

Structural integrity assessment of radioactive waste storage trenches at the Philippines' centralized radioactive waste management facility

Gerardyne Anne M. Liu*, Angelo A. Panlaqui, Ray Matthew A. Bunquin, Andrew B. Valle, Ronald E. Piquero, and Kristine Marie R. Dean

Department of Science and Technology - Philippine Nuclear Research Institute, Commonwealth Avenue, Diliman, Quezon City, 1101 Philippines

ABSTRACT

This study evaluates the structural integrity of the reinforced concrete radioactive waste storage trenches (Trenches A, B, and C) at the Philippine Nuclear Research Institute (PNRI). Integrating non-destructive testing, destructive concrete coring, and ETABS dynamic modeling in accordance with the National Structural Code of the Philippines, the assessment revealed a severe, systemic degradation of the existing concrete matrix with a compressive strength of 12.77 MPa, below the 21 MPa seismic requirement. This material defect is responsible for various structural vulnerabilities, exposing a disconnect between the trenches' global geometric stiffness and their local member detailing. Structural analysis showed that Trench A failed drift checks, while Trenches B and C passed global drift checks but still exhibited localized member deficiencies. To mitigate radiological risk and align the facility with modern seismic standards, structural retrofitting is recommended, specifically utilizing reinforced concrete jacketing to restore column capacities and externally bonded Carbon Fiber Reinforced Polymers (CFRP) to strengthen roof slabs.

INTRODUCTION

Radioactive wastes are produced from medical, industrial, and research applications of radioactive materials in the Philippines (Valdezco et al. n.d.). Safe management of these wastes is necessary for the protection of the environment and public health (International Atomic Energy Agency 2006). The Philippine Nuclear Research Institute (PNRI), with the Department of Environmental and Natural Resources (DENR) as the implementing agency of the Toxic Substances, Hazardous Wastes

and Nuclear Wastes Control Act of 1990 (Republic Act 6969), is particularly responsible for the regulation, control, and management of radioactive materials. Through the institute's Radiation Protection Services Section (RPSS) of the Nuclear Services Division (NSD), PNRI currently operates and maintains the Centralized Radioactive Waste Management Facility (RWMF) for the collection, segregation, treatment and interim storage of radioactive wastes.

The development of radioactive waste management activities at PNRI began in the early 1970s, while the PNRI Radioactive Waste Management Facility (RWMF) was formally established and operationalized as the country's centralized facility for radioactive waste conditioning, immobilization, and interim storage during 1989–1992 (Lim et al. 2026). The facility includes administrative, processing, and storage buildings used for radioactive waste management activities. The original storage infrastructure included Trenches A and B, which were constructed with concrete hollow block (CHB) walls and reinforced concrete (RC) roof slabs and had a combined storage capacity of approximately 100 m³ (Millares-Garcia 1994). Although the exact construction dates of Trenches A and B could not be established from the available archival records, their documented existence by 1994 indicates that they had been in service for at least approximately 31 years at the time of the 2025 structural investigation. In 2013, the larger Trench C was added to accommodate the increasing volume and variety of radioactive waste processed at the facility. Thus, it had been in service for approximately 12 years at the time of the structural assessment. The layout of the storage building, including Trenches A, B, and C, is shown in Figure 1. The available historical and structural investigation records do not specify an original design service life for the individual storage trenches. Consequently, a numerical remaining useful life cannot be reliably determined solely from

*Corresponding author

Email Address: gamliau@pnri.dost.gov.ph

Date received: 31 May 2026

Dates revised: 27 August 2026; 14 September 2026

Date accepted: 16 September 2026

DOI: <https://doi.org/10.54645/202619SupQXE-13>

KEYWORDS

Radioactive waste management, Seismic vulnerability assessment, Concrete degradation, Environmental containment, Structural integrity, Philippine Nuclear Research Institute

chronological age. In this study, the suitability of the structures for continued service was therefore evaluated based on their observed physical condition, measured material properties, reinforcement detailing, and structural performance under current code-prescribed loading.

