SHINE® Medical Technologies

July 27, 2021

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U.S. Nuclear Regulatory Commission
ATTN: Document Control Desk
Washington, DC 20555

References: (1) SHINE Medical Technologies, LLC letter to the NRC, "SHINE Medical Technologies, LLC Application for an Operating License," dated July 17, 2019 (ML19211C143)
(2) NRC letter to SHINE Medical Technologies, LLC, "SHINE Medical Technologies, LLC – Request for Additional Information Related to Structural Engineering Topics (EPID No. L-2019-NEW-0004)," dated June 28, 2021 (ML21173A012)

SHINE Medical Technologies, LLC Application for an Operating License
Response to Request for Additional Information

Pursuant to 10 CFR Part 50.30, SHINE Medical Technologies, LLC (SHINE) submitted an application for an operating license for a medical isotope production facility to be located in Janesville, WI (Reference 1). The NRC staff determined that additional information was required to enable the staff's continued review of the SHINE operating license application (Reference 2).

Enclosure 1 provides the SHINE response to the NRC staff's request for additional information.

If you have any questions, please contact Mr. Jeff Bartelme, Director of Licensing, at 608/210 1735.

I declare under the penalty of perjury that the foregoing is true and correct.
Executed on July 27, 2021.

Very truly yours,

DocuSigned by:
Jim Costedio
F52DB96989224FF...

James Costedio
Vice President of Regulatory Affairs and Quality
SHINE Medical Technologies, LLC
Docket No. 50-608

Enclosure

cc: Project Manager, USNRC
SHINE General Counsel
Supervisor, Radioactive Materials Program, Wisconsin Division of Public Health

## ENCLOSURE 1

## SHINE MEDICAL TECHNOLOGIES, LLC

## SHINE MEDICAL TECHNOLOGIES, LLC APPLICATION FOR AN OPERATING LICENSE RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION

The U.S. Nuclear Regulatory Commission (NRC) staff determined that additional information was required (Reference 1) to enable the continued review of the SHINE Medical Technologies, LLC (SHINE) operating license application (Reference 2). The following information is provided by SHINE in response to the NRC staff's request.

### Chapter 3 – Design of Structures, Systems, and Components

#### RAI 3.4-18

The SHINE OLA should include adequate information to describe the Facility Structure (FSTR), including its seismic isolation, if any, its safety-related SSCs, anchorage of its safety-related SSCs, or their ability to resist their own inertia to avert their collapse during design basis events. In its response to RAI 3.4-8 parts (2) and (3), SHINE did not provide sufficient design details regarding the construction of the FSTR as it relates to seismic damage necessary for the NRC staff to complete its evaluation. The overall description of the FSTR and its integration to safety and non-safety-related SSCs, the performance of which could affect the safety function of FSTR and related SSCs are essential for the NRC staff to reach a reasonable assurance safety determination.

The response to RAI 3.4-8 parts (2) and (3) did not summarize the FSTR construction materials used. It also did not include some aspects of its size, its height (minimum, maximum), and thickness of its outer shell (minimum, maximum). Additionally, the description and seismic qualification for some of the FSTR's SSCs, such as the integration of the RPF metal tanks to the FSTR, were not complete. Similarly, the response was not clear on the capacity of shield plugs to remain in place after a design basis event. The response also did not clarify whether an unsatisfactory performance of the non-safety related structures, such as the Exhaust Stack, FSTR partition walls, and the non-safety-related portion of the FSTR could affect the safety-related portion of FSTR and relevant SSCs during a design basis event.

This information is needed to confirm conservatism in the design of the FSTR and its SSCs to external hazards and to design basis events (e.g., seismic, aircraft impact, blast loads).

(a) Complete the description of the FSTR. State its height (minimum, maximum), the thickness (minimum, maximum) of its outer concrete shell (roof, walls, etc.). Identify the materials used for its construction and associated standards (e.g., Type II cement with concrete compressive strength (fc') of 4,000 pounds per square inch, rebar ASTM 706, structural steel ASTM A-36).

(b) For the non-safety-related exhaust stack, state its proximity to other safety-related structures (i.e., FSTR, nitrogen purge system (N2PS). Confirm that the failure of the exhaust stack would not affect the structural integrity of the aforementioned SSCs from external hazards and design basis events. If SHINE relies on the exhaust stack to remain in place or otherwise perform during a design basis event to prevent damage to safety-related SSCs, identify the industry standards to which it has been designed.

(c) For other non-safety-related structures or components of structures (including the non-safety-related portions of the FSTR and its internal partitions) not designed for seismic, aircraft impact, or blast loading effects, discuss how their potential failure/collapse would not impact safety-related SSCs and the defense-in-depth design characteristics of the FSTR.

(d) Describe the seismic isolation of the safety-related portion of the FSTR from its non-safety portion. If a gap was included, state its size, location, and its adequacy for seismic isolation of safety and non-safety-related portions of the FSTR.

(e) Discuss whether the supercell(s) are an integral part of the FSTR. If so, discuss its (their) anchorage. If not anchored, discuss its (their) integration to the FSTR and its safety-related SSCs.

(f) Clarify whether the prefabricated/precast concrete removable shield plugs (or cover plugs) are qualified to remain in place during a design basis event.

Following responses to the above requests (a) through (g), update the SHINE FSAR accordingly.

**SHINE Response**

(a) SHINE has revised Section 3.4 of the FSAR to provide a complete description of the main production facility structure (FSTR), including the height, thickness of the outer concrete shell, materials used for construction, and the associated standards. A mark-up of the FSAR incorporating these changes is provided as Attachment 1.

(b) The exhaust stack is nonsafety-related and measures 67 feet tall from the zero-floor elevation of the FSTR to its peak. The exhaust stack is approximately 40 feet from the safety-related portion of the FSTR and over 85 feet from the nitrogen purge system (N2PS) structure. Failure of the exhaust stack would not affect the structural integrity of the safety-related portion of the FSTR because the safety-related portion of the FSTR is designed to withstand aircraft impact loading which would bound any potential loading due to exhaust stack collapse onto the safety-related portion of the FSTR. Failure of the exhaust stack would not affect the structural integrity of the N2PS structure because the N2PS structure is located outside of the potential fall path of the exhaust stack. The exhaust stack is not relied upon to remain in place (or otherwise perform) during a design basis event to prevent damage to safety-related structures, systems, or components (SSCs).

SHINE has revised Subsection 9a2.1.1.1 of the FSAR to clarify that the exhaust stack is nonsafety-related and that potential failure or collapse of the exhaust stack does not result in an unfavorable interaction with safety-related SSCs. A mark-up of the FSAR incorporating these changes is provided as Attachment 1.

(c) Section 3.4 of the FSAR provides a description of the FSTR, including the height of the nonsafety-related portion of the FSTR. SHINE has revised Section 3.4 of the FSAR to include additional dimensional details of the safety-related portion of the FSTR, as described in the SHINE Response to RAI 3.4-18(a). Based on the height differential at the adjoining areas of the safety-related portion of the FSTR and the nonsafety-related portion of the FSTR, the nonsafety-related portion of the FSTR cannot collapse onto the safety-related portion of the FSTR. An FSTR defense-in-depth (DID) function related to chemical storage room ventilation may be impacted by a collapse of the nonsafety-related portion of the FSTR; however, failure of this DID function does not create a risk of exceeding the SHINE safety criteria described in Section 3.1 of the FSAR. No other FSTR DID or safety-related functions would be impacted by a collapse of the nonsafety-related portion of the FSTR.

