Seismic and Structural Evaluation of Philippine Research Reactor-1 Structures

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Abstract

The Philippine Research Reactor 1 (PRR-1), operated by the Philippine Nuclear Research Institute (PNRI), is the first and only nuclear facility that has operated in the country until its shutdown in 1988. The civil structures of PRR-1 were originally designed for a peak ground acceleration of 0.2 g and the response of these structures in the event of an earthquake is a major concern from the viewpoint of operational safety considering that the structures are more than 60 years old. In support of the planned reoperation of the facility with the Subcritical Assembly for Training, Education, and Research (SATER), PNRI contracted the services of a private company to undertake the structural evaluation of the existing facilities. The contract included geotechnical investigation and seismic hazard analysis, which are reported in this paper. Key results demonstrated that the PRR-1 structures are suitable for operating SATER and are classified as Hazard Category 4. Moreover, the findings from the evaluation will serve as the basis for the retrofitting and refurbishment activities that will be conducted for the PRR-1 facility to meet the life-safety performance objective of SATER.

***Keywords**:* geotechnical, PRR-1, SATER, seismic hazard analysis, structural evaluation

1. Introduction

The Philippine Research Reactor-1 (PRR-1) at the Philippine Nuclear Research Institute (PNRI) was the first and only nuclear facility that was successfully operated in the Philippines from 1963 to 1984 but it has been shut down since 1988 (Asuncion-Astronomo *et al.*, 2019a). Since then, PNRI is in possession of slightly irradiated Training, Research, Isotope, General Atomics (TRIGA) fuel rods that have been in wet storage for 30 years. In an effort to revive the nuclear science knowledge and augment nuclear expertise in the country, its availability has prompted PNRI to reuse the PRR-1 TRIGA fuel in a Subcritical Assembly for Training, Education, and Research (SATER) to build capacity in nuclear science and technology (Asuncion-Astronomo *et al.*, 2019b). Since the facility is already more than 60 years, the response of PRR-1 structures in the event of an earthquake is considered a major concern from the viewpoint of operational safety. The civil structures of PRR-1 were originally designed taking into consideration the lateral forces of earthquake corresponding to an acceleration of 0.2 gravity (General Electric Company, 1960). In preparation for the operation of the subcritical assembly that will make use of the existing PRR-1 facility, structural evaluation is necessary to ensure the adequacy and to ascertain the integrity of PRR-1 structures. The evaluation will ensure operational safety for the continued use of the PRR-1 structures considering the site's proximity to the valley fault system (VFS).

Recently, resistance to seismic loading has been considered significantly with numerous programs being undertaken in many countries since several structures have been damaged by large earthquakes throughout history (International Atomic Energy Agency [IAEA], 2001, 2003; Králik, 2017). The evaluation of the seismic safety of nuclear facilities, including research reactors, has assumed importance following the accident at the Fukushima-Daiichi nuclear power plant (Katona, 2017). This is particularly important for old facilities especially nuclear power plants and high-power research reactors that are more susceptible to aging issues. These structures are considered the ultimate physical barrier between the reactor core and the public in general. Seismic evaluation is highly relevant in the case of PRR-1, which is being repurposed for a subcritical reactor even though the very low hazard associated with the operation of the subcritical facility and aging will have minimal to no effect on its safety-related functions (IAEA, 2019; Astronomo and Marquez, 2021).

The facility is located within a region that is tectonically and seismically active as it is just outside the Philippine Mobile Belt and is sandwiched between the Manila Trench on the west, Philippine Infanta Fault in the east and the Lubang Fault in the south (Bautista and Oike, 2000; AMH Philippines, Inc., 2020a). Analyzing the seismic hazard at the site can be a means to mitigate other seismic hazards brought about by ground shaking (e.g., landslides, liquefaction and lateral spreading). There are existing structures within the PRR-1 facility where the SATER tank assembly will be established. Thus, it is necessary to provide hazard levels to individual structural members since seismic performance objectives may vary from one structure to another.

This paper presents an overview of the structural evaluation and assessment that were conducted by the external contractor (AMH Philippines, Inc.). The contents were based on the synthesis of the consultant's final reports and limited to the following existing PRR-1 structures: fuel storage tank and work platform assembly, reactor pool and the reactor building (AMH Philippines, Inc., 2020b). Key results of the seismic hazard analysis and geotechnical investigation (AMH Philippines, Inc., 2020a, 2020c) were presented and the main findings on the evaluated structures will become the basis for the most suitable strengthening and retrofitting scheme appropriate for the existing condition of the building.

2. Methodology

According to the scope of work prepared by PRR-1 operators, the contractor performed the evaluation and assessment of PRR-1 structures based on the latest national building code, which included the National Structural Code of the Philippines (NSCP) (2015) (AMH Philippines, Inc., 2020b). Other relevant codes and standards were also applied, as necessary.

2.1 Seismicity and Soils

Over the past 400 years, Metro Manila, Philippines has been affected by several large earthquakes (Nelson *et al.*, 2000). However, seismological records indicate seismic shocks of minor intensity in the Diliman area. (Bautista and Oike, 2000). The active VFS, which cuts through the Greater Metro Manila, consists of the West and East Valley Faults. Paleo-seismic studies at the West Valley Fault, based on carbon-14 dating, estimated a recurrence interval of high-magnitude earthquakes between 200 to 400 years during the last 1,400 years (Daligdig *et al.*, 1997). Other studies reported a recurrence interval of 400 to 600 years (Nelson *et al.*, 2000). A vertical-to-horizontal ratio ranging from 0.26 to 0.56 was computed for the West Valley Fault (Rimando and Knuepfer, 2006) with a lateral ground acceleration.

The PRR-1 SATER site is generally underlain by the Late Pliocene to Early Pleistocene Guadalupe Tuff Formation (GTF) – the regional bedrock of Metro Manila (Gervasio, 1968). These rock formations are in the category of very soft rock to hard/very dense soil which is relatively less affected by seismic shock than the alluvial formations in the Manila area (Bureau of Mines and Geosciences, 1982; National Environmental Protection Council, 1987; Dela Paz *et al.*, 1989).