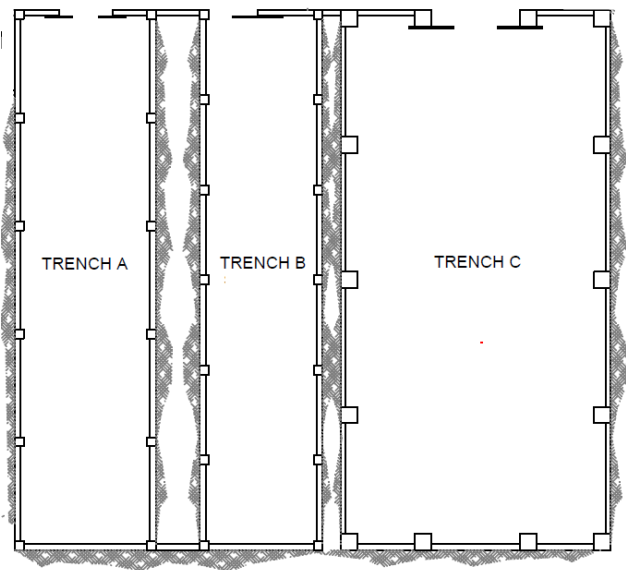


Figure 1: Plan Layout of the Storage Building

Given the extended period of service of the original storage infrastructure, deterioration and operational deficiencies have progressively become evident. Flooding has been observed inside the trenches during meteorological events such as heavy rains and typhoons. To address water intrusion, a roof spanning the entire storage building was installed during the 2010s. Subsequently, cracks and noticeable deflection were observed on the roof slab of Trench B, raising concerns regarding the structural integrity of the storage facility and its continued ability to provide adequate environmental protection for the stored radioactive waste.

The available PNRI-RWMF literature reviewed for this study has primarily addressed facility development, radioactive waste management operations, and regulatory alignment (Lim et al. 2026), while the broader concrete-engineering literature has demonstrated the complementary use of non-destructive and destructive methods for condition assessment (Ivanchev 2022; Tao and Chen 2023; Borah et al. 2026). The integration of these approaches with code-based structural modeling is particularly relevant for aging radioactive waste storage structures, where local material deterioration and member-level deficiencies may not be evident from global structural response alone. This study bridges these areas by combining visual inspection, non-destructive testing, concrete coring, and analytical structural evaluation of Trenches A, B, and C at the PNRI-RWMF.

This study evaluates the current structural integrity of the PNRI-RWMF storage trenches using site-inspection, material-testing, and structural-analysis outputs generated through the structural investigation commissioned by the Radioactive Waste Management Team and performed by ITC Corp. The present paper interprets these consultant-generated data for Trenches A, B, and C in the context of radioactive-waste storage safety and current structural requirements. The assessment is intended to provide an engineering basis for decisions on further detailed evaluation, rehabilitation, and retrofit planning. Maintaining the structural

integrity of the storage trenches is important because these structures form part of the engineered system that supports safe interim storage and environmental protection of radioactive waste packages.

MATERIALS AND METHODS

The Radioactive Waste Management Team developed the scope of work and commissioned ITC Corp. to conduct the structural investigation and evaluation of the RWMF storage building. The assessment was carried out in accordance with applicable design codes, material codes, and industry standards. These included the National Structural Code of the Philippines (NSCP) 7th Edition (2015), the American Institute of Steel Construction (AISC) Manual of Steel Construction 8th Edition, and applicable American Concrete Institute (ACI) standards covering Building Code Requirements for Structural Concrete (ACI 318-08), Manual of Standard Practice for Details and Detailing (ACI 315-99), Specifications for Structural Concrete for Buildings (ACI 301-99), and Control of Cracking in Concrete Structures (ACI 224R-01). Applicable American Society for Testing and Materials (ASTM) standards were likewise used as references for material testing and evaluation.

Site Inspection

The storage building was located within the RWMF compound at PNRI and was constructed below the natural grade line, hence its perimeter walls were subjected to lateral earth pressure from the adjacent soil. A visual inspection was carried out to identify observable deterioration, cracking, deflection, exposed reinforcement, and other signs of structural distress and to determine the structural members requiring further investigation.

Nondestructive Testing

Selected structural members were subjected to nondestructive testing using a rebar scanner and a rebound hammer.

The surface hardness and estimated compressive strength of the concrete were evaluated using a Proceq SilverSchmidt N digital rebound hammer. Prior to testing, the concrete surfaces were visually inspected and selected test locations were cleaned to remove dirt, loose particles, coatings, and other surface irregularities that could influence the measurements. Areas exhibiting cracks, spalling, honeycombing, construction joints, or other visible defects were avoided. Where applicable, reinforcement locations were identified using a rebar scanner to ensure that impact points were positioned away from embedded reinforcing steel.

Before field testing, the rebound hammer was verified in accordance with the manufacturer's recommendations using a calibration anvil to confirm proper instrument performance. The instrument settings were configured in the Hammerlink software environment, including the averaging mode, conversion curve, specimen form factor, carbonation factor, and measurement units.

A systematic test grid was established on each structural element to obtain representative surface hardness measurements. For columns, test locations were distributed at multiple elevations and on accessible faces. For beams, measurements were taken along the member length while avoiding regions directly over reinforcing bars. For slabs and wall elements, test points were arranged using a regular grid pattern. The recommended test layout adopted during the assessment is presented in Table 1.

