

Article

Stiffness Analysis at Failure State of Reinforced Concrete and Prestressed Concrete Tubular Members Under Internal Blast Loading

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Abstract

Large civil infrastructure, such as nuclear power plant containment vessels, is predominantly constructed using prestressed concrete (PSC) or reinforced concrete (RC). Previous experimental studies investigated the internal blast responses of reduced-scale open-ended reinforced concrete containment vessel (RCCV) and prestressed concrete containment vessel (PCCV), providing insight into displacement-based structural behavior. However, these studies were limited by the inability to directly measure internally reflected wall pressures and by the lack of experimental data for enclosed boundary conditions. In this study, a displacement-calibrated LS-DYNA simulation framework is developed to extend prior experimental findings to both open-ended and enclosed RCCV and PCCV configurations. An internal detonation of ammonium nitrate–fuel oil (ANFO) is simulated at the center of a cylindrical vessel. The simulation models are calibrated using reduced-scale open-ended experimental displacement time histories. Simulation results are post-processed to construct force–displacement relationships based on discrete load–displacement points across charge levels and their bilinear regression. Using the resulting stiffness indices and a stiffness-based scaling procedure, failure-inducing internal blast loads are estimated for real-scale vessels under conditions where direct internal pressure measurement is not feasible. The proposed framework enables response-based assessment of semi-confined internal explosions and supports model-informed safety evaluation of containment-type structures.

Keywords: internal blast; blast load; elastic stiffness; plastic stiffness; ANFO

1. Introduction

Prestressed concrete containment vessels (PCCVs) and reinforced concrete containment vessels (RCCVs) constitute the primary pressure-retaining barrier in nuclear reactor containment structures and other safety-critical infrastructures [1], and, to secure constructability, durability, and global structural stiffness, these large vessels are commonly realized as prestressed-concrete (PSC) or reinforced-concrete (RC) tubular/cylindrical systems whose long-term integrity has been extensively addressed through durability-focused assessments and full-scale or scaled experimental programs, including pressure and shaking-table tests for RCCVs, post-test analytical evaluations for scaled PCCVs, and in-service monitoring during integrated leakage rate testing [2–7]. Nevertheless, despite their robustness under conventional design actions, concrete containment structures may exhibit vulnerabilities under short-duration extreme events such as blast or impact because tensile cracking and spalling can localize damage and thereby trigger an abrupt



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deterioration of load-carrying capacity, while PSC systems, although generally providing higher initial stiffness and confinement than RC systems, may still suffer a rapid loss of resistance when instantaneous overpressure compromises confinement effectiveness or prestress continuity [8]. In addition, localized deformation and strain concentration in the steel liner around penetrations have been reported, indicating that global displacement alone may not fully explain the initiation of local failure mechanisms [9]. Complementary studies on ultimate internal pressure capacity and aging/degradation effects of containment structures provide a broader context for interpreting such local mechanisms within the overall pressure-retaining limit state [10,11]. Among such extreme scenarios, internal explosions are of particular concern because the pressure–time environment is strongly governed by boundary openness and compartment effects [12–15], and, although numerous experimental and numerical studies have investigated internal blast responses, a systematic framework remains limited in which a numerical model validated under an open-ended condition is consistently extended—while preserving specimen-consistent material properties and modeling settings to fully enclosed conditions that are difficult to realize experimentally. Accordingly, this study cross-validates a finite-element (FE) model against published open-ended internal blast test data using specimen-consistent material properties, and then applies the validated model to enclosed configurations to evaluate pressure–displacement response and to estimate failure-inducing load levels through stiffness-based analysis.

Literature Review

Moreover, when ANFO or ammonium-nitrate-based explosives are considered, the basis and uncertainty of ANFO-to-TNT equivalency should be clarified because blast parameters may vary with charge characteristics and with the equivalency metric selected, as indicated by recent parameter-characterization studies and ANFO-based experimental programs [16–18]. To overcome the practical limitations of internal instrumentation survival and to obtain reliable internal pressure–response information, recent studies have increasingly combined experiments and simulations for internal explosions in earth-covered magazines, reinforced concrete framed systems, reinforced concrete shear walls, and large-scale multi-box structures, collectively highlighting the strong influence of boundary conditions and compartment effects on damage development and structural response [19–23], while complementary experimental frameworks have been reported for partially confined or internal explosion conditions in reinforced concrete structures and protective concrete panels, and simplified approaches have been proposed for pressure loading and transmission to adjacent compartments [24–26]. For tubular reinforced concrete and prestressed concrete members subjected to internal ANFO detonation, experimental databases have reported increasing mid-span displacement and characteristic cracking patterns with charge weight for reinforced concrete specimens and reduced displacement due to enhanced confinement and initial stiffness for prestressed concrete specimens with comparable geometry and reinforcement [27,28]; nevertheless, direct reinforced concrete–prestressed concrete comparisons under identical geometry, materials, and loading, as well as validated numerical workflows capable of reproducing internal blast pressure histories, particularly when extending from open-ended validation to enclosed scenarios, remain limited in the open literature [12–15,27,28].

2. Details of Internal Blast Test [27,28]

This study utilizes internal blast test data reported in prior studies conducted under partially open-ended configurations [27,28]. This section summarizes only the information directly required for numerical model input and validation, including specimen geome-

try, loading conditions, and measurement items, while detailed experimental procedures, instrumentation setups, raw data, and additional results are referred to the original publications. This approach is adopted to clearly emphasize that the primary contribution of the present study lies not in reproducing experiments, but in extending a literature-validated numerical model, under identical modeling assumptions, to enclosed configurations in order to compare and evaluate internal blast responses.

RCCV & PCCV Test Specimen Details

The specimens reported in the prior studies consisted of 1/20-scale cylindrical RCCV and PCCV models based on a Korean nuclear containment structure. As shown in Figure 1, the representative dimensions of the specimens include an outer diameter of 2700 mm, an inner diameter of 2000 mm, a wall thickness of 350 mm, and an overall height of 3600 mm. The geometries of the RCCV and PCCV specimens are illustrated in Figure 2, and the material properties of the concrete, reinforcing bars, and prestressing tendons are summarized in Table 1. As shown in Figure 3a, the prior tests reproduced internal blast loading by placing an ANFO charge at the center of the mid-span cross-section of the cylindrical structure, with both ends of the specimen left open so that the blast pressure could be released through the open ends, forming an open-ended condition. Because direct measurement of pressure on the internal surface of the structure is not feasible, blast pressures were measured using free-field pressure meters located 7000 mm away from each open end of the specimen, as shown in Figure 3b. Structural displacements were measured using linear variable displacement transducers (LVDTs) installed at three locations, namely at the mid-span (0°), the mid-span (90°), and a position 1000 mm away from the mid-span, as illustrated in Figure 4. The experimental data reported in the prior studies are summarized in Table 2. The test cases and ANFO charge weights adopted in the present study follow exactly those reported in the literature. For the RCCV tests, ANFO charge weights of 15.88, 20.41, 22.68, and 24.95 kg were reported for RC35, RC45, RC50, and RC55, respectively. For the PCCV tests, ANFO charge weights of 22.68, 24.95, 27.22, 29.48, and 31.75 kg were reported for PSC50, PSC55, PSC60, PSC65, and PSC70, respectively. The open-ended validation in the present study focuses particularly on the RC50/RC55 and PSC50/PSC55 cases, for which the observed response quantities reported in the literature are extracted and compared using identical definitions.

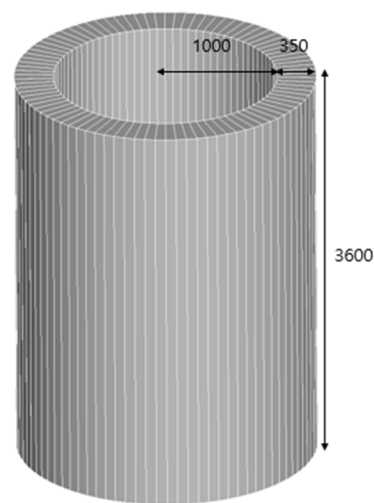


Figure 1. A 1/20-scale model of the Kori Nuclear Power Plant units of Korea (unit: mm) [26,27].