Further investigations, which included the drilling of one borehole at a depth of 11.22 m at WGS84 Zone 51N coordinates 290,604 m E and 1,621,680 m N, revealed that the subsurface of the area has a highly weathered layer of very dense sand underlain by siltstone layers. The rock layers have rock quality designation (RDQ) values of 10 to 17% leading to the assumption that the siltstone layers have high cohesion at 15 kPa. For the area's site class, the average measured shear wave velocity (V_{S30}) of 760.0 m s^{-1} of the top 30 m of the soil profile is accepted (NSCP, 2015). This value is within the lower boundary of the site class S_B (rock) consistent with both the regional geology and the results of the geotechnical investigation. Geotechnical investigations showed that the subsoil condition in the PRR-1 building site has a bearing capacity of 900 kPa assuming an isolated foundation system with a depth of 1.5 m and width of at least 1 m (AMH Philippines, Inc., 2020c).

2.2 Structure Descriptions

The Philippine Research Reactor (PRR-1) is an open pool-type nuclear research reactor obtained from the government of the United States under the Atoms for Peace Program. Its regular operation began in 1963 at an original rated power of 1 MW. Aging and obsolescence issues were encountered in the late 1970s, especially with reactor instrumentation, leading to the complete replacement of PRR-1 instrumentation in 1980 (Leopando, 2004).

From 1984 to 1988, the rated power of PRR-1 was successfully raised to 3 MW after conversion. However, a liner leak in the reactor pool along with other aging reactor components and limited funding for full rehabilitation prevented the resumption of operations. As a result, the PRR-1 has been in extended shutdown for more than three decades while the slightly irradiated TRIGA fuel rods have been stored underwater in a fuel storage tank. Based on current needs and resources, it was decided that the fuel rods will be utilized in a subcritical reactor assembly. The fuel storage tank is located next to the reactor pool and also adjacent to the SATER tank assembly. The layout of the structures inside the PRR-1 reactor building is shown in Figure 1.

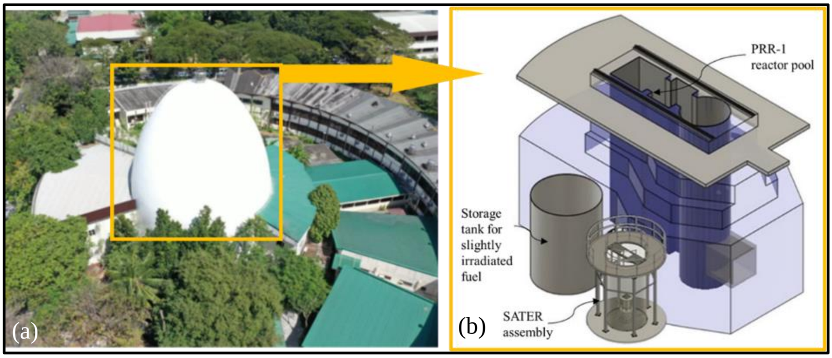

Figure 1. Location of the SATER tank assembly relative to the fuel storage tank (a) and the former reactor pool inside the PRR-1 building (b)

2.3 Design Criteria

2.3.1 Structure

The results presented in this paper covered the following structures within the PRR-1 building: (1) the fuel storage tank and work platform assembly, (2) the former reactor pool and (3) the reactor building. Idealized models of the structures are shown in Figure 3 and the orientation of the fuel storage tank and reactor pool inside the reactor building is shown in Figure 1b. Detailed descriptions of the structures are provided in the following sections.

The fuel storage tank with work platform assembly consists of a 4.5-mm thick and 4.88-m tall SS304 tank that is 3.66 m in diameter with radial L2 x 2 x 1/8” angle stiffeners at heights of 0.61, 1.22, 1.83, 2.44, 3.66 and 4.88 m from the 4.06-m diameter, 50-mm thick concrete base. The work platform assembly is composed of 11 Pipe8SCH30 columns: eight inner and three outer columns, which support the 3-mm thick work platform surrounded by railings with an outer and inner diameter of 60.325 and 52.502 mm, respectively. L2 x 2 x 1/8” angles extend radially under the platform between the pipe columns. The steel stairs constitute C7 x 14.5 stringers and seven stair threads of 3-mm thickness.

The former reactor pool has massive concrete walls that previously served as the reactor biological shield with thickness ranging from 0.45 to 3.0 m from top to the bottom. The reactor pool houses the thermal column, cooling tank, 6- and 8-in beam ports, pneumatic tubes, cooling pipes, demineralizer pipes and pool overflow pipes, among others. The structure stands 9 m and is embedded 2 m from the reactor bay floor level. The top of the reactor pool is accessible through the reactor bridge connected to the second floor of the reactor building.

The PRR-1 building is divided into the east and the west wing, and the central reactor dome. It is designed using reinforced concrete walls supported by strip footing with a spherical roof of 100-mm thickness. The rebars are set apart such that the spacing increases gradually from the top to the bottom of the sphere. The thickness of reinforced concrete walls of the main structure is 300 mm with 16 mm diameter reinforcement bars spaced horizontally and vertically at 300 mm. A polar crane is situated at the rim of the dome supported by concrete corbels. It is used to carry equipment with a maximum load of 10 tons.

2.3.2 Codes and Specifications

Considering that the SATER facility will operate at zero power and in subcritical configuration, a hazard category – four classifications were used in accordance with the IAEA safety requirements (IAEA, 2019). A graded approach was likewise implemented for the equivalent seismic categorization for the system, structure and components (SSC) (i.e., conventional design codes were applied following the national practice for seismic design of non-nuclear applications) (IAEA, 2019). The following structural codes and specifications were used in the structural evaluation and assessment: National Structural Code of the Philippines (NSCP) (2015), American Petroleum Institute (API) 650 (API, 2012), American Concrete Institute (ACI) 349-01 (ACI, 2001), and American Concrete Institute (ACI) 318-14 (ACI, 2014).

2.3.3 Loading

Dead Load

Dead loads consisted of the weight of all materials permanently incorporated into the structure. Table 1 lists the dead load values adopted for the evaluation of structural components.