SHINE has revised Section 3.4 of the FSAR to state that interior partition walls that are co-located with safety-related SSCs and must maintain structural integrity to prevent unacceptable interactions with safety-related SSCs are classified as Seismic Category II. Seismic Category II interior partition walls are designed to remain in place or not interfere with the safety function of the adjacent SSCs during and after a design basis earthquake (DBE). A mark-up of the FSAR incorporating these changes is provided as Attachment 1.

The potential failure/collapse of nonsafety-related SSCs, including the nonsafety-related portions of the FSTR and its internal partitions, would not impact the ability of safety-related SSCs to perform their safety functions.

(d) Seismic isolation of the safety-related portion of the FSTR from the nonsafety-related portion of the FSTR is provided via a seismic separation joint (seismic gap). A seismic gap of 1 inch (in.) is specified to avoid interaction between the safety-related portion of the FSTR and the nonsafety-related portion of the FSTR at the foundations below elevation (EL) 0 feet (ft.), the floor slab at EL 22 ft., and roof at EL 40 ft. The maximum lateral deflection of the nonsafety-related portion of the FSTR is less than 1/2 in., based on the earthquake and wind design requirements of the International Building Code (IBC) (Reference 3). The maximum lateral deflection at roof elevation of the FSTR is less than 3/8 in. Therefore, the sum of maximum lateral deflections between the safety-related portion of the FSTR and the nonsafety-related portion of the FSTR is less than the provided 1 in. seismic gap.

There is a small section of the nonsafety-related portion of the FSTR (i.e., an area approximately 62 ft. by 30 ft. that is directly west of the Analytical Lab and Quality Control [QC] Lab as shown on Figure 1.3-1 of the FSAR) which is located on top of the slab that is shared with the safety-related portion of the FSTR. This 62 ft. by 30 ft. portion of the nonsafety-related FSTR is seismically isolated from the rest of the nonsafety-related FSTR and from the adjacent walls of the safety-related FSTR with a 1 in. seismic gap . The mass and stiffness of this 62 ft. by 30 ft. portion of the nonsafety-related FSTR was accounted for in the design of the safety-related portion of the FSTR.

SHINE has revised Section 3.4 of the FSAR to clarify the description of the seismic gap between the safety-related and nonsafety-related portions of the FSTR. A mark-up of the FSAR incorporating these changes is provided as Attachment 1.

(e) The supercell is not an integral part of the FSTR. The supercell is a series of ten hot cells connected to the FSTR by embedded carbon steel plates attached to the reinforced concrete slab with post-installed concrete anchors. The FSTR response spectra at the supercell location in the radioisotope production facility (RPF) is used as a design input for a detailed mass and stiffness finite element model of the supercell carbon steel framing, stainless steel hot cell liners, and lead shielding that is seismically analyzed dynamically. Base reactions from the detailed supercell model are applied to finite element models of the embedded carbon steel plates and are used to size the embedded plates and anchors.

Accelerations and displacements within the supercell are extracted from the dynamic analysis as inputs for the evaluation of attached SSCs.

SHINE has revised Subsection 4b.2.2.2.1 of the FSAR to clarify that the supercell is anchored to the FSTR and that the supercell is seismically qualified to perform its confinement and shielding safety functions during and following a DBE. A mark-up of the FSAR incorporating these changes is provided as Attachment 1.

(f) The shield plugs are precast concrete removable members designed to remain in-place during a DBE. The shield plugs are thick reinforced concrete blocks, inset in vaults to prevent lateral movement during a DBE. Vertical seismic accelerations of the shield plugs due to a postulated DBE do not exceed 1g; therefore, vertical displacement of the plugs due to a DBE is not credible.

SHINE has revised Section 3.4 of the FSAR to clarify that the shield plugs are seismically qualified to remain in place during a DBE. A mark-up of the FSAR incorporating these changes is provided as Attachment 1.

**RAI 3.4-19**

Section 3.4.5.1, "Aircraft Impact Analysis," of the SHINE OLA states that the FSTR has been designed consistent with DOE-STD-3014-2006, Appendix F of American Concrete Institute (ACI) 349-13, and Chapter NB of ANSI/AISC N690-12 for permissible ductility limits. It also states, in part, that "[t]o evaluate the capability of the structure to withstand impact from an aircraft, each wall that is subject to potential impact from an aircraft missile is evaluated. Figure 3.4-7 shows the openings in the building which are evaluated as missile barriers." The SHINE OLA further states that each FSTR wall that protects the safety-related SSCs was evaluated for impacts at its center and at critical locations near the edge of the wall panel.

In its response to RAI 3.4-11 regarding aircraft impact locations, SHINE stated that "[t]he structural screening and evaluation of the FSTR demonstrates that the building components meet the structural screening and evaluation guidelines at all [aircraft] impact locations and there is no safety-related equipment supported from the building in the vicinity of the postulated impact; therefore, the risk is deemed small and the results are documented."

It is not clear by limiting the analysis to postulated impacts at wall centers and at critical locations near the edge of each wall panel, the evaluation was adequate to assess other wall locations where safety-related SSCs (e.g., cranes) may be attached and the risk to be identified as "small." It is also not clear whether the postulated impact locations considered the structural adequacy of structural connections. Further, it is not clear what method of analysis SHINE has used for the aforementioned impact evaluations to assess wall deformation, cracking (if any), and risk so that the FSTR continues to maintain its structural integrity. In addition, it is not clear whether punching shear was implicitly evaluated as part of the perforation process consistent with DOE-STD-3014-2006 so that the risk to structural damage that could reduce facility's defense-in-depth is deemed to be small as well, and thus help terminate the analysis without consideration of fuel fires that may demand implementation of Appendix N4 to ANSI/AISC N690-12 for conformance of structural design to fires.

(a) Discuss how the localized structural evaluation at postulated impact locations can adequately represent the behavior at other wall critical locations where safety-related SSCs are attached, for example where crane runway systems are attached.

(b) Clarify whether the postulated impact locations at edge of each wall panel considered effects on structural connections.

(c) Discuss the method of analysis used and its conservatism in the screening and evaluation of wall deformations due to impact (including those at missile barrier locations identified in Figure 3.4-7 of the SHINE OLA), leading to the conclusion that the FSTR continues to maintain its structural integrity and its defense-in-depth with a risk level of concern deemed as "small." Include the basis for eliminating punching shear of the FSTR exterior walls.

Following responses to the above requests (a) through (c), update the SHINE FSAR accordingly.