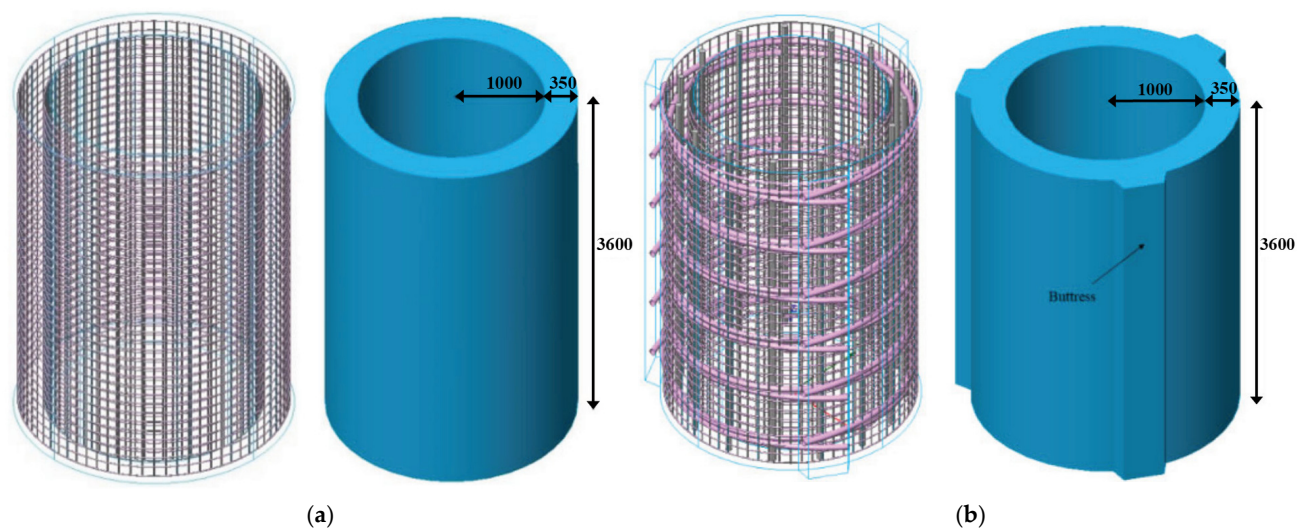


Figure 2. Specimen details (unit: mm). (a) RC specimen; (b) PSC specimen [26,27].

Table 1. Mechanical properties of the concrete, rebar and tendon used in the tests [26,27].

Concrete	Compressive strength (MPa)	40.00
	Elastic modulus (GPa)	30.46
	Poisson's ratio	0.17
Rebar	Yield strength (MPa)	413.68
	Tensile strength (MPa)	620.53
	Elongation (%)	7.00
Tendon	Yield strength (MPa)	1600
	Ultimate strength (MPa)	1730
	Unit weight (kg/m)	1.101

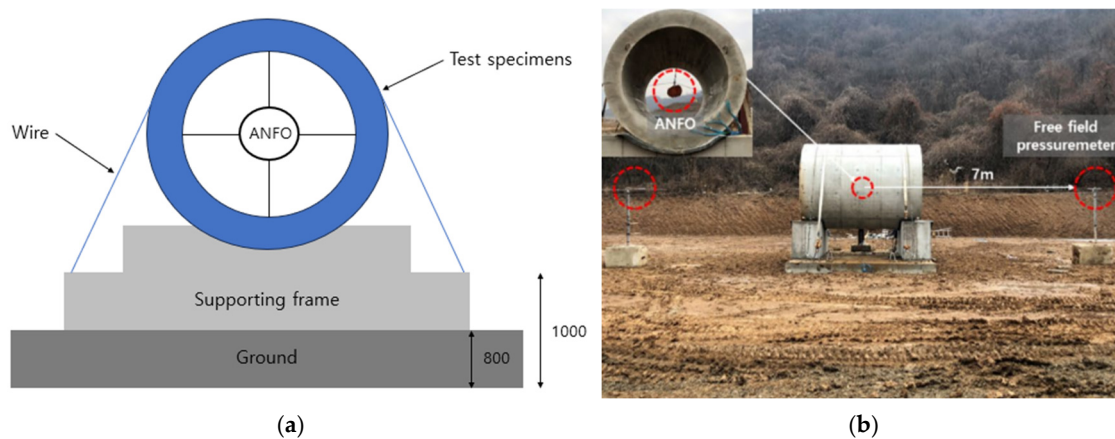


Figure 3. Internal blast test setup (unit: mm). (a) Details of specimen experimental setup; (b) Free field pressuremeter locations [26,27].

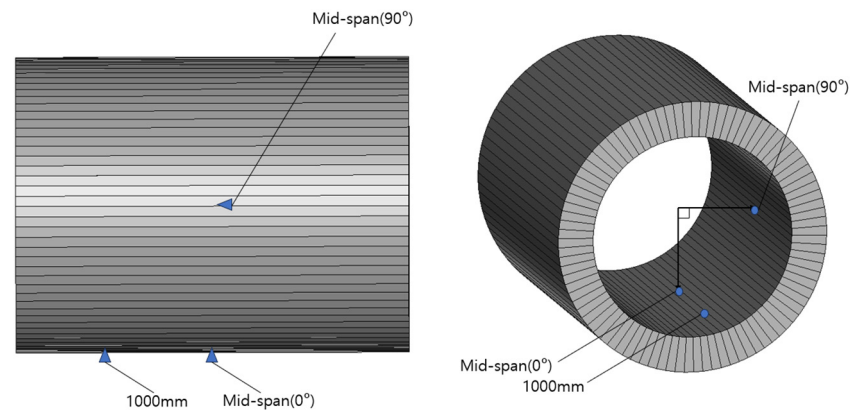


Figure 4. Measurement locations for displacement and internal blast pressure [26,27].

Table 2. Summary of the internal blast test data [26,27].

Value		RC35	RC45	RC50	RC55	PSC50	PSC55	PSC60	PSC65	PSC70
Peak pressure (MPa)		0.172	0.297	0.317	0.339	0.289	0.344	0.375	0.386	0.404
Duration (msec)		5.981	5.856	5.826	5.811	6.062	6.156	6.482	6.566	6.638
Impulse (MPa-msec)		0.360	0.379	0.387	0.444	0.528	0.534	0.554	0.569	0.570
Maximum displacement (mm)	Mid-span (0°)	6.57	14.67	15.27	16.25	6.62	9.56	10.28	11.49	12.01
	Mid-span (90°)	3.95	7.39	8.76	11.29	3.80	4.49	8.49	4.60	9.63
	1000 mm (0°)	5.58	8.13	8.37	8.64	4.88	5.49	4.61	8.47	9.63
Environmental condition	Temperature (°C)	9.2	6.3	2.9	−6.0	−5.1	−5.1	8.1	−4.3	−3.7
	Humidity (%)	45	41	16	31	34	34	33	30	35

3. Internal Blast Loading Analysis Using the Simulation Tool

The structural response of reinforced and prestressed concrete vessels subjected to internal blast loading is a highly transient dynamic phenomenon, in which not only the peak response but also the temporal evolution of displacement provides essential information for evaluating structural behavior. Accordingly, the numerical model developed in this study was not validated solely based on peak response values. Instead, model reliability is assessed through a systematic comparison of displacement–time histories obtained from numerical simulations and corresponding experimental measurements.

As shown in Figure 5, the numerical simulations successfully reproduce the principal response characteristics observed in the experiments, including the initial inertia-dominated response immediately after detonation, the time required to reach peak displacement, the subsequent oscillatory decay, and the development of residual displacement. Because the experimental pressure data are measured under free-field conditions rather than directly on the internal structural surface, direct quantitative comparison of pressure–time histories is inherently limited. Under these constraints, displacement–time histories are identified as the most physically consistent and reliable validation metric. Based on this comparison, the numerical model is confirmed to reasonably capture the overall dynamic response characteristics of the specimens subjected to internal blast loading.