Live Load

Live loads underwent variation in magnitude and distribution over different time ranges, and these were the maximum loads expected by intended use. The minimum values of these loads depended on the occupancy and were specified by the governing codes. Table 2 shows the values of live loads adopted in the structural assessment of the structures based on the prescribed values of the NSCP (2015) section 205.

Table 1. Dead load parameters

Material	Weight
Concrete	24.0 kN/m^3
Steel	77.0 kN/m^3
Storage tank cover	1.072 kN
Tank contents	7.073 kN
6” CHB	3.30 kN/m^3
8” CHB	4.45 kN/m^3
Polar crane	26.37 tonnes

Table 2. Live load parameters

Use/occupancy	Uniform load
Work platform	1.90 kPa
Stairs	4.80 kPa
Railings	0.89 kPa
Catwalk for maintenance access	1.90 kPa
Multi-purpose room, control room, office, toilet	2.4 kN/m^2
General laboratory	2.9 kN/m^2
Lobby, roof deck	4.8 kN/m^2
Mechanical and electrical room, storage	6.0 kN/m^2
Polar crane live load	9 tonnes

Fluid Load

The fluid load involved the hydrostatic force exerted by fluids with well-defined pressure. A liquid-retaining structure such as the fuel storage tank carried the fluid pressure was taken as the fluid’s unit weight times the height. The considered fluid for the fuel storage tank was water at 20 °C with a unit weight of 9.81 kN/m^3. Fluid loads were not considered for the reactor pool and building structures.

Seismic Load

The PRR-1 site is not transected by any seismic source. However, the active West Valley Fault is only about 3.6 km from the project site. Hence, a site-specific study was performed to quantify the appropriate seismic design forces on-site. Near-source effects were accounted for in the development of the seismic hazards.

The seismic load parameters for the response spectrum analysis pertinent to seismic hazard analysis were in accordance with API 650 annex E (API, 2012) and NSCP (2015), listed in Table 3.

Table 3. Seismic load design parameter (API, 2012; NSCP, 2015)

Fuel storage tank	Value/ remark	Reactor pool	Value/ remark
Importance factor (I)	1.0	Importance factor (I)	1.5
Soil profile (Sp)	SB	Seismic source type	Type A
Seismic zone factor (Z)	0.4	Soil profile (Sp)	Type 2 or SB/rock
Near source factor (Na)	1.34	Seismic zone factor (Z)	0.4
Near source factor (Nv)	1.78	Near source factor (Na)	1.34
Seismic response modification: Coefficient (R)	2.2	Near source factor (Nv)	1.78
Response modification factor: Convective component (Rc)	2.0	Seismic coefficient (Ca)	0.536
Response modification factor: Impulsive component (Ri)	4.0	Seismic coefficient (Cv)	0.715
		Resistance factor (R)	1.0

2.3.4 Loading Combinations

Allowable stress design (ASD) was utilized for the plate element and reactor pool with corresponding response modification factors specified in API 650 annex E while load and resistance factor design (LRFD) was utilized in the analysis and design of the rest of the structure. Load combinations applied to the rest of the structure are based on NSCP (2015).

2.3.5 Material Strength

The material strength properties used in the analysis were based on the PRR-1 final report on materials sampling and testing as listed in Table 4 (AMH Philippines, Inc., 2020d).

Table 4. Material strength properties

Material	Property	Value (MPa)
Steel (storage tank)	Minimum yield strength (F_y)	205
	Ultimate tensile strength (F_u)	515
Concrete	Compressive strength ($f'c$)	17
Steel (reactor pool and building)	Minimum yield strength (F_y)	275
	Ultimate tensile strength (F_u)	480

2.4 Model Analysis

The structural assessment of PRR-1 facilities followed the workflow diagram in Figure 2. The fuel storage tank and its work platform assembly, the reactor pool and the reactor building were modeled using finite element analysis (FEA) carried out through an elastic approach and were analyzed using STAAD.Pro CONNECT Edition version 22.02.00.26 (Bentley Systems).

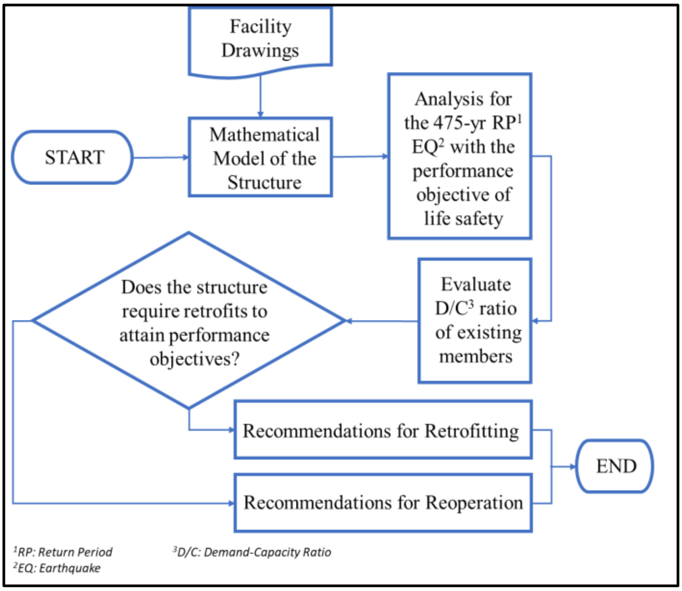

Figure 2. Workflow diagram for structural assessment

The fuel storage tank and work platform assembly were modeled as a hybrid of connected surface and line elements while the reactor pool was modeled as solid elements with concrete material properties obtained from the material testing results. A mathematical model for the reactor building composed of beam elements and plate elements was also created to investigate the behavior of the structure based on the as-built plans.

Through ASD, the limit state for bending was considered critical for the steel plate elements, and appropriate safety factors were employed as strength reductions with reference to NSCP (2015). The stress distribution in concrete elements was determined through principal stresses and the bending moments were computed from normal stresses to evaluate the adequacy of existing reinforcing steel bars in the structure. The idealized models of these structures (Figure 3) were subjected to the prescribed loads and loading combinations. Evaluation of the demand-capacity (D/C) ratio of the various structural members was done through the LRFD method with respect to the provisions set in the NSCP (2015).