**SHINE Response**

a) There are no SSCs attached to walls or roof slabs that may be subject to aircraft impact. The two cranes within the facility, which have runways that are mounted to the exterior walls, are classified as nonsafety-related SSCs. All exterior walls, interior labyrinth (missile barrier) walls at access points, and roof slabs bounding the safety-related areas of the FSTR are evaluated for the effects of a postulated aircraft impact.

b) The effects of postulated impacts on structural connections have been considered. All exterior walls, interior labyrinth walls, and roof slabs are reinforced concrete and are interconnected with fully reinforced, robust moment connections. Forces from edge and corner impact evaluations are used to check the edge of each wall for one-way and two-way shear to ensure the integrity of connections.

c) The aircraft impact analysis methodology is described in Subsection 3.4.5.1 of the FSAR. All exterior walls, interior labyrinth (missile barrier) walls at access points, and roof slabs bounding the safety-related areas of the FSTR are evaluated for the effects of a postulated aircraft impact in accordance with the U.S. Department of Energy (DOE) Standard DOE-STD-3014-2006, Accident Analysis for Aircraft Crash into Hazardous Facilities (Reference 4). Small openings (smaller than the diameter of the aircraft engines) were screened out of the aircraft impact evaluation. Each of the structural elements is first evaluated for local damage using empirical equations and shown to be thick enough to prevent scabbing. A global evaluation is then performed by considering impacts at the center, edge, and corner. This evaluation verifies that the energy imparted into the structure will not cause excessive deformations or large-scale failure. The basic moment and shear capacities of the concrete sections are determined using American Concrete Institute (ACI) 349-13, Code Requirements for Nuclear Safety-Related Concrete Structures & Commentary (Reference 5). Allowable limits for available energy dissipation are determined according to Appendix F of ACI 349. Both flexural ductility past yield and rotational constraints are evaluated to determine the maximum permissible deflection. One-way shear is evaluated at the edges of walls and slabs, based on the reactions developed from the impact energy. Punching shear is addressed by increasing the thickness of those elements 20 percent greater than the thickness required to prevent perforation in accordance with Subsection F.7.2.3 of ACI 349 and Table I of DOE-STD-3014-2006. This ensures that punching shear of the bounding structural elements is precluded. SHINE has revised Subsection 3.4.5.1 of the FSAR to clarify the basis for the statement that punching shear was not postulated. A mark-up of the FSAR incorporating these changes is provided as Attachment 1.

> The structural screening and evaluation aligns with the guidelines of Section 4.3 of DOE-STD-3014-2006 and demonstrates that the building will not experience excessive damage or deformation. As stated in the SHINE Response to RAI 3.4-19(a), there are no safety-related SSCs attached to walls that may be subject to aircraft impact. Therefore, according to Subsections 3.3.1 and 3.3.2 of DOE-STD-3014-2006, the risk is deemed small.

**<u>RAI 3.4-20</u>**

Chapter 2 of SHINE OLA states, in part, that "[t]he ASCE standards provide minimum load requirements for the design of buildings and other structures that are subject to building code requirements.…" Consistent with American Society of Civil Engineers (ASCE) 7-05, Section 2.5 and its commentary, the strength and stability of a structure shall be checked to ensure that it is capable of withstanding the effects of extraordinary (i.e., low-probability) events, such as fires, explosions, and vehicular impacts. Section 3.4.5.1 of the SHINE OLA states that the FSTR has been designed consistent with DOE-STD-3014-2006. The DOE-STD-3014-2006 discusses modes of impact and states that an "aircraft can crash into the structure either by skidding or by flying directly into it." SHINE determined that the Challenger 605 or Hawker 400 aircraft as the design basis aircraft for analyzing the postulated aircraft impacts. According to Table 2.2-12 of SHINE OLA the Southern Wisconsin Regional Airport (SWRA) has multiple runways one of which (runway 22) is less than a half a mile from the projected center of the SHINE facility. Given the proximity of the facility to SWRA runways, it may be possible to have an accidental skidding of an aircraft at the FSTR rooftop.

In RAI 3.4-16, the NRC staff requested SHINE to state whether the effects of traction in the design of the concrete roof and supporting steel truss (including stability of the compression flanges of the truss) for a postulated global response mode of impact with a horizontal velocity component, were considered. In its response SHINE stated, in part, that:

> Oblique or skidding impacts of the aircraft were not explicitly evaluated because a direct impact was determined to be controlling. This is because non-direct impacts would impart a portion of their energy perpendicular to the plane of wall or roof element (similar but less than the energy due to direct impact) and a portion into the plane of the wall or roof diaphragm, which is designed to be robust enough to distribute in-plane forces to adjacent supporting structural members. Non-direct impacts acting nearly in-plane with the impacted structural elements would likely glance off thereby imparting less overall energy to the structure.

The RAI response also states that the roof diaphragm is 12 inches thick and the truss is not designed to be composite with the roof and "any horizontal component of an aircraft strike would be dissipated through the roof diaphragm directly which is much stiffer against in-plane (horizontal) loading than the truss elements." When discussing the non-safety-related portion of the FSTR, however, the SHINE OLA states, in part, that the "concrete on metal deck mezzanine slab and metal deck roof slab are diaphragms that transfer the lateral loads to a series of vertical brace systems.…"

It is not clear in the RAI response, how the soft fuselage could bounce off of a rigid surface unless the collision between the fuselage and the FSTR rooftop is perfectly elastic. In SHINE's described perfectly elastic collision scenario, it is reasonable to conclude that the generated horizontal forces would be considerably less than those associated with a skidding aircraft across the FSTR roof top. It is also not clear whether the FSTR roof system relies completely on the 12-inch rigid concrete diaphragm for distribution of lateral seismic forces to shear walls or it is assisted by the metal deck roof slab for the transfer of lateral loads as well. In addition, it is not clear whether the analysis for seismic qualification of the FSTR adequately bounds the analysis for a skidding aircraft across the roof of the FSTR.

(a) Describe the safety-related portion of the FSTR roof system. Discuss whether the steel roof decking participates in the distribution of lateral forces to vertical elements. If not, state the reason.

(b) Clarify whether the seismically generated lateral forces at rooftop bound those of a skidding aircraft lateral forces and hence the analysis for seismic qualification of the FSTR bounds the analysis of a skidding aircraft across the roof top of the FSTR.

Following responses to the above requests (a) and (b), update the SHINE FSAR accordingly.

**<u>SHINE Response</u>**

(a) There are two safety-related roof systems of the FSTR, one roof system is shared by the irradiation facility (IF) and RPF, and one roof system is for the non-radiologically controlled seismic area. There is a series of steel trusses that span in the east-west direction of the IF and RPF roof and are supported by the major shear walls of the FSTR. These trusses support a 5 inch (in.) thick leave-in-place reinforced concrete form slab on metal deck. This form slab provides temporary support for the main structural slab during the construction phase of the project and is attached to the steel trusses with headed shear studs. The form slab is vertically supported by structural steel shelf angles at the face of all shear walls and is not integrated into the walls. The main 12 in. thick reinforced concrete structural slab is placed on top of the 5 in. thick form slab. Because of the aspect ratio of the roof, the main structural slab primarily spans in the north-south direction between trusses. The main roof slab is connected to the main shear walls of the facility on all sides. Both layers of vertical reinforcement in all major shear walls are fully developed into the main roof slab. The main roof slab is not physically attached to the form slab or the roof trusses.

    As the main roof slab is not physically attached to the form slab or the roof trusses, it is designed to transfer the lateral loads on the roof (i.e., seismic, wind) directly to the shear walls through diaphragm action. The form slab and roof trusses are, therefore, only required to withstand lateral seismic loading due to their own self-weight and do not participate in the distribution of lateral forces to vertical elements.

    The roof system for the non-radiologically controlled seismic area is a 20 in. thick free-spanning cast-in-place reinforced concrete roof that transfers all of its vertical and horizontal loading to the shear walls that support it.