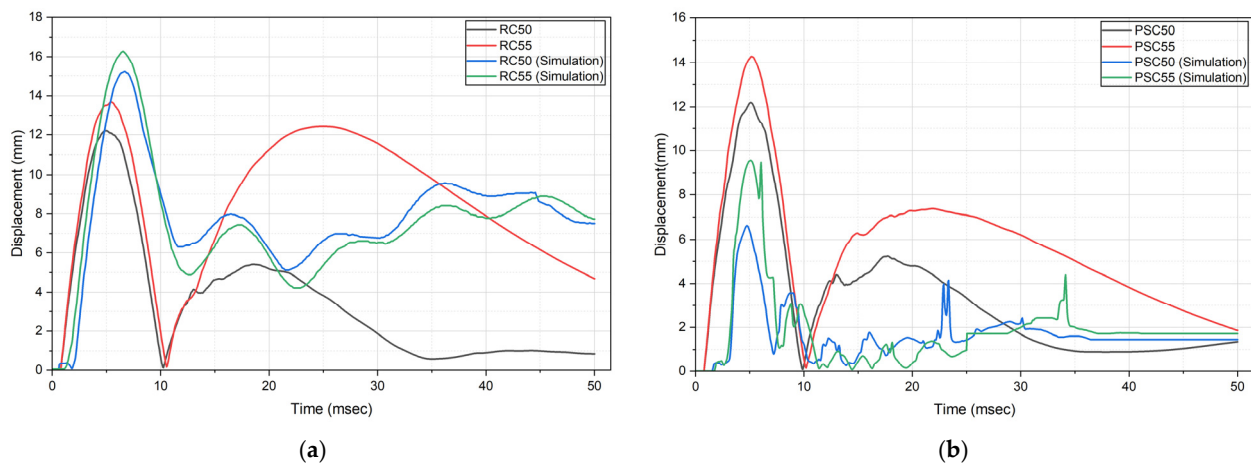


Figure 5. Comparison of experimental and numerical displacement time histories under internal blast loading. (a) RCCV specimens; (b) PCCV specimens [27,28].

3.1. Materials Modeling

The reinforced concrete and prestressed reinforced concrete vessels are modeled in LS-DYNA to evaluate reflected-pressure and displacement responses under internal blast loading. Two numerical configurations are considered. The first configuration represents the partially open-ended structure corresponding to the experimental specimen, and the second configuration represents an enclosed structure in which blast pressure is prevented from escaping through the ends. The configurations are presented in Figure 6.

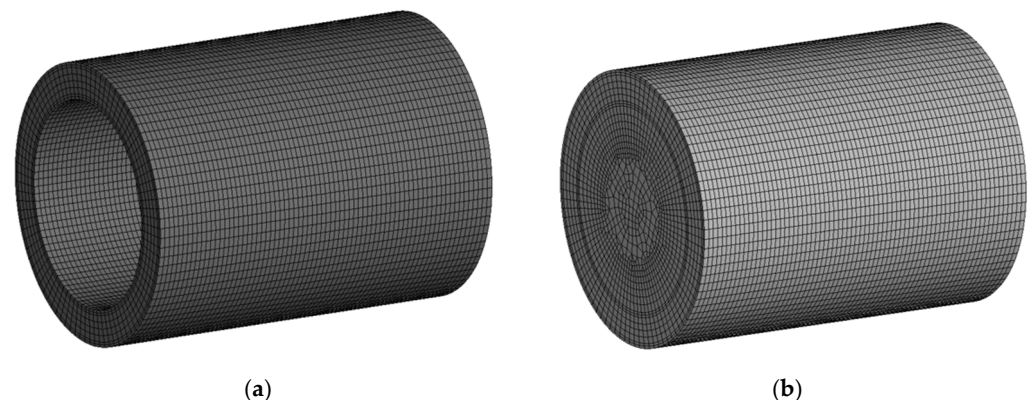


Figure 6. Modeled concrete structure configuration. (a) Open-ended structure modeling; (b) Enclosed structure modeling.

For both configurations, the wall thickness is set to 350 mm, and the material properties are defined to be consistent with those of the tested specimens. The RCCV and PCCV models are constructed using a design compressive strength of 40 MPa for concrete, D13 reinforcing bars, and 15.2 mm prestressing tendons. The overall geometry and reinforcement layout are modeled to match the experimental specimens, as shown in Figure 7.

Concrete behavior is modeled using *MAT_CONCRETE_DAMAGE_REL3 to represent nonlinear response under blast loading, including stiffness degradation and damage accumulation under compression and tension. The strain-rate effect of concrete is incorporated through the LCRATE curve so that strength enhancement varies with the effective strain rate. Concrete failure is represented using *MAT_ADD_EROSION, and the erosion criteria are defined using a maximum principal stress threshold of 41.2 MPa and an equivalent strain limit of 0.40. These criteria are applied consistently to all concrete elements to prevent excessive element distortion during progressive damage development while allowing localized crack-

ing and damage concentration prior to element deletion. Reinforcing bars and prestressing tendons are modeled using *MAT_PIECEWISE_LINEAR_PLASTICITY to represent the elastic–plastic response of steel. The stress–strain relationships are defined according to the material specifications used in the experimental program. Strain-rate dependence of steel is not considered by setting the Cowper–Symonds parameters to zero, while transient dynamic characteristics are captured through explicit time integration, inertia effects, and stress-wave propagation. Bonded interaction between the solid concrete elements and embedded beam elements is modeled using *CONSTRAINED_LAGRANGE_IN_SOLID. Penalty coupling is adopted to provide stable force transfer between reinforcement and surrounding concrete without introducing mesh distortion under high-rate loading.

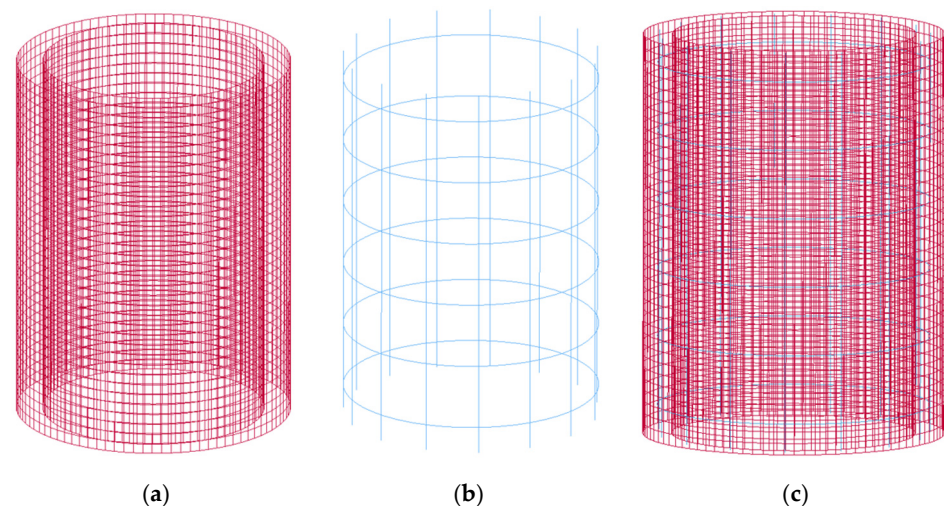


Figure 7. Modeled rebar and tendon configuration. (a) Rebar (RCCV); (b) Tendon (PCCV); (c) Rebar and tendon (PCCV).

Prestressing of the horizontal tendons is implemented using *INITIAL_AXIAL_FORCE_BEAM, and an initial prestressing force of 280 kN is uniformly applied to each tendon. In the experiments, the measured prestress forces vary in the range of 220–380 kN due to anchorage slip and time-dependent relaxation effects. In the numerical simulation framework, the prestressing force is therefore applied as a uniform initial value to all tendons to establish a consistent baseline condition for comparative analysis between RCCV and PCCV specimens. To evaluate the influence of experimentally observed prestress variability, an additional sensitivity analysis is performed by varying the initial prestressing force within the measured range. Specifically, prestressing force levels of 220 kN, 280 kN, and 380 kN are applied to a representative enclosed specimen, and the corresponding displacement–time and reflected pressure–time responses are compared, as shown in Figure 8. The results indicate that, while minor differences in the absolute displacement magnitude are observed, the overall temporal response characteristics and dominant structural behavior under internal blast loading remain similar across the considered prestressing force levels. Based on these observations, the use of a uniform prestressing force of 280 kN is considered reasonable for capturing the global structural response and for evaluating displacement-based internal blast behavior under controlled conditions. The simulation results are therefore interpreted in a comparative sense, focusing on relative response trends rather than exact replication of specimen-specific prestress states. The three circumferential tendons spaced at 120° intervals are idealized as circular hoop rings, and their prestressing effect is represented by an inward compressive pressure applied to the outer concrete surface, with a magnitude corresponding to the equivalent prestressing action. This idealization preserves the global confinement effect of circumferential prestressing while maintaining numerical efficiency.