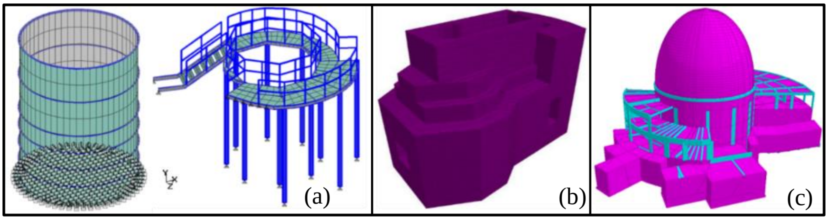

Figure 3. STAAD model of the fuel storage tank and platform (a), reactor pool (b) and reactor building (c)

For the seismic hazard analysis, spectral accelerations used were extracted from the individual response spectra associated with ground motions with return periods or recurrence intervals of 100, 475, 1,000, 2,475, 4,975, 10,000, and 100,000 years. All response spectra were generated for a damping ratio of 5%; although for steel structures, 2% of damped response spectra were provided. For PRR-1 existing structures, an elastic analysis was carried out through the response spectrum analysis (RSA) to capture a more realistic response. In the case of the reactor pool, an additional static equivalent lateral force (ELF) analysis was implemented.

3. Results and Discussion

3.1 Fuel Storage Tank and Work Platform Assembly

Seismic analysis of the structure was carried out in accordance with API 650 annex E through the application of hydrodynamic forces on the tank, namely:

lateral inertial pressure on the wall (Pw), hydrodynamic impulsive pressure (Pi) and hydrodynamic convective pressure (Pc).

The liquid-containing tank subjected to seismic loading carried loads that were divided into the contributions previously listed. Spectral response analysis (SRA) was included as part of the total seismic loading. As shown in Figure 4, an estimate of 475-year earthquake return period and 5% damping was selected under a Hazard Category (HC) of HC-4 in accordance with IAEA safety requirements (IAEA 2003, 2019).

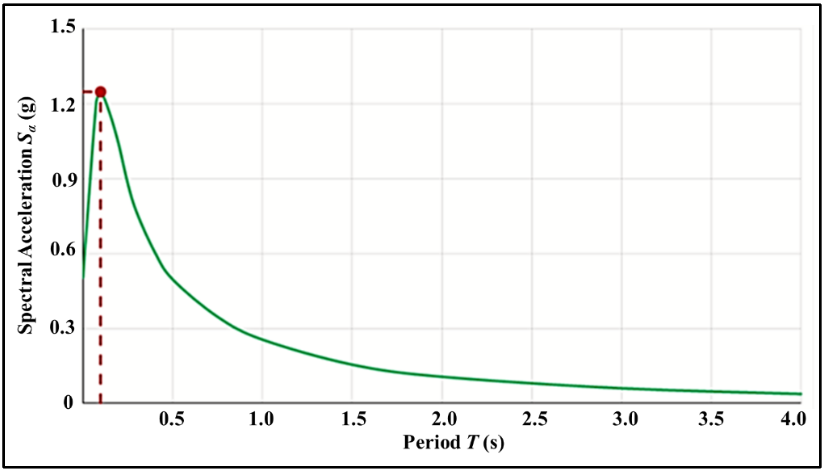

Figure 4. 475-year return period earthquake response spectrum curve for damping (ζ = 5%)

The earthquake force induced the lateral inertia force of one accelerating wall of the tank shell. The impulsive component was caused by the inertia of the liquid retained by the rigid tank. The convective component involved sloshing and fluid vibration in the tank as shown in Figure 5.

Visual inspection of the fuel storage tank showed no findings that compromise its structural integrity in its current condition. However, by investigating the most critical load combination, the principal stress developed in the plates of the fuel storage tank under the seismic load combination (D + F + E/1.4) peaked at a maximum compressive principal stress of 11.5 MPa at the tank shell near the base (Figure 6). In addition, the maximum compressive base pressure developed was shown to reach a value of 58 kPa (Figure 6), while the maximum horizontal resultant displacement for the fuel storage tank for seismic load combination was found close to where the pressure was greatest with a displacement magnitude of 0.322 mm. The maximum and minimum support reactions occurred for the seismic load combination at the pin supports at the base. These amounted to around 15.3 and -5.3 kN, respectively.

In the fuel storage tank work platform, the most critical case for each parameter was the gravity load combination (D + L) where maximum stress of 59.78 MPa developed near the stairs of the assembly (Figure 7). The maximum computed horizontal displacement of the work platform was around 0.6 mm, while the maximum computed vertical deflection for the stairs was about 3.4 mm.

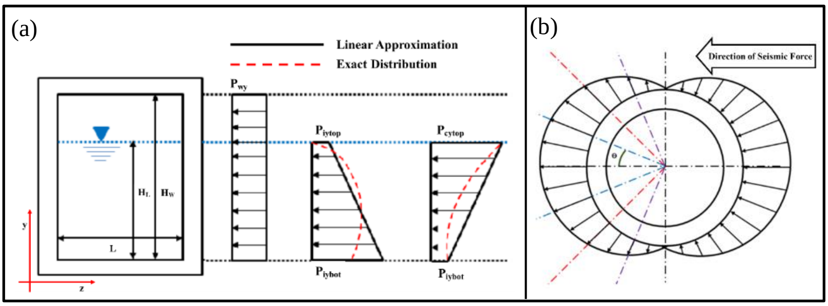

Figure 5. Lateral (a) and radial (b) seismic force distribution in the fuel storage tank