(b) As stated in the SHINE Response to RAI 3.4-20(a), the main structural roof slab is 12 in. thick. The roof slab is designed as a diaphragm using the maximum lateral roof accelerations, combined using the square root off the sum of the squares (SRSS) method, from the seismic analysis model. The resulting lateral design acceleration is 0.7g. The effective seismic mass of the roof slab is approximately 246 pounds per square foot (psf). The total roof area is calculated to be approximately 16,100 sq. ft. The resulting total seismic load from the roof slab is 2772 kilopounds force (kips) (0.7g * 246 psf * 16,100 sq. ft.). By comparison, the maximum out-of-plane shear load due to a perpendicular impact on the main roof slab is approximately 860 kips. A skidding aircraft impact will have vertical and horizontal force components, and the maximum horizontal force component is not larger than the maximum out-of-plane shear load due to perpendicular impact. Since the maximum out-of-plane shear load due to perpendicular impact is smaller than the total lateral seismic force from the roof, the seismic qualification of the roof slab bounds the skidding aircraft impact scenario.

SHINE has revised Subsection 3.4.5.1 of the FSAR to clarify that the FSTR seismic loading bounds any aircraft impact scenario that produces lateral forces. A mark-up of the FSAR incorporating these changes is provided as Attachment 1.

**RAI 3.4-21**

As noted in SHINE OLA and in SHINE's response to RAI 3.4-13, the assessment of FSTR crane loads and loading combinations is consistent with the American Society for Mechanical Engineers (ASME) NOG-1, "Rules for Construction of Overhead and Gantry Cranes" (ASME, 2004). For civil/structural designs, however, typically local building codes reference such industry standards as the International Building Code (IBC), ASCE 7, or others, as applicable. In that regard, in its response to RAI 3.4-13, SHINE describes the following:

- The RPF and irradiation facility (IF) cranes were evaluated using the deterministic approach of ASME NOG-1 with maximum potential crane load combinations considered, concluding that the design approach is conservative and "margin exists."

- ASME NOG-1 derived impactive (live) loads are less conservative than those provided in Section 4.9 of ASCE 7-05 and Section 1607 IBC for vertical and lateral loads. SHINE also stated that the impactive loads are approximately 10 percent of the seismic loads and stated, in part, that "[i]n light of the conservatism associated with the seismic loads and the magnitude of the difference between the impact and seismic loads, the slight difference in impact load factors is negligible."

In addition, SHINE stated that crane runway systems are not attached to exterior (FSTR) walls, and as such are sheltered from blast and aircraft impact loading.

As noted above, typical building codes for calculating crane loads follow industry codes and standards as those of ACI, ASCE, IBC, etc., as applicable. Accordingly, the following is not clear to the NRC staff:

- It is not clear what level of conservatism/margins there is in the SHINE structural facility design for crane loadings calculated based on ASME NOG-1 requirements versus those of the aforementioned building codes. It is also not clear whether the margins have been quantified for each (i.e., IF, RPF) of the crane runway systems and associated supports.

- It is not clear what building industry code or standard SHINE used to derive crane impactive loads. Typically, codes, such as ASCE or IBC, calculate the crane impactive loads based on percentage of a crane's lift capacity and its dead load. If SHINE based its crane impactive loads on seismic loads, it is not clear what was the basis for such determination. It is also not clear, whether the design methodology used resulted in a more conservative crane support system design when compared to that based on local and applicable civil/structural building code designs.

Further, it is not clear how the IF and RPF crane runway systems are supported in the FSTR. It is not clear whether they are attached to external walls and if so how. It is also not clear how the implemented crane runway system design isolates a crane from external forces/lateral loads (e.g., blast loads, aircraft impact), including base induced shear.

(a) Discuss whether the design margins have been quantified for each of the crane runway systems (i.e., IF, RPF) and if so, how conservative the design of the crane runway system (including its supports) is when calculating the necessary design crane loads based on ASME NOG-1.

(b) Provide the basis for using seismic loads as crane impactive loads. Clarify whether the loads evaluated based on building codes are less conservative than those derived from the discussed methodology.

(c) Describe the crane runway system including its supports for each of the FSTR cranes (i.e., IF, RPF) and how the design isolates the cranes from external forces including base induced shear.

Following responses to the above requests (a) through (c), update the SHINE FSAR accordingly.

**SHINE Response**

(a) The SHINE Response to RAI 3.4-13 (Reference 6) describes how the IF and RPF crane loads were derived for the analysis of the overall building. The IF and RPF crane loads used for the building analysis are based on the maximum potential lifted load and the peak seismic accelerations form the in-structure response spectra developed at the crane elevations and are larger than the actual crane reactions. All building sections, including the walls directly supporting the IF and RPF cranes, are designed with a minimum of 10 percent margin for all failure modes under all loading conditions. So, the IF and RPF crane loads that were used are conservative and the building is designed to have 10 percent margin even when considering the conservative crane loading.

    The seismic analysis of the IF and RPF cranes is performed in accordance with the requirements of American Society for Mechanical Engineers (ASME) NOG-1, “Rules for Construction of Overhead and Gantry Cranes” (Reference 7). The IF and RPF crane runway girders and associated support systems have been evaluated based on loads that are equal to or greater than the IF and RPF crane reactions determined from the seismic analysis of the cranes. A minimum 7 percent design margin exists for any component in the IF and RPF crane runway systems.

(b) The impactive loads used for the building and the IF and RPF crane runway system designs are based on a percentage of the crane lift capacity and its dead load in accordance with ASME NOG-1 (Reference 7). The percentage of the crane lift capacity and its dead load in ASME NOG-1 differs from the values in American Society of Civil Engineers (ASCE) 7-05, “Minimum Design Loads for Buildings and Other Structures” (Reference 8) and the IBC (Reference 3). The ASME NOG-1 impactive loads are consistent with other loads considered in the building design. The ASME NOG-1 values for lateral impact loads are on the order of 15 kips and the difference between ASCE 7-05 or IBC and ASME NOG-1 loads amount to a few kips, with the ASME NOG-1 values being less conservative. In contrast, the seismic lateral loads are on the order of 150 kips. The small (i.e., a few kips) difference in impactive loads from the use of ASME NOG-1 is insignificant in comparison with the magnitude of the horizontal loads considered due to a DBE. Because the DBE loads are conservatively calculated as described in the SHINE Response to RAI 3.4-21(a), and because the DBE loads have a much larger magnitude than the impact loads, the difference in impact loading (i.e., between ASME NOG-1 and ASCE 7-05 or IBC) has no effect on the building design. Whether applying ASME NOG-1, ASCE 7-05, or IBC the impactive loads used for the building and the IF and RPF crane runway system designs are bounded by the loads considered due to a DBE.

(c) The IF and RPF crane runway girders are built-up W-shapes with cap channels attached to its top and bottom flanges. The girders are bolted to the top surface of cantilevered hollow structural steel (HSS) corbels, which are welded to embedment plates cast into the walls. The top flange of the built-up runway girder is restrained at both ends and the middle of each simply-supported girder section.

The IF and RPF crane runway systems are designed to isolate the cranes from the exterior walls in a manner that prevents aircraft impact loading and base induced shear loading on the cranes. This is accomplished by utilizing a sacrificial component that fails upon being overloaded laterally whether by displacements imposed on the wall under aircraft impact loading or DBE. The IF and RPF crane trolley girders have bumpers designed to engage once the sacrificial component fails, thus changing the load path for large lateral load events and ensuring the crane remains in-place on the runway girder.

SHINE has revised Subsection 9b.7.2.1 of the FSAR to clarify that the IF and RPF cranes are designed to remain in-place on the runway girder, with or without a load, during and after a seismic event. A mark-up of the FSAR incorporating these changes is provided as Attachment 1.