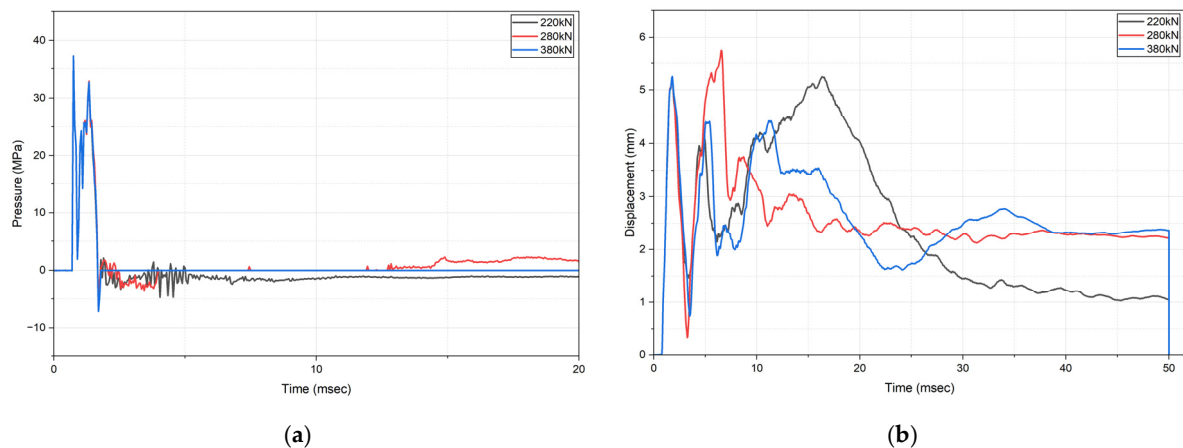


Figure 8. Comparison of internal blast responses for different prestressing forces. (a) Reflected pressure-time histories; (b) Displacement-time histories.

3.2. Numerical Settings and Stability Control

The numerical model is solved using explicit time integration to capture the highly transient response of internal blast loading. The concrete domain is discretized using three-dimensional solid elements, and reinforcing bars and prestressing tendons are modeled using beam elements. The nominal mesh size in the blast-loaded wall region is set to 50 mm, selected to balance stress-wave resolution, local deformation capture, and computational cost. This mesh resolution is consistent with those adopted in previous blast simulation studies and mesh-resolution investigations for reinforced concrete structures under internal or confined blast conditions [29–31]. The same mesh density is applied consistently to all simulation cases to ensure comparability of results. The time step is automatically controlled by the explicit stability condition based on the Courant criterion. Artificial mass scaling is not introduced in order to preserve inertial response and stress-wave propagation characteristics. Solution stability is monitored by tracking the evolution of kinetic energy and internal energy throughout the analysis to confirm physically consistent energy exchange during the blast response. Hourglass control is applied to suppress nonphysical zero-energy deformation modes associated with reduced-integration solid elements. A stiffness-based Flanagan–Belytschko formulation is employed with an hourglass coefficient $QM = 0.05$. This value is selected to remain well below the instability threshold recommended in the LS-DYNA manual ($QM \approx 0.15$), while providing sufficient suppression of hourglass modes without introducing excessive artificial stiffness, which may occur in stiffness-based formulations under large deformation conditions. Throughout all simulations, the hourglass energy remained within stable and acceptable levels, confirming the appropriateness of the selected coefficient. A formal mesh convergence study and extensive parameter sensitivity analysis were not performed due to the high computational cost of repeated internal blast simulations. Instead, the adequacy of the selected mesh density and numerical control parameters is assessed through calibration against experimentally measured displacement time histories. The agreement in the temporal evolution of displacement responses supports the suitability of the adopted numerical settings for capturing the dominant global structural behavior within the scope of the present study.

3.3. Boundary Condition Modeling

To prevent rigid-body motion induced by the internal blast, nodes located at both ends and on the side surfaces are constrained using *NODE_SET and *BOUNDARY_SPC_SET, as shown in Figure 9a. Because the internal blast loading acts predominantly in the surface-normal direction, translational and rotational degrees of freedom are constrained for all

nodes except those within the blast-loaded region. For the PCCV model, unintended tendon motion may occur during the prestressing introduction if tendon ends are not restrained. Therefore, as illustrated in Figure 9b, tendon end nodes are defined using *NODE_SET and constrained via *BOUNDARY_SPC_SET. The interior surface of the straight wall segment subjected to internal blast pressure is defined using *LOAD_SEGMENT_SET. The detonation is assumed to occur at the center of the internal volume, and *LOAD_BLAST is applied using an air-burst option to represent blast-wave propagation and reflection within the internal air domain and to obtain an accurate displacement response of the wall section. In addition, a boundary-condition sensitivity study is conducted to reduce uncertainty associated with end restraints. Three representative boundary conditions are examined by varying translational and rotational constraints at the ends.

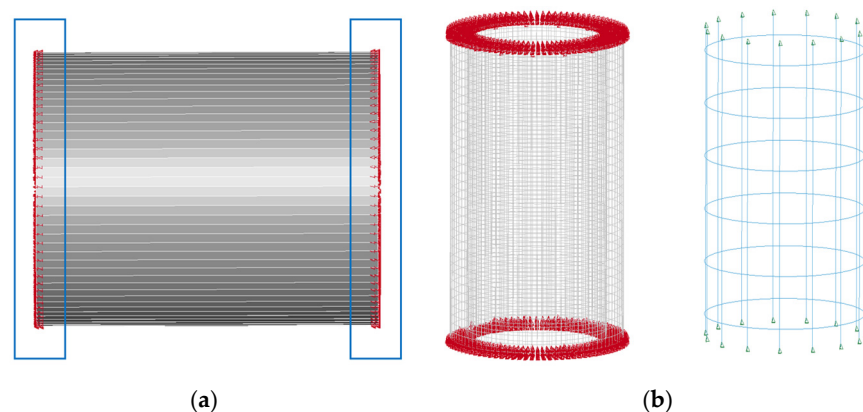


Figure 9. Boundary condition setup. (a) End-node selection with *NODE_SET; (b) End-node selection with *BOUNDARY_SPC_SET.

As shown in Figure 10, the pressure–time and displacement–time responses are extracted at mid-span (0°) and compared with the experimental results measured at the same location. Among the examined cases, the configuration with fully restrained ends and rotational restraint exhibited the closest agreement with the experimental responses at mid-span (0°), particularly in terms of peak displacement level, early-time transient oscillation characteristics, and the absence of unrealistically large permanent deformation. In contrast, the configuration allowing end translation showed pronounced displacement accumulation, while the configuration with rotational release exhibited an excessively stiff response with suppressed displacement amplitude. Based on these comparisons, the fully restrained configuration is selected as the baseline boundary condition for subsequent analyses to represent the effective restraint provided by the experimental support system. Although venting and pressure relief are primarily governed by the structural geometry and the blast-loading formulation, the end-restraint condition can influence the effective pressure–time decay observed at mid-span (0°) through interaction between wall motion, internal wave reflection, and the resulting transient confinement level. Accordingly, the boundary-condition sensitivity study in Figure 10 is interpreted as also reflecting the sensitivity of the effective venting/pressure-relief response at mid-span (0°) under the experimentally consistent open-ended configuration.