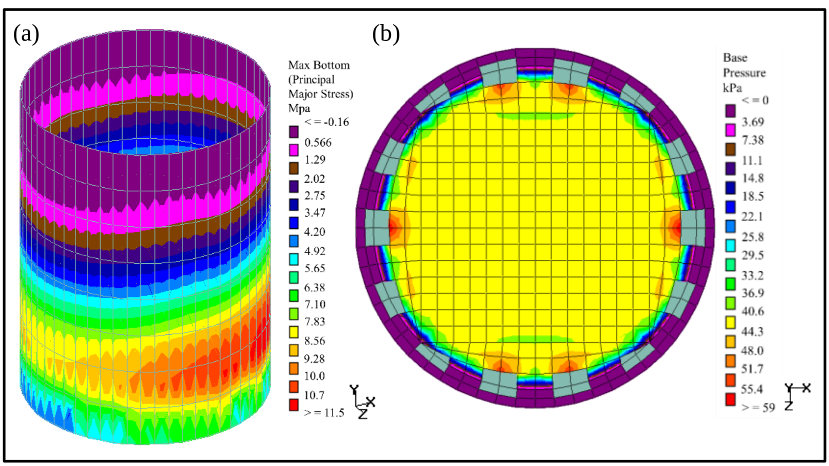

Figure 6. Plate stress contour of maximum principal stress developed for 4.5-mm thickness (a) and maximum base pressure (b)

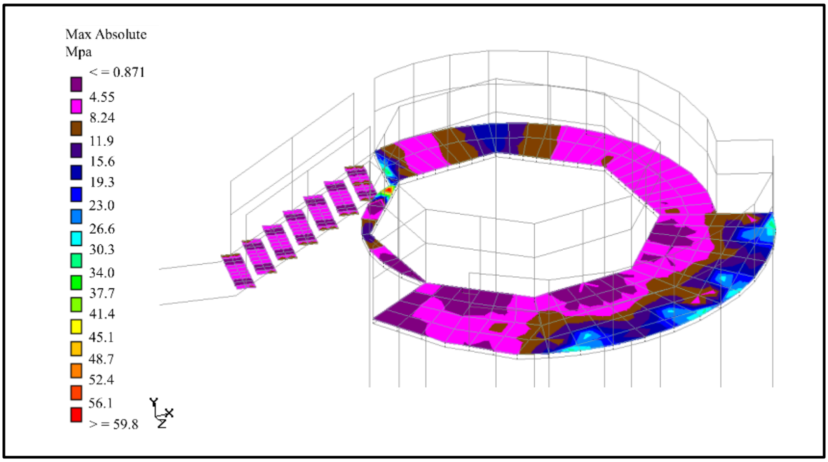

Figure 7. Plate stress contour of fuel storage tank work platform

The maximum and minimum stresses developed in the 4.5-mm thick tank and work platform are summarized in Table 5.

Table 5. Summary of maximum stresses for the fuel storage tank and work platform

Stressed develop (MPa)		Allowable stress (MPa)		References
		Fuel storage tank		
Extrema of principal stress (max/min)	11.5/-7.0	Flexural yielding strength requirement (FS = 1.67)	122.75	NSCP (2015)
		Local buckling strength requirement (FS = 1.67)	34.28	NSCP (2015)
Maximum Shear Stress	0.16	Shearing strength requirement (FS = 1.67)	12.35	NSCP (2015)
		Work platform		
Extrema of principal stress (max/min)	59.8/-32.2	Flexural yielding strength requirement (FS = 1.67)	122.75	NSCP (2015)
Maximum shear stress	7.9	Shearing strength requirement (FS = 1.67)	12.35	NSCP (2015)

The factored load combination with the most critical effect on each member for strength was used to determine utilization ratios which are tabulated in Table 6 for each structural member in the assembly. Every critical member was shown to have a D/C ratio of less than 1.0.

Table 6. Tank and work platform with stairs D/C ratio

Group	D/C Ratio
Angle stiffeners	$0.239 \leq 1.0$
Work platform pipe columns	$0.094 \leq 1.0$
Work platform angles	$0.904 \leq 1.0$
Work platform I-beams	$0.339 \leq 1.0$
Work platform railings	$0.317 \leq 1.0$
Stair stringers	$0.316 \leq 1.0$
Stair railings	$0.375 \leq 1.0$

The actual thickness of 4.5 mm resulted in a maximum principal major stress of 11.50 MPa, which was significantly lower than the allowable stress. Nevertheless, the NSCP (2015) provision for stability and limiting slenderness of the tank still requires an increase of the thickness to 8.5 mm.

3.2 Reactor Pool

The SRA and the equivalent lateral force (ELF) procedures were considered in the seismic hazard analysis of the reactor pool. The 475-year response spectrum was extracted employing the complete quadratic combination (CQC) modal combination method (Figure 8). A 2% damping was used in the structural analysis considering the relatively rigid nature of the structure, thereby allowing only a limited amount of damping.

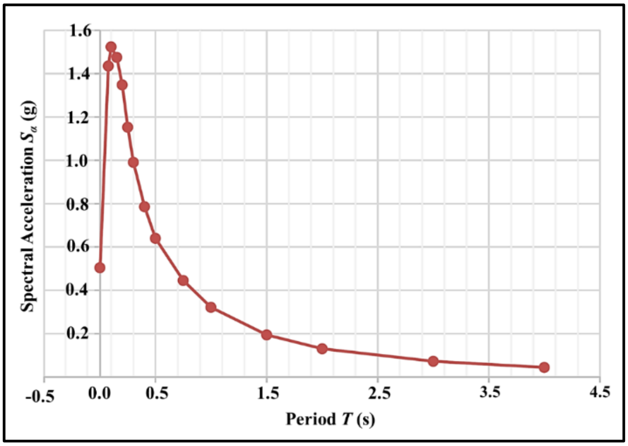

Figure 8. 475-year return period earthquake response spectrum curve for damping (ζ = 2%)

Seismic forces were determined based on the equivalent static force procedure and computed following the provisions of NSCP (2015). The nearest seismic source to the project site is the west VFS with an approximate distance of 3.6 km to the site. Based on the geotechnical report, the site is Type 2 or SB/rock soil classification as listed in Table 3. Since the reactor pool will not be used for SATER, it was analyzed at no fluid condition and with minimum total service forces assumed to act non-concurrently on the direction of each of the main axes of the structure.