**RAI 3.4-22**

In response to RAI 3.2-1 SHINE clarified that the N2PS structure performs, supports, and/or protects a safety function. SHINE also added Section 3.6, "Nitrogen Purge System Structure," to the FSAR to describe the N2PS structure. SHINE FSAR Section 9b.6.2, "Nitrogen Purge System," further describes the N2PS system.

SHINE FSAR Sections 3.6 and 9b.6.2 note that the N2PS piping and structure are seismically qualified; however, it is not clear that the supports are seismically qualified, specifically the nitrogen tube supports within the N2PS structure.

Clarify whether the N2PS supports are seismically qualified and identify what design code was used to design the supports. Update the appropriate FSAR section(s) to include this level of detail.

**SHINE Response**

N2PS pipe supports and high-pressure nitrogen tube supports are seismically qualified in accordance with ASCE 7-05 (Reference 8). SHINE has revised Subsection 9b.6.2.3 of the FSAR to clarify the seismic qualification of N2PS supports. A mark-up of the FSAR incorporating these changes is provided as Attachment 1.

**References**


1. NRC letter to SHINE Medical Technologies, LLC, "SHINE Medical Technologies, LLC – Request for Additional Information Related to Structural Engineering Topics (EPID No. L-2019-NEW-0004)," dated June 28, 2021 (ML21173A012)

2. SHINE Medical Technologies, LLC letter to the NRC, "SHINE Medical Technologies, LLC Application for an Operating License," dated July 17, 2019 (ML19211C143)

3. International Code Council, Inc., "International Building Code," IBC, 2015.

4. U.S. Department of Energy, "Accident Analysis for Aircraft Crash into Hazardous Facilities," DOE-STD-3014-2006, 2006

5. American Concrete Institute, "Code Requirements for Nuclear Safety-Related Concrete Structures & Commentary," ACI 349-13, 2014

6. SHINE Medical Technologies, LLC letter to the NRC, "SHINE Medical Technologies, LLC Operating License Application Response to Request for Additional Information," dated January 29, 2021 (ML21029A102)

7. American Society of Mechanical Engineers, "Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder)," NOG-1-2004, 2004

8. American Society of Civil Engineers, "Minimum Design Loads for Buildings and Other Structures," ASCE 7-05, 2006

**ENCLOSURE 1**
**ATTACHMENT 1**

**SHINE MEDICAL TECHNOLOGIES, LLC**

**SHINE MEDICAL TECHNOLOGIES, LLC APPLICATION FOR AN OPERATING LICENSE**
**RESPONSE TO REQUEST FOR ADDITIONAL INFORMATION**

**FINAL SAFETY ANALYSIS REPORT CHANGES**
**(MARK-UP)**

10 pages follow

## 3.4 SEISMIC DAMAGE

Seismic analysis criteria for the main production facility structure (FSTR) are supported by the detailed guidance provided by the referenced Regulatory Guides and sections of NUREG-0800, Standard Review Plan for the Review of Safety Analysis for Nuclear Power Plants (SRP).

The FSTR includes the irradiation facility (IF), the radioisotope production facility (RPF), the non-radiologically controlled seismic area, and a nonsafety-related area. The IF contains the irradiation units (IUs) and tritium purification system (TPS), and the RPF contains the supercell and below-grade tanks. The non-radiologically controlled seismic area contains the control room, battery rooms, uninterruptible electrical power supply rooms, and other miscellaneous support rooms. The RPF, IF, and non-radiologically controlled seismic area are within the seismic boundary and are classified as Seismic Category I. These areas contain the safety-related structures, systems, and components (SSCs). To the south of the seismic boundary are the shipping and receiving areas, as well as other areas that contain nonsafety-related support systems and equipment. This part of the structure is not Seismic Category I. The areas outside the seismic boundary do not contain safety-related SSCs.

The IF, RPF, and non-radiologically controlled seismic area comprise the safety-related portion of the FSTR. The dimensions of the safety-related portion of the FSTR at grade level are approximately 212 feet (ft.) (64.6 meters [m]) in the north-south direction and 158 ft. (48.2 m) in the east-west direction. Each of the three main areas of the safety-related portion of the FSTR is a parallel, single-story box-type structure designed with cast-in-place reinforced concrete shear walls. The major structural elements include the foundation mat, mezzanine floor, roof slab, and shear walls. ~~The roof slabs of the IF and RPF are supported by steel roof truss systems. The reinforced concrete mezzanine slab on metal deck is vertically supported by structural steel beams and columns, and laterally restrained by interior reinforced concrete walls. A large section of the basemat in the RPF is recessed below grade, where a series of below grade tanks, valve pits, and other mechanical systems are located. Each of these tanks is separated by cast-in-place reinforced concrete walls and is covered by a series of precast concrete shield plugs that create a removable slab at the same elevation as the rest of the basemat.~~ Depending on their function, interior walls are cast-in-place reinforced concrete, reinforced masonry, or gypsum mounted to metal studs.

The IF and RPF have a shared sloped main roof slab with a low point elevation of approximately 45 ft. (13.7 m) and a high point elevation of approximately 56 ft. (17 m). The IF and RPF roof dimensions are approximately 212 ft. (64 m) in the north-south direction and 126 ft. (38.4 m) in the east-west direction. The IF and RPF roof slab is 12 inches (in.) (0.3 m) thick and has a 5 in. (0.13 m) thick leave-in-place form slab on metal deck beneath it. The IF and RPF roof slab is supported by a series of roof trusses, which are made out structural steel shapes having a yield strength of 50 kilopounds per square inch (ksi) (6.89 MPa).

The non-radiologically controlled seismic area roof slab is 20 in. (0.51 m) thick and has a high point elevation of approximately 22 ft. (6.71 m). The non-radiologically controlled seismic area roof dimensions are approximately 148 ft. (45.1 m) in the north-south direction and 32 ft. (9.75 m) in the east-west direction.

Interior to the IF and RPF there is a mezzanine with 8 in. (0.2 m) thick reinforced concrete slab on metal deck, vertically supported by structural steel beams and columns (structural shapes with yield strength of 50 ksi [6.9 MPa]), and laterally restrained by interior reinforced concrete walls. A large section of the RPF mat slab is recessed 12 ft. (3.66 m) to 23 ft. (7 m) below the main mat slab, where a series of 1 ft. (0.3 m) thick (minimum) reinforced concrete walls divides the area. The exterior below grade walls in the recessed portion of the RPF range from a minimum of 2 ft. (0.61 m) thick to a maximum of 3.5 ft. (1.07 m) thick, and the basemat is 2.5 ft. (0.76 m) thick. The RPF below grade areas are covered by a series of precast concrete shield plugs. A section of the IF mat slab is recessed 16 ft. (4.9 m) below the main mat slab, where a series of 4.5 ft. (1.37 m) thick (minimum) reinforced concrete walls divides the area. The exterior below grade walls in the recessed portion of the IF range from a minimum of 4 ft. (1.2 m) thick to a maximum of 5.83 ft. (1.8 m) thick, and the basemat is 3 ft. (0.91 m) thick. Shield plugs cover the IU cells which are located above the recessed portions of the IF. Shield plugs in the IF and RPF are seismically qualified to remain in place during a design basis earthquake (DBE).