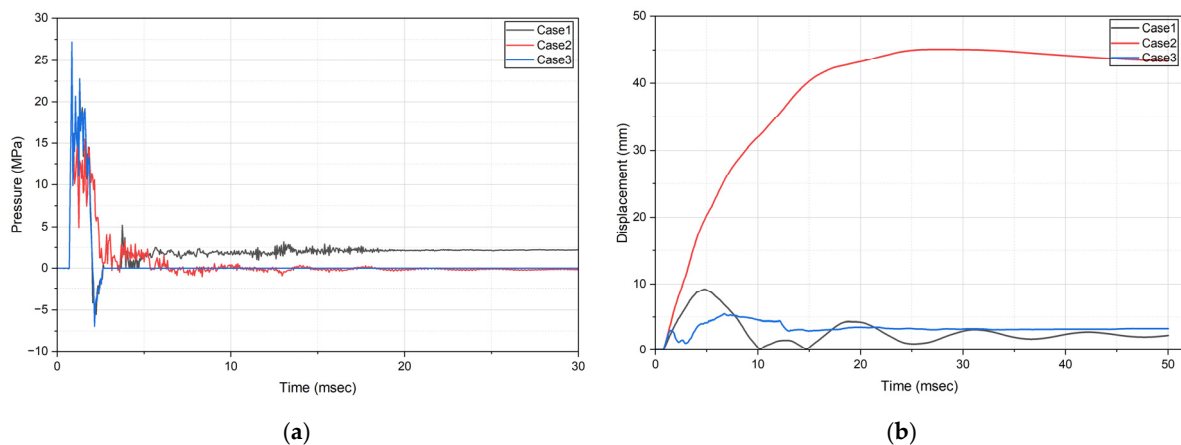


Figure 10. Comparison of pressure–time and displacement–time responses under different boundary conditions (mid-span (0°)) (a) Pressure–time histories; (b) Displacement–time histories.

3.4. Load Blast Modeling

Blast loading is implemented using the *LOAD_BLAST local card in LS-DYNA. This formulation requires TNT-referenced input and converts an explosive mass to an equivalent TNT mass based on the Chapman–Jouguet (CJ) detonation velocity relationship, as expressed in Equation (1).

$$M_{TNT} = M_e \frac{v_e^2}{v_{TNT}^2} \quad (1)$$

In Equation (1), M_{TNT} denotes the equivalent TNT mass used as input for *LOAD_BLAST, M_e is the mass of the explosive, v_e is the Chapman–Jouguet detonation velocity of the explosive, and v_{TNT} is the Chapman–Jouguet detonation velocity of TNT. Because the *LOAD_BLAST formulation is referenced to TNT, the ANFO charge mass is converted into an equivalent TNT mass using the TNT equivalency factor recommended in UFC 3-340-02 [16]. The conversion is expressed in Equation (2).

$$TNT_{kg} = ANFO_{kg} \times 0.82_{TNT \text{ equivalency factor}} \quad (2)$$

In Equation (2), TNT_{kg} is the equivalent TNT mass in kilograms, $ANFO_{kg}$ is the ANFO mass in kilograms, and 0.82 represents the TNT equivalency factor for ANFO adopted from UFC 3-340-02 [16].

The TNT equivalency factor adopted in this study is consistent with common engineering practice for blast-resistant structural analysis and has been employed in recent experimental studies involving ANFO-based explosives. Previous studies have shown that far-field positive-phase blast parameters of ammonium nitrate–based explosives can be reasonably characterized using TNT equivalency in terms of impulse and peak overpressure [17]. In addition, experimental investigations have demonstrated that TNT-equivalent modeling provides reliable correlation with measured structural responses of reinforced concrete panels subjected to internal and semi-confined blast loading conditions [18].

The case labels RC35–RC55 and PSC50–PSC70 follow the experimental naming convention based on ANFO charge weight expressed in pounds. However, all numerical inputs and outputs in the present simulations are handled exclusively in SI units. Accordingly, the ANFO charge masses are first converted from pounds to kilograms and then transformed into equivalent TNT masses prior to application of the *LOAD_BLAST model. Based on this procedure, ANFO charge masses of 35, 45, 50, 55, 60, 65, and 70 lb are converted into TNT-equivalent masses of 13.02, 16.74, 18.60, 20.46, 22.32, 24.17, and 26.04 kg, respectively.

The converted TNT-equivalent charge masses are summarized in Table 3 and are used as input for the blast loading in each simulation.

Table 3. Conversion of ANFO charge weight (lb) to TNT-equivalent mass (kg) used as input in LS-DYNA simulations.

ANFO (lb)	35	45	50	55	60	65	70
TNT (kg)	13.02	16.74	18.60	20.46	22.32	24.17	26.04

It is noted that the ANFO-to-TNT equivalency method is an energy-based approximation and does not explicitly account for confinement, venting/pressure-relief efficiency, or charge-location-dependent details in the internal pressure–time history. In the present study, these effects are represented through explicit modeling choices rather than by modifying the equivalency factor. Venting and pressure relief are modeled by adopting an open-ended geometry consistent with the experimental configuration, allowing rapid pressure release through the open ends immediately after detonation. Consequently, the sustained internal pressure acting on the wall is spatially limited in the open-ended configuration. In addition, the end-restraint sensitivity described in Section 3.3 provides a complementary assessment of how boundary-induced changes in wall motion can affect the effective pressure decay measured at mid-span (0°) under the same venting path. For consistency with the stiffness-based force–displacement (F–U) evaluation framework described in Section 6.1 and adopted in prior studies, the effective blast influence region is defined around the mid-span as a finite zone extending 1000 mm on each side of the center, and the blast load is applied only within this region. The detonation location is fixed to match the experimental setup (Figure 3) to maintain specimen-consistent loading conditions and to avoid introducing additional variability. Therefore, sensitivity studies on charge location and the TNT equivalency factor are not conducted, because the primary objective is to evaluate relative response trends and stiffness-based failure-inducing load levels under experimentally consistent conditions rather than to reproduce the detailed internal pressure–time history. The applicability range and potential uncertainties associated with the TNT-equivalency-based *LOAD_BLAST model under internal and semi-confined conditions are explicitly acknowledged, and the results should be interpreted as response- and stiffness-based estimates within the scope of the stated assumptions.

3.5. Sensitivity Analyses and Model Robustness

To evaluate the robustness of the numerical model and to verify that the predicted responses are not governed by numerical artifacts, sensitivity analyses are conducted with respect to three key modeling parameters: (i) mesh resolution, (ii) concrete compressive strength, and (iii) blast influence length.

All sensitivity analyses are evaluated using response quantities extracted at the mid-span (0°), which represents the most critical location governing the global structural response under internal blast loading. For each sensitivity parameter, both qualitative response characteristics (time-history shape, oscillatory behavior, and decay trend) and quantitative peak response values are examined. Error ratios are calculated with respect to the baseline model configuration adopted in this study in order to assess the relative deviation associated with each parameter variation. Based on the combined assessment of physical consistency, numerical stability, and error magnitude, the appropriateness of the selected modeling parameters is verified.

3.5.1. Mesh Sensitivity Analysis

Mesh sensitivity analysis is conducted to evaluate the influence of spatial discretization on the predicted reflected pressure response, which is particularly sensitive to stress-wave resolution and numerical dispersion in explicit dynamic analyses. Three mesh resolutions, 40, 50, and 60 mm, are examined for both open-ended and enclosed structural configurations.

The comparison focused on reflected pressure–time histories extracted at the mid-span (0°), and the results are presented in Figure 11, while the corresponding peak reflected pressure values and error ratios are summarized in Table 4.

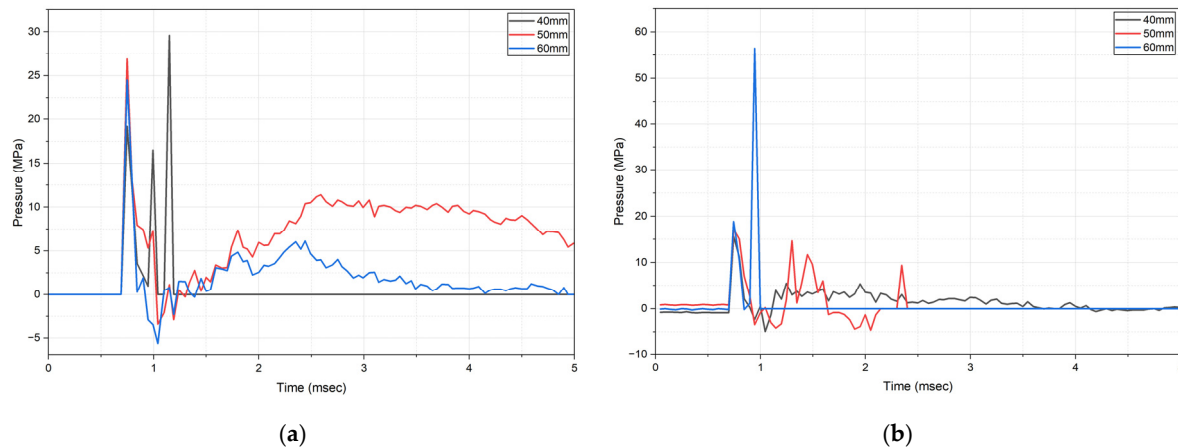


Figure 11. Mesh sensitivity analysis based on reflected pressure–time histories at the mid-span (0°). (a) Open; (b) Enclosed.