Table 1: Recommended Rebound Hammer Test Layout (ITC Corp 2025)

STRUCTURAL ELEMENT	TEST LOCATION/LAYOUT	TYPICAL SPACING
--------------------	----------------------	-----------------

Columns	Three elevations on accessible faces	300-500 mm
Beams	Along beam length at selected intervals	300-500 mm
Slabs	Uniform grid pattern	300-500 mm

At each test location, the rebound hammer was positioned perpendicular to the concrete surface and operated in accordance with the manufacturer's instructions. A minimum of ten (10) rebound measurements were recorded for each test area. Individual impact points were spaced at least 25 mm apart and a minimum distance of 25 mm was maintained from member edges to minimize boundary effects. The instrument automatically recorded the rebound values (Q-values), from which statistical parameters including the mean value, standard deviation, and estimated compressive strength were determined through Hammerlink software.

The recorded measurements were reviewed for outliers, and only valid readings were included in the final analysis. The resulting rebound hammer data were used to evaluate the relative uniformity and condition of the concrete and to estimate surface compressive strength. The rebound hammer results were interpreted as indicative values only and were correlated with other non-destructive testing methods and visual observations to support the overall structural condition assessment.

Destructive Testing

Destructive testing was conducted to determine structural characteristics that could not be established reliably determined through visual inspection or nondestructive testing methods. Chipping was performed at selected beams and columns to confirm reinforcement diameters, member dimensions, and reinforcement detailing, as well as to determine whether the perimeter walls contained structural reinforcement. Concrete core samples were also extracted from selected structural elements using a diamond-tipped core drilling machine in accordance with ASTM C42/C42M. The extracted cores were visually examined for defects, measured for diameter and length, and assigned unique identification numbers. Their ends were then ground to provide

smooth, plane, and perpendicular bearing surfaces suitable for compression testing. After conditioning in accordance with ASTM C42/C42M requirements, the specimens were tested for compressive strength following ASTM C39/C39M procedures. During testing, a continuously applied axial compressive load was imposed until specimen failure, and the compressive strength was calculated by dividing the maximum load sustained by the specimen by its cross-sectional area.

Structural Assessment

The structural integrity of the as-built members was assessed through analytical modeling using Microsoft Excel for the evaluation of beams and slabs, ASDIP Concrete Version 4.7.5.6 for column capacity assessment, and ETABS Version 9.7 for the global analysis of the building structure. The models were assigned material properties and dimensions based on the results obtained from the non-destructive and destructive testing programs conducted as part of the structural assessment.

Design Loads

Design loads and forces were determined from dead, live, wind, and seismic loads and were evaluated using the governing load combinations prescribed by NSCP 2015. Dead loads, which included the self-weight of structural components and permanently installed equipment, were assigned based on the values presented in Table 2, while occupancy live loads were taken from Table 3. Wind loads were determined in accordance with Section 207 of NSCP 2015. Seismic loads were likewise evaluated in accordance with NSCP 2015 using the parameters presented in Table 4 for the Quezon City site. As part of the seismic hazard assessment, the Valley Fault System was identified as the nearest known seismic source, located approximately 2.0 km from the site, and classified as a Type A fault source.

Table 2: Dead Loads (ITC Corp 2025).

MATERIAL	MINIMUM DENSITY (kN/m ³)	MINIMUM DESIGN LOAD (kPa)
Concrete Members	24.0	
Steel	77.0	
Soil	18.0	
M&E Utilities		0.144
Waterproofing		0.288
Floor Finish (Floor topping, 50-mm thick)		1.53

Table 3: Occupancy Live Loads (ITC Corp 2025)

Live Load	MINIMUM DESIGN LOAD (kPa)
Roof Live Load	0.96
Floor Live Load	1.92

Table 4: Seismic Design Parameters (ITC Corp 2025; NSCP 2015).

Parameter	Value
Seismic Zone	Zone 4
Seismic Zone Factor	0.40
Soil Type	SD
Near-Source Factor, Na	1.5
Near-Source Factor, Nv	2
Seismic Coefficient, Ca	0.44Na = 0.66

Seismic Coefficient, C_v

$0.64N_v = 1.28$

Seismic Importance Factor, I

1.0

Structural System Response Factor, R

8.5

Load Combinations

The structural assessment was performed using the Ultimate Strength Method (USM), which evaluates a structure's capacity under factored load conditions representing extreme but credible loading scenarios. Under this approach, the effects of dead, live, wind, and seismic loads are amplified by specified load factors to account for uncertainties in loading and material behavior, thereby providing an adequate margin of safety against structural failure.

The resulting factored demands were compared with the calculated ultimate capacities of the structural members to determine their adequacy and level of structural safety. The governing load combinations and corresponding load factors adopted in the assessment are summarized in Table 5 and were established in accordance with the provisions of NSCP 2015.

Table 5: Load Combinations (NSCP 2015).