The most critical load combination was checked to determine the maximum deformation and stress in the structure for a 475-year return period. Maximum results were obtained under the seismic load combination (1.0 D + 0.7143 E [Z]). An exaggerated illustration of the deformed shape of the structure is shown in Figure 9 and the resulting maximum deformations per direction are shown in Table 7. Overall, maximum deformations of 6.684 and 3.414 mm along the z-direction were obtained using the SRA and ELF approaches, respectively.

Table 7. Deformation using 475-year STAAD model

	SRA (mm)	ELF (mm)
Max X	1.188	0.867
Max Y	1.283	0.628
Max Z	6.684	3.414

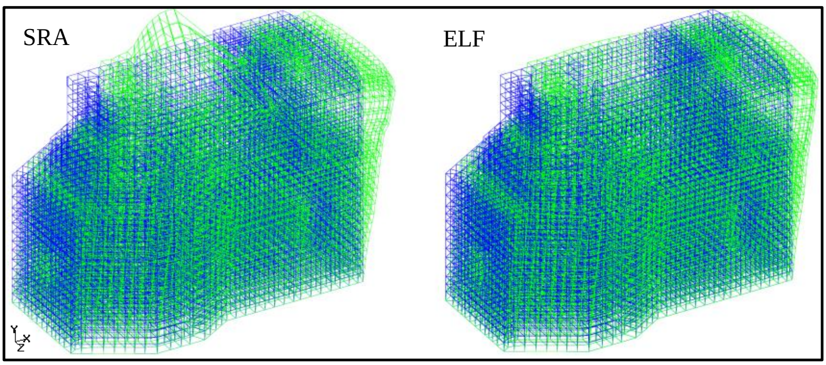

Figure 9. Deformation using 475-yr STAAD model

To show the stress distribution throughout the elevation of the structure, the reactor pool was sectioned and labeled in four parts (Figure 10). The results of the principal stress contour diagram obtained using the most critical load combination are shown in Figure 11 and the maximum values for principal stress per section for ELF and SRA are exhibited in Table 8. The values for the D/C ratio of the concrete tensile stress all exceeded the limit of 1.0, showing that the tensile stresses surpassed the allowable value of 2,060 kPa for all sections of the structure for both the ELF and SRA seismic conditions. Using the NSCP static load analysis, ELF and SRA results did not exceed the allowable compressive concrete stress of 7,650 kPa at any section of the structure. The dissimilarity in the resulting deformations and stresses from SRA and ELF methods was due to the difference in the structure's response under each method. The ELF method considered only one mode of vibration and a linear idealization of force distribution along with the height of the structure, while the SRA took the structural dynamics in which numerous modes contribute.

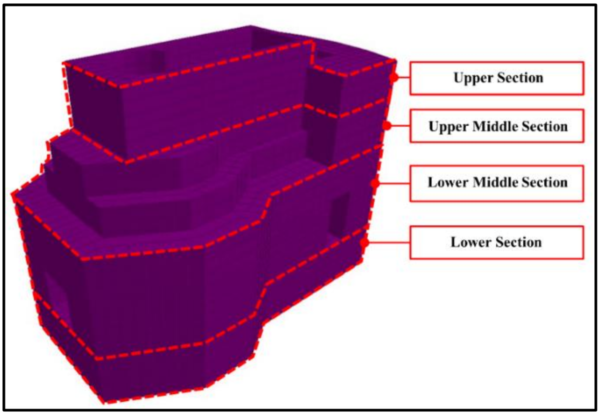

Figure 10. Reactor pool sections

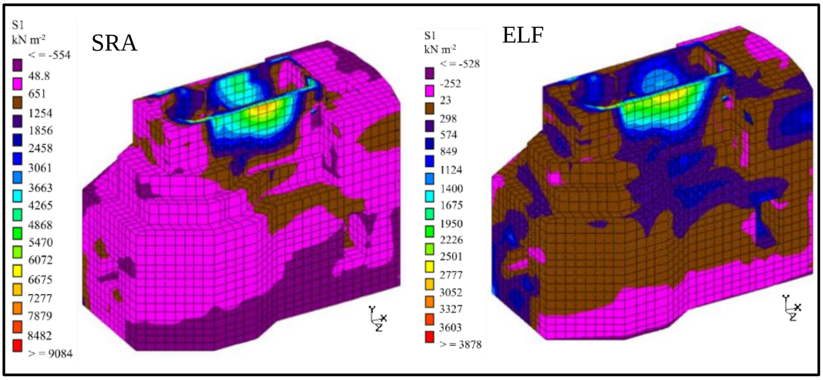

Figure 11. Principal stress contour diagram

Table 8. Resulting stresses (ELF and SRA) for the reactor pool (at no fluid condition)

Section	Allowable tensile (T)/ compressive stress (C) (kPa)	Max/min principal stresses (kPa)	ELF D/C	Max/min principal stresses (kPa)	SRA D/C
Upper	2060.00 (T) 7650.00 (C)	3187.16 -2397.69	1.55 0.31	7193.87 -6408.37	3.49 0.84
Upper middle	2060.00 (T) 7650.00 (C)	2049.94 -1864.08	0.99 0.24	6295.36 -6092.18	3.05 0.8
Lower middle	2060.00 (T) 7650.00 (C)	2979.65 -3350.54	1.45 0.44	3134.87 -3899.07	1.52 0.51
Lower	2060.00 (T) 7650.00 (C)	3135.47 -3869.76	1.52 0.51	3213.77 -3940.5	1.56 0.52

The allowable compressive stress was not expected to exceed at no fluid condition. However, the stresses could significantly exceed the allowable tensile stresses and the reactor pool would be anticipated to experience some cracking according to the spectral response analysis considering the code-prescribed 475-year return period. Life safety performance objective could still be achieved since tension reinforcement provided is sufficient. Nevertheless, retrofitting the outer top surface of the reactor pool wall introduces a high tensile strength to the structure.

In addition, reinforcement bars were also evaluated by computing bending moments from normal stresses at critical areas of the reactor pool. The required spacings for the reinforcing bars were computed and compared against the existing bars found in the as-built drawings given the bending moment. The computation for the required spacing considered the same number of pieces and diameters as the existing bars. Reinforcement bar spacings were found to be adequate throughout the structure. While the reinforcing bars seemed enough for the internal forces, an additional horizontal tensile support at the top left and right walls of the reactor pool should be provided. The large tensile stresses in this region could result in inordinately large cracks at design level earthquake of the 475-year return period.