Table 4. Summary of peak response values and error ratios for mesh resolution sensitivity analysis (mid-span (0°)).

Category	Case	Open-Ended Structure		Enclosed Structure	
		Peak Reflected Pressure (MPa)	Error Ratio (%)	Peak Reflected Pressure (MPa)	Error Ratio (%)
Mesh resolution	40 mm	20.3	−4.25	15.5	−9.9
	50 mm	21.2	-	17.2	-
	60 mm	20.4	−3.77	56.5	228.5

As shown in Figure 11a, the 40 mm mesh for the open-ended configuration produced multiple spurious pressure peaks and irregular oscillatory behavior immediately after detonation, indicating numerical instability caused by excessive wave reflection and local mesh distortion. In contrast, the 60 mm mesh exhibited an overly smoothed pressure response with delayed peak formation. In the enclosed configuration, this mesh resolution led to excessive pressure amplification due to insufficient spatial resolution of stress-wave interaction effects, as shown in Figure 11b.

Quantitative comparison of peak reflected pressures further supports these observations. As summarized in Table 4, the 40 mm mesh resulted in peak pressure deviations of −4.25% and −9.9% for the open-ended and enclosed configurations, respectively, relative to the baseline model. In contrast, the 60 mm mesh exhibited a markedly large error ratio of +228.5% in the enclosed configuration, clearly indicating nonphysical pressure amplification associated with mesh coarsening under confined blast conditions. The 50 mm mesh, however, yielded stable pressure–time histories with physically consistent peak magnitudes and decay characteristics. The predicted response captured a dominant first

pressure peak followed by a smaller secondary peak and gradual attenuation toward zero pressure, which is consistent with experimentally observed trends reported in previous studies [27,28].

Based on the combined qualitative assessment of time-history behavior and quantitative evaluation of peak pressure error ratios, a mesh size of 50 mm is selected as the baseline mesh resolution for all subsequent analyses. This mesh size provides an appropriate balance between numerical stability, stress-wave resolution, and computational efficiency, ensuring that the predicted structural responses are governed by physical behavior rather than mesh-induced numerical artifacts.

3.5.2. Concrete Compressive Strength Sensitivity

To evaluate the influence of material property uncertainty on the predicted structural response, a sensitivity analysis is performed by varying the concrete compressive strength by $\pm 10\%$ around the reference value of 40 MPa, corresponding to compressive strengths of 36 MPa and 44 MPa. All analyses are evaluated using pressure–time and displacement–time responses extracted at the mid-span (0°), which governs the global response under internal blast loading. The reflected pressure–time and displacement–time histories for the three strength levels are presented in Figure 12, respectively, and the corresponding peak response values and error ratios are summarized in Table 5. As shown in Figure 12, variations in concrete compressive strength resulted in only minor changes in the overall response shape, including the time to peak response, oscillation characteristics, and decay behavior. The pressure–time histories exhibited nearly identical peak timing and similar secondary peak development, while the displacement–time histories showed consistent global deformation patterns across all strength levels.

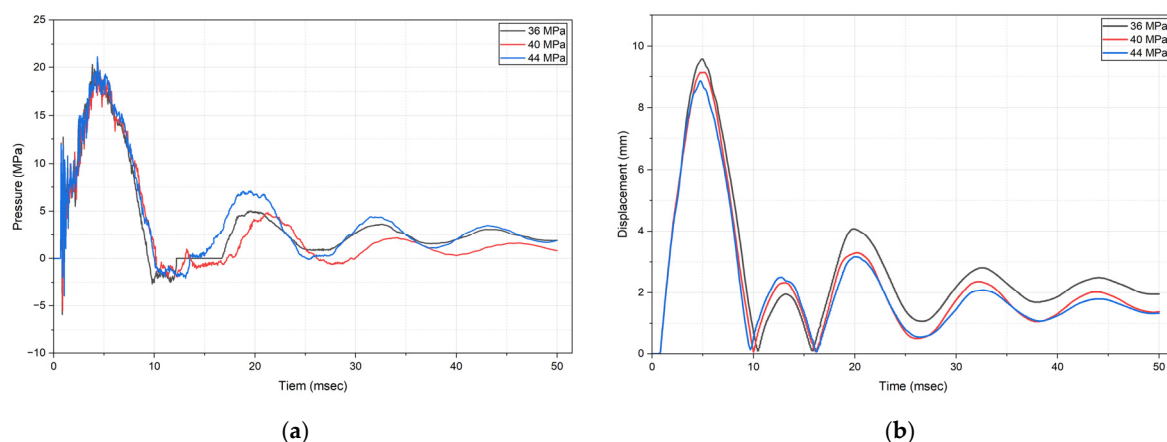


Figure 12. Concrete compressive strength sensitivity analysis at the mid-span (0°). (a) Reflected pressure–time histories; (b) Displacement–time histories.

Table 5. Summary of peak response values and error ratios for concrete compressive strength sensitivity analysis (mid-span (0°)).

Category	Case	Peak Reflected Pressure (MPa)	Error Ratio (%)	Peak Displacement (mm)	Error Ratio (%)
Concrete compressive strength	36 MPa (−10%)	20.2	−0.98	9.7	+5.43
	40 MPa	20.4	0	9.2	0
	44 MPa (+10%)	21.2	+3.92	8.9	−3.26

Quantitatively, as summarized in Table 5, the peak reflected pressure varied within -0.98% to $+3.92\%$ relative to the reference case, while the peak displacement varied within $+5.43\%$ to -3.26% . These limited error ratios indicate that moderate variations in concrete compressive strength have a relatively small influence on both pressure and displacement response quantities. The slightly larger variation observed in peak displacement compared to peak pressure reflects the cumulative effect of stiffness variation on global deformation rather than a change in the applied blast loading itself. Overall, the results demonstrate that the numerical model exhibits stable and physically consistent behavior with respect to reasonable uncertainty in concrete compressive strength. The limited sensitivity observed in both pressure and displacement responses confirms that the adopted reference compressive strength of 40 MPa provides an appropriate and robust basis for subsequent analyses, and that the predicted structural behavior is governed primarily by global response mechanisms rather than by moderate material property variations.

3.5.3. Blast Influence Length

The spatial extent of blast influence is evaluated by extracting reflected pressure and displacement responses at distances of 0, 500, 1000, and 1500 mm from the mid-span (0°). The corresponding reflected pressure–time and displacement–time histories are presented in Figure 13, and quantitative comparisons of peak response values are summarized in Table 6. As shown in Figure 13, both reflected pressure and displacement responses exhibit a clear spatial attenuation trend with increasing distance from the blast center. The mid-span region shows the most pronounced response, characterized by the highest peak pressure and displacement, whereas responses at locations farther from the center are progressively reduced. This trend is quantified in Table 6, which indicates that the peak reflected pressure decreases by approximately 57% at 500 mm, 78% at 1000 mm, and more than 90% at 1500 mm relative to the mid-span value. A similar attenuation pattern is observed for displacement, with reductions of approximately 29%, 58%, and 88% at 500, 1000, and 1500 mm, respectively. These results demonstrate that the structural response under internal blast loading is governed predominantly by the central region of the wall. In particular, responses beyond ± 1000 mm from the mid-span are significantly attenuated and contribute minimally to the global deformation behavior. Accordingly, defining the effective blast loading zone as ± 1000 mm around the mid-span is sufficient to capture the dominant pressure–structure interaction and overall dynamic response of the structure within the numerical model.

