Program*, DAC-ST-92042E-A061, Rev. 1, April 2, 2004.

[4] Babcock & Wilcox Technical Services Y-12, *Update of the NP Evaluation of Buildings 9204-2E and 9204 SSCs*, DAC-ST-92042E-A066, Rev. 4, August 1, 2013.

[5] ACI, *Code Requirements for Nuclear Safety-Related Structures*, 2001.

[6] NNSA, *Safety Evaluation Report*, *SER-9215-RS, Approving the Annual Update to the 9215 Safety Basis*, June 25, 2014

Other References:

ASCE, *Seismic Evaluation and Retrofit of Existing Buildings*, 2013.

Babcock & Wilcox Technical Services Y-12, *NPH Evaluation of Buildings 9998 and 9215*, DAC-NP-921500-A003, Rev. 0, August 5, 2002.

Babcock & Wilcox Technical Services Y-12, *Passive Design Features Subject to Degradation Evaluation for 9215* Complex, March 7, 2013.

Babcock & Wilcox Technical Services Y-12, *Passive Design Features Subject to Degradation Evaluation for Building 9204-2E*, August 29, 2013.

Babcock & Wilcox Technical Services Y-12, *Building 9215 Structural Condition Assessment*, RP-9215-F-0001-000-00, Rev. 0, June 8, 2010.

Babcock & Wilcox Technical Services Y-12, *Building 9998 Structural Condition Assessment*, RP-9998-F-0002-000-00, Rev. 0, July 15, 2010.

Babcock & Wilcox Technical Services Y-12, *Facility Risk Review: Follow-on Evaluation for Buildings 9215 and 9204-2E*, RP-9215-F-0013-000-00, May 2012.

EQE International, *Development of Seismic Evaluation Criteria for URM Infill Frames Including Independent Review of Oak Ridge HCT Test Program,* August 1998.

Erhart, S.C., NNSA Production Office Manager, Letter to D. Richardson, Acting President and General Manager, Y-12, March 21, 2014.

Lockheed Martin Energy Systems, Inc., *Structural Walkdown of 9215 and 9998*, DAC-NP-921500-A001, July 23, 2002.

NIST, *Standards of Seismic Safety for Existing Federally Owned and Leased Buildings*, ICSSC RP-8, 2011.