Load Combination	Variable	Definition
1.4D	D	Dead Load
1.2D + 1.6L + 0.5L _r	L	Occupancy Live Load
1.2D + 1.6L _r + 1.0(f _l)L	L _r	Roof Live Load
1.2D ± 1.0E _h + 1.0(f _l)L	E	Earthquake Load ($E = \rho E_h + E_v$)
1.4D + 1.0E _h + 1.0(f _l)L	E _v	0.5 Ca·I·D (Seismic Coefficient × Importance Factor × Dead Load)
1.0D - 1.0E _h + 1.0(f _l)L	f _l	1.0 for floors in places of public assembly, for live loads in excess of 4.80 kPa and for garage live load;
0.9D + 1.0E		0.5 for other live loads
1.1D + 1.0E _h		
0.7D - 1.0E _h		

RESULTS AND DISCUSSION

Visual Inspection and Physical Deficiencies

Detailed site investigations uncovered several localized and systemic physical defects across the facilities. It was observed that the concrete slab in Trench C lacks adequate concrete cover over its reinforcing steel (Figure 2). This exposes the rebars directly to environmental moisture, rapidly accelerating corrosion and inducing concrete spalling.



Figure 2: Exposed Reinforcement Bars under the Slab in Trench C.

Initial observations noted major cracking and deflection on the roof slab and ceilings of Trench B (Figure 3). The exposed slabs present major vulnerabilities due to inadequate capacity to carry additional finishing and truss loads.



Figure 3: Cracks Formed under the Slab of Trench B.

The retaining walls servicing Trenches A, B, and C are constructed using Concrete Hollow Blocks (CHB). Because these walls must resist significant lateral earth pressures, relying on CHB without proper structural reinforcement makes them highly susceptible to tilting, cracking, or other significant structural failure.

Material Properties and Concrete Strength

A critical factor in determining the structural integrity of the storage trenches was the assessment of the existing concrete compressive strength. The evaluation utilized both destructive concrete coring and non-destructive rebound hammer testing to establish the material's current capacity and uniformity.

Cylindrical concrete coring provides the most direct and reliable measure of the internal concrete matrix (Borah et al. 2026). This test yielded a critically low average compressive strength of 12.77 MPa across the structural slabs of the facilities. Specifically, the slabs in Trench A, B, and C recorded strengths of 11.02 MPa, 17.54 MPa, and 9.77 MPa (Table 6), respectively. These values fall significantly short of the 21 MPa (3000 psi) practical minimum

required by the NSCP for SMRF systems. Short core samples (L/D ratio < 2.0) were appropriately adjusted using standard mathematical correction factors to account for platen restraint

during the strength test, confirming that this 12.77 MPa average accurately represents the true state of the internal concrete.

Table 6: Compressive Strength Results from Cylindrical Test from Concrete Coring (ITC Corp 2025).

Sample Identification	Days of Moisture Conditioning	L (cm)	D (cm)	Area (sq.cm)	L/D	Fracture	KN	PSI	Actual f_c' (MPa)	CF ³	Allowable f_c' (MPa)
Trench A Slab	7 days	63.1	42	1385	1.5	SW ¹	15.9	1597.6	11.02	0.9600	21
Trench B Slab	7 days	63.7	43	1452	1.48	SW ¹	26.6	2543.4	17.54	0.9576	21
Trench C Slab	7 days	62.7	43.7	1500	1.43	CP ²	15.5	1416.8	9.77	0.9516	21

¹SW – Shear Wedge; ²CP – Cone and Plug; ³CF – Correction Factor

While the initial external investigation cited a generalized rebound hammer average of 17.5 MPa, a rigorous review of the raw rebound hammer test logs (Table 7) indicates extreme spatial variability rather than a uniform baseline strength. Rebound hammer readings fluctuated drastically across the structural framing, ranging from 7.5 MPa to 62.5 MPa, which gave an average of 38.44 MPa. This

wide standard deviation suggests that localized environmental factors, such as surface carbonation, moisture ingress, and the observed lack of protective concrete cover, may have compromised the homogeneity of the structures (Ivanchev 2022).

Table 7: Rebound Hammer Test Results (ITC Corp 2025)

Impact Counter	Mean Value (MPa)	Standard Deviation (MPa)	Allowable f_c' (MPa)	Remarks
3613	47.0	12.3	21	Pass
3603	69.0	12.8	21	Pass
3593	52.5	11.0	21	Pass
3582	20.0	5.8	21	Fail
3572	35.0	14.3	21	Pass
3562	18.0	1.8	21	Fail
3552	17.5	5.8	21	Fail
3542	45.0	6.3	21	Pass
3532	54.5	7.0	21	Pass
3522	60.0	8.3	21	Pass
Not Specified	37.0	7.0	21	Pass
3339	25.0	6.8	21	Pass
3329	22.0	13.3	21	Fail
3319	30.0	4.8	21	Fail
3309	13.5	2.3	21	Pass
3299	28.0	5.3	21	Pass
3289	20.5	4.5	21	Fail
3279	35.5	3.8	21	Pass
3267	17.0	3.0	21	Fail
3257	17.0	2.3	21	Fail
3247	22.0	5.8	21	Fail
3237	62.5	9.3	21	Pass
3227	47.5	5.3	21	Pass
3215	45.5	8.8	21	Pass
3205	38.5	2.5	21	Pass
3192	14.0	3.3	21	Fail
3182	45.5	14.0	21	Pass
3172	55.0	9.3	21	Pass
3162	55.5	10.0	21	Pass
3152	52.5	7.8	21	Pass
3142	60.5	5.0	21	Pass
3132	51.0	7.5	21	Pass
3122	54.5	10.0	21	Pass