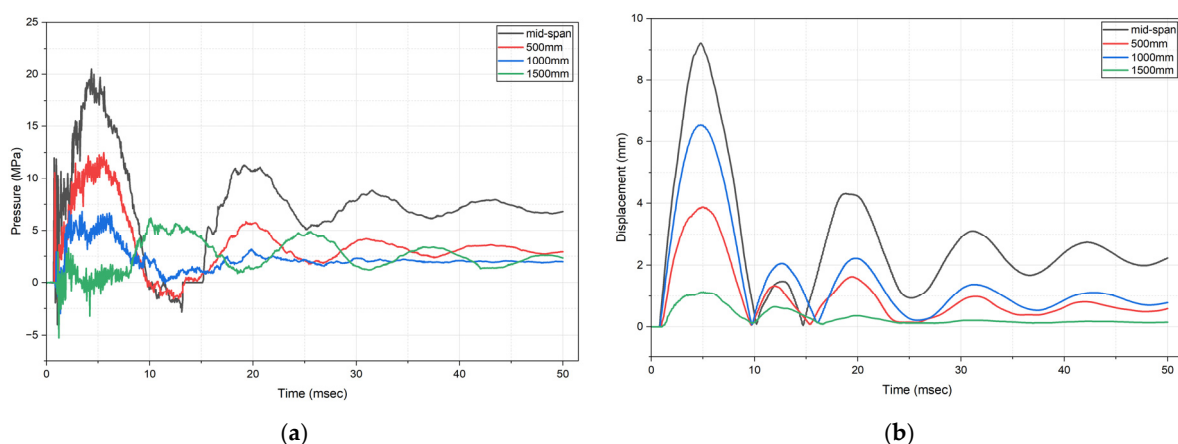


Figure 13. Blast influence length sensitivity analysis at different distances from the mid-span (0°). (a) Reflected pressure–time histories; (b) Displacement–time histories.

Table 6. Peak reflected pressure and displacement at different distances from the mid-span under internal blast loading.

Distance from Mid-Span (mm)	Peak Reflected Pressure (MPa)	Pressure Reduction (%)	Peak Displacement (mm)	Displacement Reduction (%)
0 (mid-span)	27.2	–	9.22	–
500 mm	11.8	–56.6	6.53	–29%
1000 mm	6.1	–77.6	3.87	–58%
1500 mm	2.3	91.5	1.14	–88%

Overall, the blast influence length sensitivity analysis confirms that the adopted spatial extent of blast loading is physically justified and numerically robust. The rapid attenuation of both pressure and displacement responses outside the central region indicates that the selected influence length appropriately balances computational efficiency and response fidelity, ensuring that subsequent analytical results are governed by structural behavior rather than by numerical artifacts associated with load application extent.

4. Simulation Results

Displacement is measured at specific locations during the tests, as shown in Figures 3 and 4. However, the internal blast pressure could not be directly measured, so it was obtained through simulation. To enable comparison with the test results, the simulation data is extracted at the same measurement locations. In addition, to identify the way in which internal pressure is applied to the structure, blast pressure and displacement are also examined along the wall of the specimen. The differences between the RCCV and PCCV responses are summarized in Tables 7 and 8.

Table 7. Maximum internal reflected pressure.

Blast Mass	Maximum Internal Pressure (MPa)					
	Open-Ended Structure			Enclosed Structure		
	ANFO (lb)	Mid-Span (0°)	Mid-Span (90°)	1000 mm	Mid-Span (0°)	Mid-Span (90°)
RC35	27.2	27.2	7.9	20.8	20.9	7.82
RC45	28.8	28.8	13.1	24.5	24.2	8.57
RC50	37.4	37.4	13	29.6	29.6	9.32
RC55	44.4	44.4	12	31.2	31.2	8.86
PSC50	37.3	37.4	12.8	29.3	29.2	9.23
PSC55	44.4	44.4	12.1	28.3	28.2	10.4
PSC60	50.1	50.1	13.6	29.6	30.0	10.5
PSC65	45.6	44.5	15.3	32.9	32.9	12.6
PSC70	50.9	50.9	15.6	35.1	35.1	13.1

Table 8. Maximum displacement.

Blast Mass	Maximum Displacement (mm)					
	Open-Ended Structure			Enclosed Structure		
	ANFO (lb)	Mid-Span (0°)	Mid-Span (90°)	1000 mm	Mid-Span (0°)	Mid-Span (90°)
RC35	9.22	9.17	3.87	9.55	9.54	4.12
RC45	11.6	11.4	4.77	11.5	11.4	5.11

Table 8. Cont.

Blast Mass	Maximum Displacement (mm)					
	Open-Ended Structure			Enclosed Structure		
	ANFO (lb)	Mid-Span (0°)	Mid-Span (90°)	1000 mm	Mid-Span (0°)	Mid-Span (90°)
RC50	12.2	13.0	5.27	13.0	12.7	5.65
RC55	13.7	13.7	7.21	14.2	14.3	6.14
PSC50	12.2	12.0	4.96	12.5	11.9	5.43
PSC55	14.3	13.5	5.41	14.9	14.4	5.83
PSC60	14.6	14.9	5.89	14.8	14.9	6.33
PSC65	15.7	15.7	6.40	16.5	16.1	6.85
PSC70	18.9	18.7	6.72	19.1	18.6	7.17

4.1. Analysis of Simulation Results

As shown in Figure 14, the reflected pressures at the mid-span (0°) of open-ended RCCV detonated with 35, 45, 50, and 55 lbs ANFO charges are 20.5, 24.6, 29.6, and 31.2 MPa, respectively, while for the enclosed RCCV, the corresponding values are 20.8, 24.5, 29.6, and 31.2 MPa, respectively, indicating an increase induced by charge weights of 35, 45, 50, and 55 lbs in pressure with increasing charge weight in both open-ended and enclosed cases. The associated displacements increased accordingly, from 9.22, 11.6, 12.2, and 13.7 mm, respectively, for the open-ended RCCV. For the enclosed specimens, the displacements increased from 9.55, 11.5, 13.0, and 14.2 mm, respectively. A similar trend is observed at the mid-span (90°) location. At the 1000 mm location, both blast pressure and displacement are smaller than at the mid-span, but they still increase with charge weight.

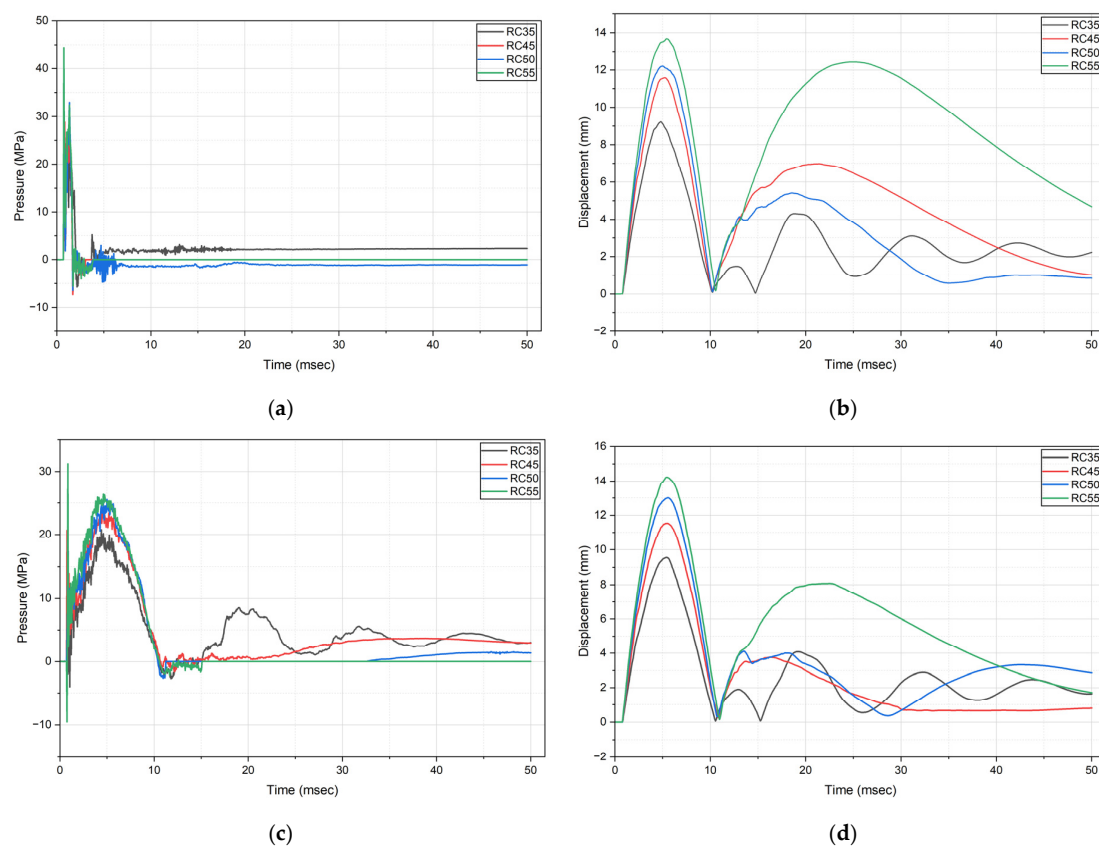


Figure 14. RCCV simulation results. (a) Open-ended RCCV pressure (mid-span (0°)); (b) Open-ended RCCV displacement (mid-span (0°)); (c) Enclosed RCCV reflected internal pressure (mid-span (0°)); (d) Enclosed RCCV displacement (mid-span (0°)).