The reinforced concrete structural elements of the safety-related portion of the FSTR are constructed of Type II concrete with a design compressive strength at 28 days of 6000 pounds per square inch (psi) (41.37 MPa) and American Society for Testing and Materials (ASTM) A706/A706M-16, Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement (ASTM, 2016) steel reinforcement bars with a minimum design yield strength of 60 ksi (413.7 MPa). The exterior above grade walls and major shear walls range from a minimum of 2 ft. (0.61 m) thick to a maximum of 2.33 ft. (0.71 m) thick. The reinforced concrete structures are founded on a 3 ft. (0.91 m) thick mat slab that is thickened to 4.5 ft. (1.37 m) thick around the building perimeter.

The dimensions of the nonsafety-related portion of the FSTR at grade are approximately 77 ft. (23.5 m) in the north-south direction and 158 ft. (48.2 m) in the east-west direction. Additionally, the southwest corner of the safety-related basemat contains a part of the nonsafety-related portion of the FSTR. The dimensions of this nonsafety-related part are approximately 63 ft. (19.2 m) in the north-south direction and 32 ft. (9.8 m) in the east-west direction. The safety-related ~~basemat and structures on the safety-related basemat are~~portion of the FSTR is seismically isolated from the nonsafety-related portion of the FSTR via a seismic separation joint (i.e., seismic gap).

The nonsafety-related portion of the FSTR is a two-story steel framed structure with a roof height of approximately 40 ft. (12.2 m). The concrete on metal deck mezzanine slab and metal deck roof slab are diaphragms that transfer the lateral loads to a series of vertical brace systems. The FSTR also includes a nonsafety-related, isolated, self-supporting steel on reinforced-concrete foundation cantilevered exhaust stack with a height of approximately 67 ft. (20.4 m) located east of the nonsafety-related portion of the FSTR.

The FSTR is modeled to the analyses described in this chapter. The concrete walls, slabs, and basemat are modeled using thick shell elements. The steel structural members are modeled using three-dimensional beam elements. Interior partition walls made of concrete are modeled using thick shell elements. Interior partition walls made of masonry or gypsum are isolated from the lateral load resisting system of the building and are not explicitly modeled, but their mass is accounted for. Interior partition walls that are co-located with safety-related SSCs, and must maintain structural integrity to prevent unacceptable interactions with safety-related SSCs, are classified as Seismic Category II. The excavated soil volume of the soil-structure interaction (SSI) analysis is modeled using solid elements. Seismic mass is considered in the model in accordance with SRP Section 3.7.2 (USNRC, 2013a). Figure 3.4-1 and Figure 3.4-2 provide three-dimensional views of the structural model.

#### 3.4.3.2.4 Combined Methods of Qualification

Based on the available information, component complexity, and functional requirements, the above mentioned analytical and test methods may be combined in various sequence and content to achieve seismic qualification of the subject components.

### 3.4.4 SEISMIC INSTRUMENTATION

Seismic instrumentation is not required under Section IV(a)(4) of Appendix S to 10 CFR 50 or Section VI(a)(3) of Appendix A to 10 CFR 100 because the main production facility is not a nuclear power plant. However, the facility has nonsafety-related seismic instrumentation to record accelerations experienced at the site during a seismic event.

The seismic instrumentation establishes the acceptability of continued operation of the plant following a seismic event. This system provides acceleration time histories or response spectra experienced at the facility to assist in verifying that safety-related SSCs at the main production facility can continue to perform their safety functions.

Seismic monitoring is performed by the process integrated control system (PICS), which is described in Section 7.3. Indication of a seismic event results in an alarm in the facility control room.

### 3.4.5 SEISMIC ENVELOPE DESIGN FOR EXTERNAL HAZARDS

#### 3.4.5.1 AIRCRAFT IMPACT ANALYSIS

The safety-related structures at the SHINE facility are evaluated for aircraft impact loading resulting from small aircraft which frequent the Southern Wisconsin Regional Airport (SWRA). The analysis consists of a global impact response analysis and a local impact response analysis.

The global impact response analysis is performed using the energy balance method, consistent with U.S. Department of Energy (DOE) Standard DOE-STD-3014-2006 (DOE, 2006). The permissible ductility limit for reinforced concrete elements is in accordance with Appendix F of ACI 349-13 (ACI, 2014). The permissible ductility limit for truss members is determined from Chapter NB of ANSI/AISC N690-12 (ANSI/AISC, 2012). The calculated values are then used to create the appropriate elastic or elastic-plastic load deflection curves. From these curves, the available energy absorption capacity of the structure at the critical impact locations is determined. The Challenger 605 was selected as the critical aircraft for the global impact analysis based on a study of the airport operations data. The Challenger 605 is evaluated as a design basis aircraft impact. The probabilistic distributions of horizontal and vertical velocity of impact are determined from Attachment E of Lawrence Livermore National Laboratory UCRL-ID-123577 (UCRL, 1997) to correspond to 99.5 percent of impact velocity probability distribution.

Each wall that protects safety-related equipment was evaluated for perpendicular impacts at the center of the wall panel and at critical locations near the edge of the wall panel. Each roof that protects safety-related equipment was evaluated for perpendicular impacts near the end of the roof truss, at the center of the roof truss, at the center of the roof panel between trusses or walls. The evaluation of the roof slab for horizontal seismic loading bounds any aircraft impact scenario that produces lateral forces, so horizontal aircraft impact scenarios are not explicitly evaluated for the roof.

The local response evaluation was conducted using empirical equations in accordance with DOE-STD-3014-2006 (DOE, 2006). The structure was shown to resist scabbing and perforation. A punching shear failure was not postulated because all sections are shown to have a thickness 20 percent greater than the thickness required to prevent perforation, based on Appendix F of ACI 349-13 (ACI, 2014). Scabbing and perforation thickness requirement was calculated using DOE-STD-3014-2006 (DOE, 2006).

Because engine diameter and engine weight are both critical for the local evaluation, the local impact evaluation was performed for the Hawker 400 as well as the Challenger 605 aircraft. The Challenger 605 and Hawker 400 are evaluated as design basis aircraft impacts.

To evaluate the capability of the structure to withstand impact from an aircraft, each wall that is subject to potential impact from an aircraft missile is evaluated. Figure 3.4-7 shows the openings in the building which are evaluated as missile barriers.

The design basis aircraft impacts have been evaluated against the acceptance criteria of ACI 349-13 (ACI, 2014) for concrete and ANSI/AISC N690-12 (ANSI/AISC, 2012) for steel and it has been demonstrated that all components of the FSTR structure that are relied upon to provide impact protection have adequate energy absorption capacity to perform their design basis function.

#### 3.4.5.2 EXPLOSION HAZARDS

Because the SHINE facility is not licensed as an operating nuclear reactor, explosions postulated as a result of the design basis threat as defined in Regulatory Guide 5.69, Guidance for the Application of Radiological Sabotage Design-Basis Threat in the Design, Development and Implementation of a Physical Security Program that Meets 10 CFR 73.55 Requirements (USNRC, 2007e), are not considered. However, accidental explosions due to transportation or storage of hazardous materials outside the facility and accidental explosions due to chemical reactions inside the facility are assessed.

The maximum overpressure at any safety-related area of the facility from any credible external source is discussed in Subsection 2.2.3). The seismic area is protected by outer walls and roofs consisting of reinforced concrete robust enough to withstand credible external explosions as defined in Regulatory Guide 1.91, Revision 2, Evaluations of Explosions Postulated to Occur at Nearby Facilities and on Transportation Routes Near Nuclear Power Plants (USNRC, 2013c).