In structural diagnostics, when discrepancies arise between NDT surface hardness and destructive core testing, the core testing dictates the absolute structural capacity (Pratama et al. 2025). The variance in the rebound hammer data serves to map the severe degradation across the facilities (Tao and Chen 2023), but the coring data of 12.77 MPa establishes the definitive baseline for mathematical modeling. Collectively, both tests confirm that the existing concrete is substandard. This material defect limits the load-carrying capacity of the primary structural frames across all trenches and underscores the necessity for extensive retrofitting interventions to ensure long-term durability and operational safety.

Structural Analysis and Global-Local Interaction

To evaluate the facilities under service, seismic, and wind loads, mathematical modeling was conducted using linear elastic structural analysis. The assessment was divided into global lateral stability (drift checks) and local member capacity (columns, beams, and slabs). The findings reveal a critical disconnect between the geometric stiffness of certain trenches and the actual structural detailing of their individual members.

Global Lateral Stability and Drift Check

Table 8 lists the modal participating mass ratios of 99.9185%, 99.6039% and 99.9992% for Trenches A, B, and C, respectively. These were generated by ETABS to determine the percentage of the building's total mass that is mobilized during specific vibrational events such as seismic activities. NSCP 2015 requires that the sum of the ratios reach a minimum of 90%, which proves that the model successfully captured most of the building's dynamic behavior, where the subsequent drift checks and fundamental periods are structurally valid.

Table 8: Modal Mass Participating Ratios in Percent (ITC Corp 2025).

	Trench A	Trench B	Trench C
Mode 1	99.8778	98.8117	99.9988
Mode 2	99.8778	100.0000	99.9988
Mode 3	100.0000	100.0000	100.0000
Average	99.9185	99.6039	99.9992

A structure's fundamental period is a primary indicator of its lateral stiffness and overall vulnerability to dynamic horizontal forces, such as seismic activity (ASCE 7-16 2016). In this assessment, the lateral stability of each trench was evaluated against maximum allowable story drift limit, Δ_M , per Section 208.6.5 of the NSCP 2015.

The structural model for Trench A revealed a critical state of instability, recording a high fundamental period of 0.881095 seconds (Table 9). This value denotes an under-stiffened, highly flexible structural frame that fails standard drift requirements. Under seismic loading, this lack of rigidity would induce severe lateral displacement, rendering the structure susceptible to significant P-Delta effects and global collapse. On the other hand, the models for Trenches B and C demonstrated adequate global stiffness, successfully satisfying lateral drift limits with fundamental periods of 0.766798 seconds and 0.385397 seconds, respectively. Trench C displays significant lateral rigidity.

However, while this geometric stiffness effectively prevents global overturning, fundamental structural dynamics dictate that stiffer structures inherently attract higher seismic base shear forces during an earthquake (Paredes et al. 2025). This phenomenon directly contextualizes the localized member failures observed in these facilities. While the global frame successfully resists lateral swaying, the internal forces generated by this rigid resistance exceed the capacities of the inadequately reinforced individual members.

Table 9: Drift Checks for Trench A, B, and C (ITC Corp 2025).

	Story Height (m)	Period Used (s)	Drift Limit (m)	Maximum Calculated Δ_M (m)	Remarks
Trench A	2.800	0.881095	0.0700	0.08045	Fail
Trench B	2.800	0.766798	0.0700	0.04929	Pass
Trench C	4.375	0.385397	0.0875	0.01219	Pass

Review of Primary Vertical Members

Columns serve as the critical load path transferring accumulated gravity and lateral loads to the foundation. Each trench exhibits different global stiffness characteristics, therefore, the mechanisms by which columns fail under seismic or ultimate loading conditions vary significantly. Furthermore, the concrete strength of 12.77 MPa reduces the threshold for all failure modes.

Trench A is characterized by a critical lack of lateral stiffness, as already proved by its global drift check (Table 9). Because the building is highly flexible, it experiences massive horizontal swaying during a seismic event. The gravity loads acting on the deflected columns induce severe secondary moments when the structure sways excessively. Given the low concrete strength and the lack of structural rigidity, the columns in Trench A are highly susceptible to global buckling.

Trench B, on the other hand, successfully passed its global drift checks indicating that its structural frame is stiff, with a 0.86 moment capacity ratio (Table 10). However, because the structure is adequately stiff and does not sway excessively, the columns must absorb the full vertical gravity load without the relief of structural yielding. The degraded concrete capacity makes these columns highly susceptible to pure axial crushing.