3.3 Reactor Building

The 475-year response spectrum extracted from the seismic hazard analysis using the CQC modal combination method was employed in the structural assessment of the reactor building (Figure 12). A 5% damping was used in the structural analysis as prescribed for reinforced concrete structures by the NSCP (2015).

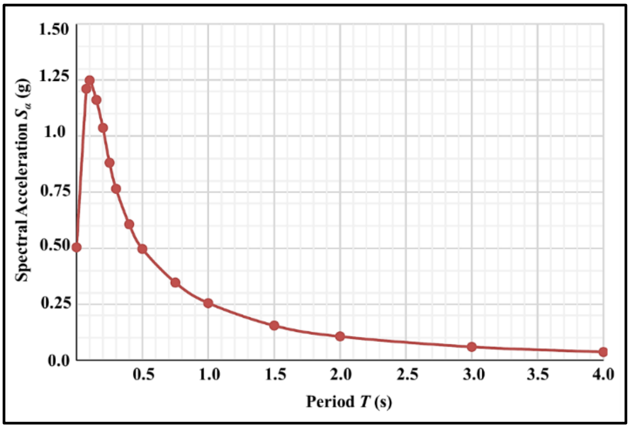

Figure 12. 475-year return period earthquake response spectrum curve for damping (ζ = 5%)

The analysis of the reactor building indicated that only selected structural members are inadequate when subjected to an earthquake with a 475-year return period based on the provisions of the NSCP (2015). Before the analysis, the walls were grouped based on location, thickness and existing reinforcing bars as shown by the wall evaluation key map in Figure 13.

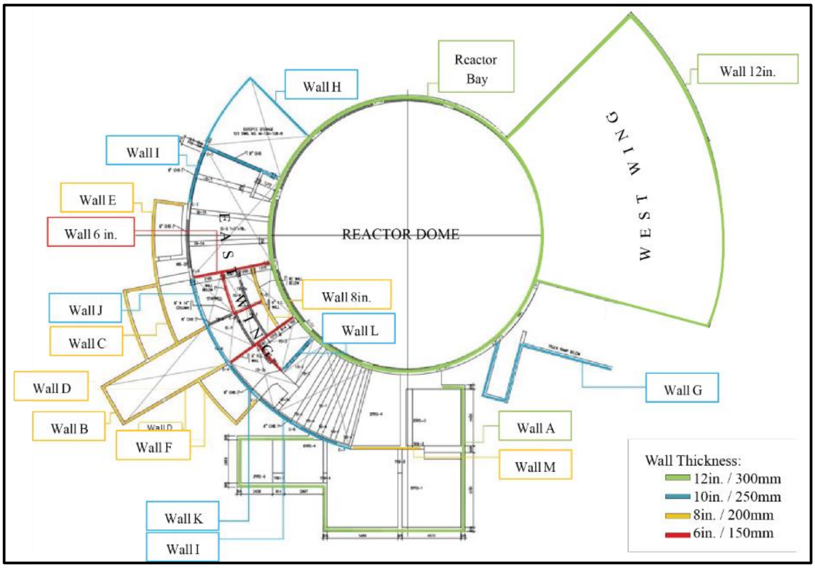

Figure 13. Wall evaluation key map

Generally, the existing reinforcing bars are sufficient in providing the required tensile strength to the walls except for the portion of Wall B at PBS-3 basement level. Wall B exhibited a deficiency in flexural capacity when lateral loads were applied to the structure. The wall reinforcement summary is shown in Table 9.

Table 9. Wall reinforcement summary

		Existing main reinforcement			STAAD Analysis		
Wall	Location	Wall thickness (mm)	Diameter (mm)	Spacing (mm)	Required spacing (mm^2/mm)	Required spacing (mm)	Remarks
	Reactor bay	300	16	300	0.6	335	Passed
A	TFS	300	16	300	0.6	335	Passed
B	PBS-3	200	12	250	0.605	187	Failed
C	PBS-1/PBS-2 /TBS-1	200	12	250	0.4	283	Passed
D	PBS-2	200	12	250	0.416	272	Passed
E	TBS-1	200	12	250	0.4	283	Passed
F	SBS-1	200	12	250	0.4	283	Passed
G	TEW	250	12	200	0.5	226	Passed
H	Isotope storage	250	12	200	0.5	226	Passed
I	-	250	12	200	0.5	226	Passed
J	TBS-1	250	12	200	0.5	226	Passed
K	SBS-1	250	12	200	0.5	226	Passed
L	BS-2	250	12	200	0.5	226	Passed
M	TFS-2	200	12	250	0.4	283	Passed
6 in	-	150	12	250	0.4	283	Passed
8 in	-	200	12	250	0.4	283	Passed
12 in	-	300	16	300	0.6	335	Passed

The dome of the reactor building is separated into segments, which were identified based on existing reinforcement bars in Figure 14. The resulting stresses and D/C ratio revealed that at one segment, segments one to two, the allowable tensile capacity at the outer face of the dome was exceeded by a 1% margin (Table 10). The principal stress contour diagram is shown in Figure 15. Retrofitting could provide additional tensile strength to the specified critical segment of the dome. Additionally, it was determined that the existing reinforcing bars at the dome are adequate as shown in Table 11.