## 3.7 REFERENCES

**ACI, 2014.** Code Requirements for Nuclear Safety-Related Concrete Structures and Commentary, ACI 349-13, American Concrete Institute, 2014.

**AEC, 1963**. Nuclear Reactors and Earthquakes, TID-7024, U.S. Atomic Energy Commission, August 1963.

**ANSI/AISC, 2012.** Specification for Safety-Related Steel Structures for Nuclear Facilities, ANSI/AISC-N690, American National Standards Institute/American Institute of Steel Construction, 2012.

**ASCE, 2000.** Seismic Analysis of Safety-Related Nuclear Structures and Commentary, ASCE 4-98, American Society of Civil Engineers, 2000.

**ASCE, 2006.** Minimum Design Loads for Buildings and Other Structures, ASCE 7-05, American Society of Civil Engineers, 2006.

**ASCE/SEI, 2005.** Seismic Design Criteria for Structures, Systems, and Components in Nuclear Facilities, ASCE 43-05, American Society of Civil Engineers, 2005**.**

**ASME, 2004.** Rules for Construction of Overhead and Gantry Cranes, ASME NOG-1, American Society of Mechanical Engineers, 2004.

**ASTM, 2016.** Standard Specification for Deformed and Plain Low-Alloy Steel Bars for Concrete Reinforcement, ASTM A706/A706M-16, American Society for Testing and Materials, 2016.

**DOE, 2006.** Accident Analysis for Aircraft Crash into Hazardous Facilities, DOE-STD-3014-2006, U.S. Department of Energy, 2006.

**NFPA, 2008.** Standard for Fire Protection for Facilities Handling Radioactive Materials, NFPA 801-2008, National Fire Protection Association, 2008.

**UCRL, 1997.** Hossain, Q.A., R.P. Kennedy, R.C. Murray, K. Mutreja, and B.P. Tripathi, Structures, Systems, and Components Evaluation Technical Support Documents, DOE Standard, Accident Analysis for Aircraft Crash into Hazardous Facilities, UCRL-ID-123577, Lawrence Livermore National Laboratory, 1997.

**USNRC, 1978.** Development of Floor Design Response Spectra for Seismic Design of Floor-Supported Equipment or Components, Regulatory Guide 1.122, Revision 1, U.S. Nuclear Regulatory Commission, 1978.

**USNRC, 2007a.** Design Basis Tornado and Tornado Missiles for Nuclear Power Plants, Regulatory Guide 1.76, Revision 1, U.S. Nuclear Regulatory Commission, 2007.

**USNRC, 2007b.** Barrier Design Procedures, NUREG-0800, Subsection 3.5.3, Revision 3, U.S. Nuclear Regulatory Commission, 2007.

**USNRC, 2007c.** Tornado Loads, NUREG-0800, Subsection 3.3.2, Revision 3, U.S. Nuclear Regulatory Commission, 2007.

cell work surfaces to the import and export areas have plugs that are in-place when not actively importing or exporting materials. Plugs have the same shielding capability as the bulk material they penetrate. The penetrations are designed to minimize the potential spread of contamination in accordance with the RPP. The supercell consists of multiple individual hot cells. The hot cells are equipped with manipulators. Manipulator penetrations are designed with supplemental shielding to meet ALARA goals. The hot cells are equipped with lead glass windows. The window design incorporates compensating shielding to reduce streaming at the interface of the window and the hot cell wall. Figure 4b.2-4 shows a general depiction of the entry and exit facilities from the supercell. The supercell is anchored to the main production facility structure (FSTR) and the supercell is seismically qualified to perform its confinement and shielding safety functions during and following a design basis earthquake (DBE).

The biological shield and supporting structures are designed and constructed to remain intact during normal operations as well as during and following design basis accidents.

No neutron fluxes exist in the RPF that could result in activation of groundwater or soils.

## 4b.2.3 SHIELD MATERIALS

The RPF uses the following primary materials in different configurations to assemble the biological shield and meet the radiation exposure goals defined in Chapter 11:

- standard density (minimum 140 pounds per cubic foot [$lb/ft^3$]) (2.2 grams per cubic centimeter [$g/cm^3$]) concrete with reinforcing steel,
- lead,
- steel, and
- lead glass.

The concrete is of the ordinary type, with no special additives for shielding purposes.

The biological shielding for the tank vaults, pipe trenches, carbon delay bed vault, waste drum storage (except upper portions), and portions of the RLWI shielded enclosure is reinforced concrete. Waste drum storage bore hole upper portions (i.e., covers and compensating shielding) are lead. Portions of the RLWI shielded enclosure include steel. The supercell shielding is primarily lead. Lead glass windows are used for viewing purposes. Lead is used for localized shielding around components with high activity. Alternative shielding materials and configurations that provide equivalent or increased shielding properties may be used.

### 4b.2.3.1 Shielding Calculations

Calculations are performed with the software package named MCNP (Monte Carlo N-Particle Transport Code). MCNP is developed and validated by Los Alamos National Laboratory (LANL) and distributed by the Radiation Safety Information Computational Center (RSICC) at Oak Ridge National Laboratory (ORNL) (LANL, 2011). MCNP uses a Monte Carlo based particle (neutrons and photons) transport method to generate a set of particle tracks through a model of the facility geometry. The Monte Carlo method generates a statistical set of results for individual particles transported through the geometry. Enough particles are simulated to obtain statistically significant results. Conservative assumptions are used to define the overall shielding properties of the concrete and reinforcing bar assuming no reinforcing bar is conservative for gamma

# 9a2 IRRADIATION FACILITY AUXILIARY SYSTEMS

## 9a2.1 HEATING, VENTILATION, AND AIR CONDITIONING SYSTEMS

### 9a2.1.1 RADIOLOGICALLY CONTROLLED AREA VENTILATION SYSTEM

The radiological ventilation (RV) systems include supply air, recirculating, and exhaust subsystems required to condition the air and provide the confinement and isolation needed to mitigate design basis accidents. The main production facility utilizes three ventilation systems in the radiologically controlled area (RCA) to maintain the temperature and humidity of the RCA and to progress air from areas of least potential for contamination to areas with the most potential for contamination:

- Radiological ventilation zone 1 (RVZ1)
  - RVZ1 recirculating subsystem (RVZ1r)
  - RVZ1 exhaust subsystem (RVZ1e)
- Radiological ventilation zone 2 (RVZ2)
  - RVZ2 exhaust subsystem (RVZ2e)
  - RVZ2 supply subsystem (RVZ2s)
  - RVZ2 recirculating subsystem (RVZ2r)
- Radiological ventilation zone 3 (RVZ3)

Figure 9a2.1-1 provides the ventilation zone designations within the main production facility.

Chapter 6 provides a detailed description of the SHINE confinement strategy for limiting the potential for release of radioactive materials to occupied spaces and the environment.

#### 9a2.1.1.1 Design Bases

The design bases of the RV systems include:

- Provide confinement at ventilation zone 1 confinement boundaries. See Chapter 6 for a description of the specific portions of the RVZ1 system credited as being a confinement boundary.
- Provide isolation at the RCA boundary. See Section 7.5 for a description of the specific portions of the RVZ1, RVZ2, and RVZ3 systems that provide the isolation functions.
- Confine airborne radiological materials in an accident scenario.
- Provide ventilation air and condition the RCA environment for workers.
- Provide makeup air and condition the RCA environment for process equipment.
- Filter exhaust streams prior to them being exhausted out of the RCA.
- Maintain occupational exposure to radiation as low as reasonably achievable (ALARA) and to ensure compliance with the requirements of 10 CFR 20.
- Exhaust hazardous chemical fumes.