Trench C is the stiffest facility with the lowest period of 0.39 seconds but recorded widespread column moment capacity failures. With the already low concrete capacity of 12.77 MPa, the concrete column lacks the capacity to resist seismic forces making it susceptible to shear failures. The column also exceeds the axial-moment interaction limits (Figure 4C) wherein there is no capacity to resist combined stresses leading to simultaneous crushing and flexural yielding.

Table 10: Flexural Ratios of Columns in Trenches A, B, and C (ITC Corp 2025).

	Mu (KN-m)	ϕM_n (KN-m)	Mu/ ϕM_n	Pass/Fail
Trench A	244	39.4	6.2000	Fail
Trench B	37	42.8	0.8600	Pass
Trench C	244	187.9	1.3000	Fail

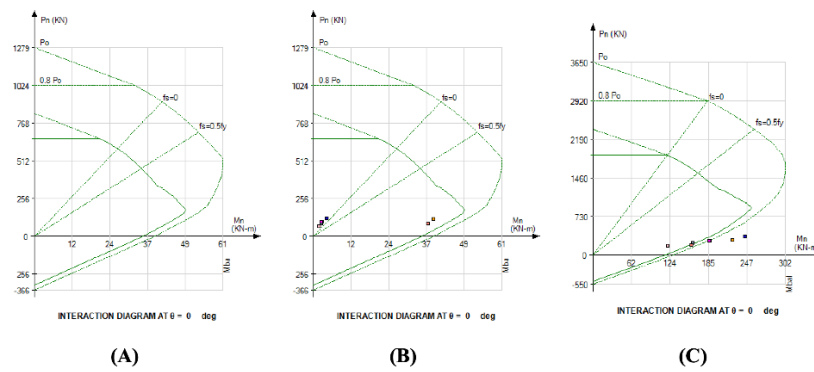


Figure 4: Column Interaction Diagrams for (A) Trench A, (B) Trench B, and (C) Trench C (ITC Corp 2025).

Review of Primary Horizontal Members: Beam

The beams, responsible for gathering roof loads and resisting flexure, exhibited varying degrees of failure across the trenches. Trench A demonstrated widespread capacity and deflection failures in the consultant analysis, with the affected beam members identified in Figure 5. Trench B beams passed the reported capacity and deflection checks. Trench C beams generally passed ultimate capacity checks but did not satisfy the reported deflection limits.

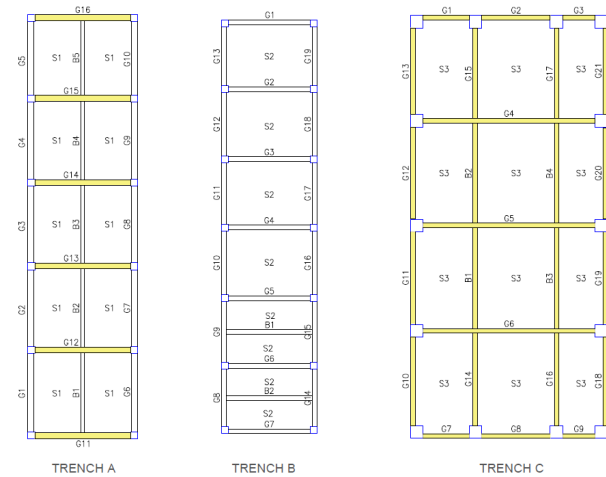


Figure 5: Failed Beam Members (Highlighted in Yellow).

Review of Primary Horizontal Members: Slabs

The roof slabs of the facility act as the environmental-protective envelope for radioactive waste. It is also responsible for transferring lateral forces to the beams and columns with its horizontal diaphragms.

The slabs in Trench A have adequate reinforcement, making them safe under the applied loads. In contrast, although Trenches B and C exhibited stable global framing, their slabs failed due to insufficient reinforcement. For Trench B, the measured rebar spacing was 240 mm, exceeding the maximum allowable spacing of 225 mm specified in Table 11. Similarly, Trench C had a measured spacing of 230 mm, which also surpassed the allowable limit of 225 mm.

The slabs designed with inadequate reinforcement lack the necessary tensile capacity to distribute concentrated loads. The wide rebar spacing increases the risk of cracking, deflection, and localized punching shear failures. This inherently compromises the operational safety of the facility.

Table 11: Summary of Failed Slabs and Rebar Spacing of Failed Slabs (ITC Corp 2025)

	Slab	Allowable Deflection (mm)	Actual Deflection (mm)	Deflection Check	Rebar Spacing (mm) of failed slabs	
					Actual	Maximum
Trench A	S1	9.1	2.3	Safe		
	S2	9.4	2.6	Safe		
	S3	9.4	2.5	Safe		
	S4	9.7	3.0	Safe		
Trench B	S1	7.0	7.2	Fail	240	225
	S2	7.0	7.2	Fail	240	225
	S3	10.3	7.2	Safe		
	S4	10.3	7.2	Safe		
Trench C	S1	11.7	11.0	Safe		
	S2	11.7	13.8	Fail	230	225
	S3	11.7	11.0	Safe		
	S4	11.9	14.8	Fail	230	225
	S5	11.9	14.8	Fail	230	225
	S6	15.0	12.0	Safe		