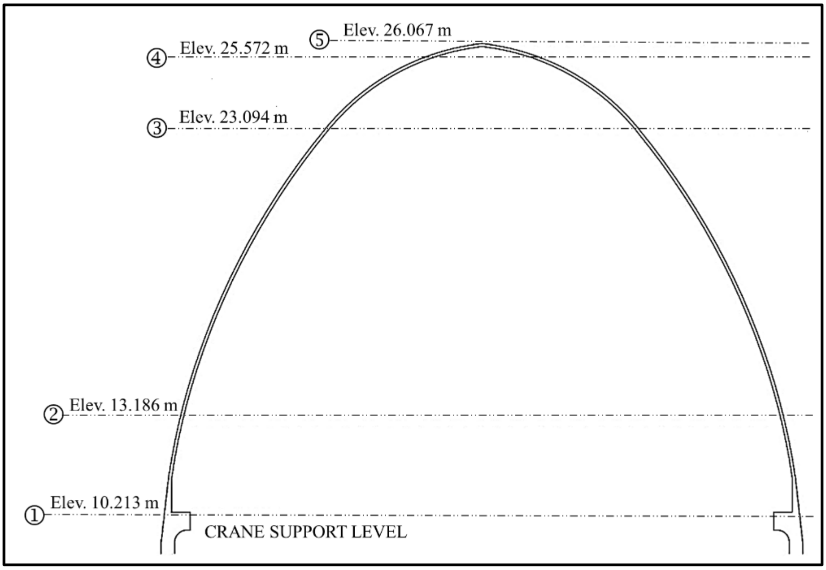

Figure 14. Dome segments

Table 10. Dome stresses summary

Segment	Stress	Surface	Allowable (kPa)	Actual (kPa)	D/C Ratio
1 - 2	Max principal	Outer	2060	2083.690	1.01
	Min principal	Outer	7650	-2954.013	0.39
	Max principal	Inner	2060	1769.631	0.86
	Min principal	Inner	7650	-2205.963	0.29
2 - 3	Max principal	Outer	2060	1438.196	0.70
	Min principal	Outer	7650	-1718.549	0.22
	Max principal	Inner	2060	1623.646	0.79
	Min principal	Inner	7650	-1634.392	0.21
3 - 4	Max principal	Outer	2060	648.544	0.31
	Min principal	Outer	7650	-750.556	0.10
	Max principal	Inner	2060	692.977	0.34
	Min principal	Inner	7650	734.291	0.10
4 - 5	Max principal	Outer	2060	139.507	0.07
	Min principal	Outer	7650	-319.021	0.04
	Max principal	Inner	2060	163.763	0.08
	Min principal	Inner	7650	-291.857	0.04

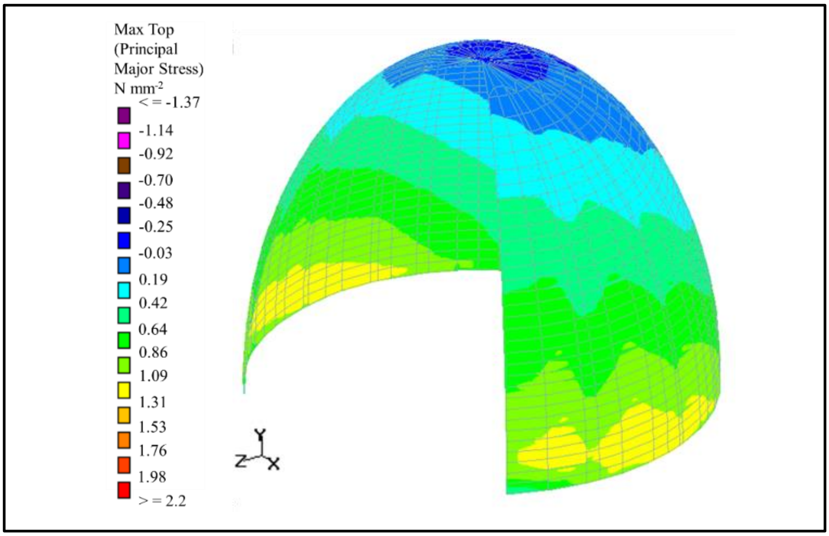

Figure 15. Principal stress contour diagram

Table 11. Dome Reinforcement Summary

Segment	Existing main reinforcement			STAAD analysis		
	Wall thickness (mm)	Diameter (mm)	Existing spacing (mm)	Required spacing (mm^2/mm)	Required spacing (mm)	Remarks
1-2	100	12	75	0.2	565	Passed
2-3	100	12	115	0.2	565	Passed
3-4	100	10	100	0.2	393	Passed
4-5	100	10	150	0.2	393	Passed

The column capacities proved to be adequate in resisting the loads imposed on the structure. However, the current dimensions and columns were found below the minimum standards (NSCP [2015] section 418.7.2) and minimum flexural capacities (NSCP [2015] section 418.7.3), respectively. The columns failing these provisions were found in the second and third levels, wherein most of these columns in the east wing and west wing are not braced by walls.

About 60% of the beams in the structure were found to be below the provisions set in chapter 4 of the NSCP (2015) for maximum spacing for shear reinforcements, and about 2% of these beams were due to load requirements. For flexure, about 25% of the beams did not satisfy the mentioned provisions. Most of these beams did not meet the requirements for minimum required flexural reinforcements for moment capacity design. Only a few beams have inadequate flexural capacity due to load requirements.

4. Conclusion

The probabilistic seismic hazard analysis for the site performed for return periods of 475, 1,000, 2,475, 4,975 and 10,000 years with 5 and 2% damping ratios resulted in response spectra that peaked at a period of 0.10 s, indicating that resonance may occur for very rigid structures. The idealized models of the evaluated structures subjected to the prescribed loads and loading combinations (i.e., gravity load, seismic load, fluid load, etc.) were analyzed for maximum principal stresses. For the storage tank and work platform, every critical member had a D/C ratio of less than 1. However, the actual thickness of the tank led to a maximum principal stress of 11.5 MPa. The NSCP provision for stability and limiting slenderness of the tank still requires an increase in thickness. The reactor pool exceeded the D/C ratio of 1 in all the sections although reinforcement bar spacings are adequate throughout the structure. In the reactor building, the resulting stresses and D/C ratio indicated that only one segment in the reactor dome exceeds the allowable tensile capacity by a 1% margin; most existing wall and dome reinforcement were found adequate. Geotechnical investigations revealed that the subsoil condition in the PRR-1 building site has a bearing capacity of 900 kPa assuming an isolated foundation system with depth and width of 1.5 at least 1 m, respectively. Lastly, results demonstrated that the PRR-1 structures are suitable for operating SATER as an HC-4 facility.

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