Nonsafety-related portions of the RV systems are constructed to the requirements of Chapters SPS 362, SPS 363 and SPS 364 of the Wisconsin Administrative Code. Nonsafety-related piping is designed, installed, tested and inspected in accordance with American Society of Mechanical Engineers (ASME) B31.9, Building Services Piping (ASME, 2017). The exhaust stack is nonsafety-related and potential failure or collapse of the exhaust stack does not result in an unfavorable interaction with safety-related structures, systems, or components (SSCs).

The high-pressure nitrogen gas storage is contained in integrally forged pressure vessels (i.e., high-pressure nitrogen gas tubes) designed to meet the requirements of ASME Boiler and Pressure Vessel Code (BPVC), Section VIII, Rules for Construction of Pressure Vessels (ASME, 2010). The tubes and associated piping, manual isolation valves, high point vents, low point drains, self-regulating pressure reducing valves, relief valves, check valves, and pressure instrumentation for the supply system are located in the N2PS structure, an above-grade reinforced concrete structure adjacent to the main production facility. The N2PS structure and equipment are designed to remain functional during and following a seismic event. N2PS pipe supports and high-pressure nitrogen tube supports are seismically qualified in accordance with American Society of Civil Engineers (ASCE) Standard 7-05, Minimum Design Loads for Buildings and Other Structures (ASCE, 2006). Additionally, the N2PS structure is designed to withstand the impact of tornado missiles.

The tubes are manifolded so they will act in unison and have a common remote fill connection to allow refill by tanker truck delivery. One redundant high-pressure nitrogen gas tube provides service in the event of the loss of a tube or failure of the associated valves upstream of the common manifold. Each high-pressure nitrogen gas tube and the downstream piping and equipment is protected from overpressure by relief valves discharging to atmosphere above the roof of the structure, through a nonsafety-related vent path.

The N2PS is sized to provide three days of sweep gas flow to tanks containing irradiated target solution in the RPF during a loss of normal power or a loss sweep gas flow. The N2PS also provides three days of sweep gas flow to each TSV dump tank.

#### 9b.6.2.4 Instrumentation and Control

The N2PS includes pressure instrumentation to monitor the function of the self-regulating pressure reducing valves. The nitrogen tube pressure, tube discharge pressures, pressure to the IU cells, and pressure to the RPF tanks are monitored. The pressure instrument output is provided to PICS.

The N2PS includes flow switches on the piping to the IU cells and RPF tanks to provide indication of normal operation when the purge is actuated. The flow switch status is provided to PICS.

N2PS solenoid valves include valve position indication. The position status for each valve is provided to TRPS if it serves the IU cells or to ESFAS if it serves the RPF tanks.

Oxygen sensors are provided in locations near N2PS equipment. The oxygen instruments alert operators locally of an asphyxiation hazard in the event of a nitrogen leak.

TRPS actuates the N2PS purge of the affected IU on loss of normal power to an IU cell after a delay or on loss of flow in TOGS.

ESFAS actuates the N2PS purge of the RPF tanks on loss of normal power to the PVVS or on loss of flow in PVVS.

Manufacturer's Association of America (CMAA) 70, Specifications for Top Running Bridge & Gantry Type Multiple Girder Electric Overhead Traveling Cranes (CMAA, 2004); and ASME NOG-1, Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder) (ASME, 2004).

The IF overhead crane is designed to the following criteria:

- Meet seismic requirements and prevent failures of the crane that could damage safety-related equipment such that the equipment would be prevented from performing its safety function.
- Meet the single-failure-proof design criteria and construction of ASME NOG-1, Type I cranes and be designed to perform as a Service Level B – Light Service crane as described in CMAA 70.
- Secure its load in place upon a loss of power and any fault condition. The hoisting machinery and wire rope reeving system, in addition to other affected components, is designed to withstand the most severe potential overload, including two-blocking and load hang-up.
- Remain in place on the runway girder, with or without a load, during and after a seismic event.

Radioisotope Production Facility Overhead Crane

The radioisotope production facility (RPF) overhead crane is designed to meet the applicable requirements of ASME B30.2 (ASME, 2011a), CMAA 70 (CMAA, 2004), and ASME NOG-1 (ASME, 2004).

The RPF overhead crane is designed to the following criteria:

- Meet seismic requirements and prevent failures of the crane that could damage safety-related equipment such that the equipment would be prevented from preforming its safety function.
- Meet the design criteria and construction of ASME NOG-1, Type II cranes and be designed to perform as a Service Level B – Light Service crane as described in CMAA 70.
- Remain in place on the runway girder, with or without a load, during and after a seismic event.

9b.7.2.2 System Description

Irradiation Facility Overhead Crane

The IF overhead crane is a 40-ton, double girder, bridge style crane designed for the handling of shield cover plugs and equipment such as neutron drivers and process skids inside the IF. The IF overhead crane is designed to span the width of the IF and travel the length of the IF.

The use of a single-failure-proof crane with rigging and procedures that implement the requirements of NUREG-0612, Control of Heavy Loads at Nuclear Power Plants (USNRC, 1980) assures that the potential for a heavy load drop is extremely small, and therefore, analysis of the potential effects of heavy load drops are not required.

## 9b.8 REFERENCES

**ANSI, 1993.** Radioactive Materials – Special Lifting Devices for Shipping Containers Weighing 10,000 Pounds (4500 kg) or More, ANSI N14.6-1993, American National Standards Institute, 1993.

**ANSI/ANS, 1998.** Guide for Nuclear Criticality Safety in the Storage of Fissile Materials, ANSI/ANS 8.7-1998 (R2007), American National Standards Institute/American Nuclear Society, 1998.

**ASCE, 2006.** Minimum Design Loads for Buildings and Other Structures, ASCE 7-05, American Society of Civil Engineers, 2006.

**ASME, 2004.** Rules for Construction of Overhead and Gantry Cranes (Top Running Bridge, Multiple Girder), NOG-1-2004, American Society of Mechanical Engineers, 2004.

**ASME, 2009.** Code on Nuclear Air and Gas Treatment, AG-1-2009, American Society of Mechanical Engineers, 2009.

**ASME, 2010.** Rules for Construction of Pressure Vessels, Boiler and Pressure Vessel Code, Section VIII, Division 1, American Society of Mechanical Engineers, 2010.

**ASME, 2011a.** Overhead and Gantry Cranes (Top Running Bridge, Single or Multiple Girder, Top Running Trolley Hoist), B30.2-2011, American Society of Mechanical Engineers, 2011.

**ASME, 2011b.** Building Services Piping, B31.9-2011, American Society of Mechanical Engineers, 2011.

**ASME, 2013.** Process Piping, B31.3-2012, American Society of Mechanical Engineers, 2013.

**ASME, 2018.** Slings, B30.9-2018, American Society of Mechanical Engineers, 2018.

**CMAA, 2004.** Specifications for Top Running Bridge & Gantry Type Multiple Girder Electric Overhead Traveling Cranes, CMAA 70-2004, Crane Manufactures Association of America, Inc., 2004.

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