The main evaluation reveals that the RWMF storage trenches suffer from a systemic lack of structural cohesion, driven by a combination of substandard concrete strength and deficient structural detailing, such as inadequate concrete cover, improper rebar spacing, and insufficient member sizing. This underscores a critical engineering principle: passing a global seismic drift check does not guarantee structural safety if local member detailing is inadequate (ASCE 2017). Trench A is the most critical, exhibiting both global instability and localized member failures, which requires a holistic redesign incorporating shear walls or improved framing. Meanwhile, although Trenches B and C retain enough global lateral stiffness to pass drift limitations, the localized failure of their columns and roof slabs poses significant safety hazards. To enhance the facility's engineering design and complement internal member capacities with global stiffness, substantial targeted retrofitting is required. These interventions include reinforced concrete jacketing to address the substandard compressive strength of the columns, the application of carbon fiber reinforced polymer (CFRP) to strengthen deficient slabs (ACI Committee 440 2017), and proper reinforcement detailing for the CHB retaining walls.

This assessment was based on the available site inspection results, material testing data, and structural models prepared for the existing storage building. Therefore, detailed retrofitting design, construction sequencing, and cost evaluation were beyond the scope of this paper.

CONCLUSION

The comprehensive structural evaluation of the RWMF storage building revealed a systemic compromise in structural integrity, fundamentally driven by severe material degradation. Empirical data from destructive core testing established a critically low in-situ concrete compressive strength of 12.77 MPa, falling drastically short of the 21 MPa minimum baseline required by the NSCP 2015 for SMRF systems. This severe material deficiency has been the primary means for the widespread vulnerabilities observed across the trenches, reducing the flexural, shear, and axial capacities of the reinforced concrete members to unsafe levels.

Furthermore, the dynamic structural analysis demonstrates a disconnect between the global geometric stiffness of the trenches and their local member detailing. Because of variations in structural geometry, this uniform material weakness manifests in distinct failure mechanisms under ultimate loading conditions. Trench A was found to suffer from lateral instability, as its abnormally high fundamental period indicates an under-stiffened frame that fails to meet drift limits, leaving the structure highly susceptible to global buckling, drift-induced shear failures, and significant P-Delta effects. Trench B exhibited adequate global stiffness but was susceptible to flexural failures caused by an under-reinforced roof slab, which transfers excessive loads to the supporting beams and increases the potential for localized structural damage. Trench C demonstrated good stiffness and resistance to lateral movement. However, this rigidity attracts higher seismic forces that can overload the existing columns, leading to column failures and non-compliance with the strong column-weak beam principle required for seismic-resistant structures.

Therefore, the integration of material testing and mathematical modeling confirms that the existing storage trenches are structurally inadequate and incapable of safely sustaining their design service and seismic loads.

RECOMMENDATIONS

Based on the findings of this study, two approaches may be considered to address the structural deficiencies of the RWMF storage building: demolition and reconstruction of the existing storage trenches, or the implementation of a comprehensive structural retrofitting program. While demolition and reconstruction would provide a completely new structure designed in accordance with current code requirements, this option would involve significantly higher costs, longer implementation periods, disruption to facility operations, and additional waste management considerations.

As a more practical and cost-effective alternative, structural retrofitting is recommended to restore the strength, stiffness, and seismic performance of the existing trenches while maximizing the use of the remaining serviceable components of the structure. The assessment indicates that, despite the degraded concrete strength and inadequate member capacities, the existing structural system can still be rehabilitated through appropriate strengthening measures. These may include reinforced concrete jacketing of columns to improve axial, flexural, and shear capacities, enhancement of critical structural members to address identified demand-capacity deficiencies, and strengthening of the roof diaphragm using Carbon Fiber Reinforced Polymer (CFRP) systems or other suitable technologies to improve load transfer and seismic performance.

The final retrofit scheme should be developed through detailed design and analysis to ensure compliance with current NSCP requirements and achieve the required level of structural safety and performance.

ACKNOWLEDGMENTS

The authors acknowledge the support funding from the DOST Grants-in-Aid Program (Project No. 9006) and technical monitoring of DOST – Philippine Council for Industry, Energy, Emerging Technology Research and Development (DOST-PCIEERD). The authors also acknowledge ITC Corp for collaboration and technical support in conducting the structural investigation, material testing, and structural analysis of the RWMF storage building.

CONFLICT OF INTEREST

The authors declare that there is no conflict of interest.

CONTRIBUTIONS OF INDIVIDUAL AUTHORS

Gerardyne Anne M. Liu: Conceptualization, Methodology, Formal Analysis, Investigation, Writing of Original Draft, Visualization. **Angelo A. Panlaqui:** Validation, Investigation, Review & Editing. **Ray Matthew A. Bunquin:** Validation, Investigation. **Andrew B. Valle:** Validation, Investigation. **Ronald E. Piquero:** Validation, Investigation, Resources, Supervision, Project Administration, Funding Acquisition. **Kristine Marie R. Dean:** Resources, Project Administration

REFERENCES

ACI Committee 440. Guide for the design and construction of externally bonded FRP systems for strengthening concrete

- structures (ACI 440.2R-17). Farmington Hills: American Concrete Institute, 2017.
- American Concrete Institute. Building code requirements for structural concrete (ACI 318-08). Farmington Hills: American Concrete Institute, 2008.
- American Concrete Institute. Control of cracking in concrete structures (ACI 224R-01). Farmington Hills: American Concrete Institute, 2001.
- American Concrete Institute. Manual of standard practice for details and detailing (ACI 315-99). Farmington Hills: American Concrete Institute, 1999.
- American Concrete Institute. Specification for structural concrete for buildings (ACI 301-99). Farmington Hills: American Concrete Institute, 1999.
- American Institute of Steel Construction. Manual of steel construction. 8th ed. Chicago: American Institute of Steel Construction, 1980.
- American Society of Civil Engineers. Minimum design loads and associated criteria for buildings and other structures (ASCE 7-16). Reston: American Society of Civil Engineers, 2016.
- American Society of Civil Engineers. Seismic Evaluation and Retrofit of Existing Buildings (ASCE/SEI 41-17). Reston: American Society of Civil Engineers, 2017.
- American Society for Testing and Materials. ASTM standards. West Conshohocken: American Society for Testing and Materials, [n.d.].
- ASDIP Structural Software. ASDIP concrete: software for design of reinforced concrete columns [Computer software]. [n.d.].
- Association of Structural Engineers of the Philippines. National structural code of the Philippines: buildings, towers and other vertical structures, volume 1. 7th ed. Quezon City: Association of Structural Engineers of the Philippines, 2015.
- Borah N, Raj L, and Raj JA. Experimental study on comparison of equivalent concrete compressive strength of cores and rebound hammer tests with in-place and standard cubes. *Indian Journal of Engineering & Materials Science* 2026; 33(1):20-35. 10.56042/ijems.v33i01.21567
- Computers and Structures, Inc. ETABS: integrated building analysis and design software [Computer software]. Walnut Creek: Computers and Structures, Inc., [n.d.].
- International Atomic Energy Agency. Storage of Radioactive Waste. IAEA Safety Standard Series (WS-G-6.1). Vienna: International Atomic Energy Agency, 2006.
- ITC Corp. Design Analysis of the RWMF Storage Building. Quezon City: ITC Corp, [2025].
- ITC Corp. Structural Investigation Report of the RWMF Storage Building. Quezon City: ITC Corp, [2025].
- Ivanchev I. Investigation with Non-Destructive and Destructive Methods. *Applied Sciences* 2022; 12(23):12172. 10.3390/app122312172
- Lim JW, Panlaqui AA, Bunquin RMA, Piquero RE. Technical Evolution of the Philippine Radioactive Waste Management Facility: Infrastructure Development, Regulatory Alignment, and Inventory Overview. *Nuclear Engineering and Technology*. 2026;58(11):104500. 10.1016/j.net.2026.104500
- Millares-Garcia C. Management of Liquid Radioactive Wastes at PNRI. Quezon City: Philippine Nuclear Research Institute, 1994.
- Paredes J, Ramirez W, Pico F, Acosta R, Toapanta OG, and Mayacela M. Impact of Structural Stiffness on Vibration Periods of Concrete Buildings: A Systematic Review. *Materials* 2025; 18(11):2612. 10.3390/ma18112612
- Pratama VP, Setioningsih R, and Marwanto M. Comparative Analysis of Hammer Test and Compression Testing Machine Against Normal Concrete Compressive Strength Test K300. *Jurnal Teknik Sipil* 2025; 6(1). 10.51988/jtsc.v6i1.269
- Republic of the Philippines. Atomic energy regulatory and liability act of 1968. Republic Act No. 5207. Manila: Republic of the Philippines, 1968.
- Republic of the Philippines. Science act of 1958. Republic Act No. 2067. Manila: Republic of the Philippines, 1958.
- Republic of the Philippines. Toxic substances and hazardous and nuclear wastes control act of 1990. Republic Act No. 6969. Manila: Republic of the Philippines, 1990.
- Tao J, and Chen J. Experimental Study on the Spatial Variability of Concrete by the Core Test and Rebound Hammer Test. *ASCE-ASME Journal of Risk and Uncertainty in Engineering Systems* 2023; 9(4). 10.1061/AJRUA6.RUENG-1101
- Valdezco EM, Palattao MVB, Nohay CM, Reyes RY, and Venida LL. Philippine Initiatives for the Safe Management and Disposal of Radioactive Waste. Quezon City: Philippine Nuclear Research Institute, [n.d.].