As shown in Figure 15, the reflected pressures at the mid-span (0°) of the open-ended PCCV subjected to 50, 55, 60, 65, and 70 lb ANFO charges are 37.3, 44.4, 50.1, 45.6, and 50.9 MPa, respectively. For the enclosed PCCV, the corresponding reflected pressures under the same charge conditions are evaluated as 29.3, 28.9, 29.6, 32.9, and 35.1 MPa, respectively. In the open-ended configuration, the reflected pressure exhibited an increasing trend with increasing charge weight, and a similar overall trend is observed for the enclosed configuration. Correspondingly, the structural displacement increased with charge weight. For the open-ended PCCV, the maximum mid-span displacements are 12.2, 14.3, 14.6, 15.7, and 18.9 mm, respectively. For the enclosed PCCV, the mid-span displacements under the same charge conditions increased to 12.5, 14.9, 14.8, 16.5, and 19.1 mm, respectively. Similar trends are observed at the mid-span (90°) location. At the location 1000 mm away from the center, both the reflected pressure and the displacement are smaller than those at the mid-span; however, an overall increasing tendency with increasing charge weight is still maintained.

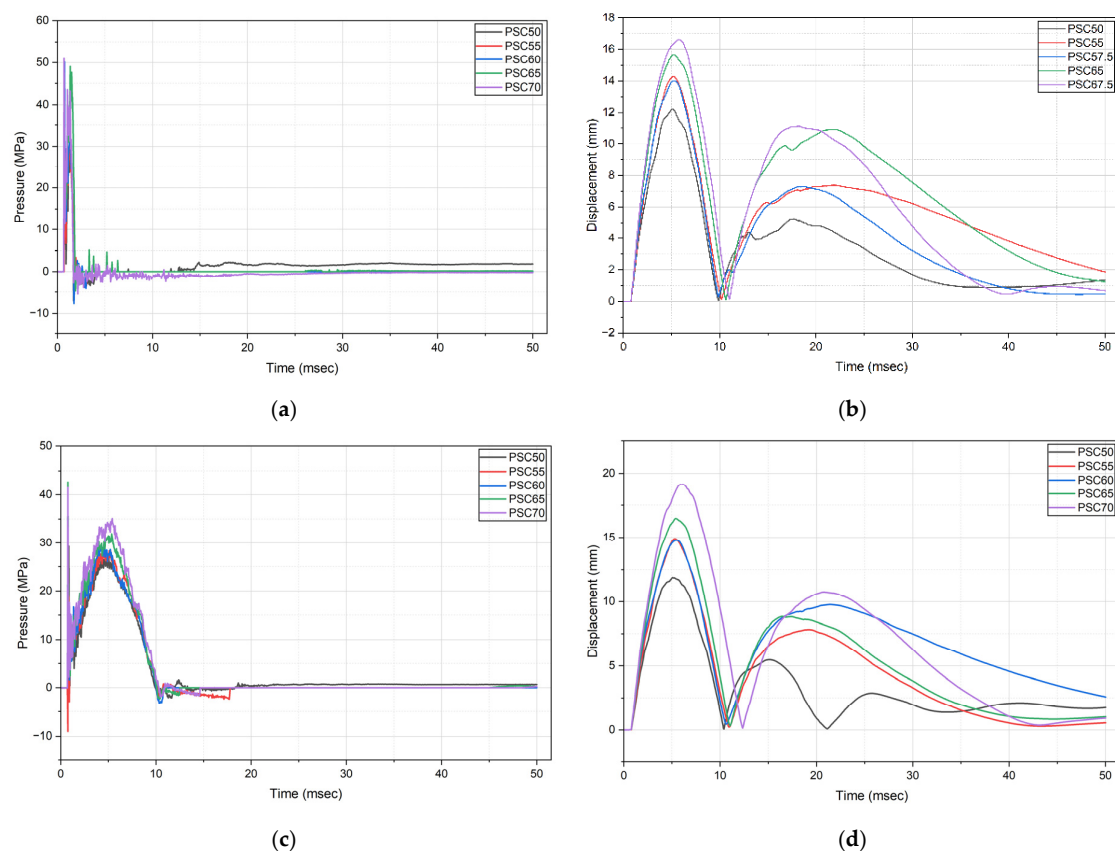


Figure 15. PCCV simulation results. (a) Open-ended PCCV pressure (mid-span (0°)); (b) Open-ended PCCV displacement (mid-span (0°)); (c) Enclosed PCCV reflected internal pressure (mid-span (0°)); (d) Enclosed PCCV displacement (mid-span (0°)).

As shown in Figures 13 and 14, both RCCV and PCCV exhibited a short-duration blast pressure response in the open-ended configuration, with the pressure duration remaining below 5 msec and rapidly decaying to near-zero values immediately after detonation. In contrast, for the enclosed configuration, the internal blast pressure persisted for up to approximately 10 msec, and a residual pressure level is observed even after the initial decay phase. These results indicate that venting in the open-ended structures effectively relieved internal pressure shortly after detonation, whereas pressure accumulation and repeated wave reflections in the enclosed structures led to prolonged pressure duration and residual internal pressure.

4.2. Comparison of Reflected Pressure for RCCV and PCCV at Equal Blast Charges

As shown in Figures 14 and 15, the pressure–time histories inside the enclosed structures exhibit oscillatory behavior characterized by alternating positive and negative pressure phases, followed by a gradual decay. In contrast, the open-ended structures show a single dominant reflected-pressure peak immediately after detonation, followed by a rapid decay toward near-zero pressure. This difference reflects the fundamental role of boundary conditions, where rapid pressure venting governs the response of open-ended structures, while repeated wave reflection and superposition dominate the enclosed configuration. The maximum internal reflected pressures extracted from the simulations are summarized in Table 7. Table 7 indicates that, for several blast charge levels, the local maximum reflected pressure at the mid-span is higher in the open-ended configuration than in the enclosed configuration for both RCCV and PCCV. This trend is particularly evident at relatively lower charge levels. This observation does not indicate a modeling inconsistency, but instead highlights the distinction between instantaneous local peak pressure and the overall severity of internal blast loading.

In the open-ended configuration, a large reflected-pressure peak forms immediately after detonation; however, the pressure duration remains extremely short due to rapid venting through the open ends. As a result, the internal pressure quickly dissipates and does not generate sustained loading on the internal wall surfaces. In contrast, in the enclosed configuration, multiple pressure peaks form sequentially after the initial peak due to repeated wave reflections within the confined cavity. Consequently, the pressure–time histories do not exhibit a single-peak pattern but instead show prolonged duration with residual pressure remaining inside the structure. These pressure histories represent not a single reflected pressure wave generated solely by the initial blast, but reflected pressures that evolve in a coupled manner with the displacement response of the structural elements in LS-DYNA. As structural displacement increases or decreases, the reflected pressure acting on the elements varies accordingly. The corresponding displacement responses are summarized in Table 8. For both RCCV and PCCV, the enclosed configuration consistently produces larger displacements than the open-ended configuration at the same blast charge. At the mid-span location, the displacement of the enclosed structures reaches approximately 1.0 to 1.1 times that of the open-ended structures, depending on the charge level and structural type. At the location 1000 mm away from the mid-span, the difference becomes more pronounced, indicating a broader spatial influence of the internal blast loading under enclosed conditions. In the open-ended structures, the displacement at 1000 mm remains approximately 0.3 to 0.4 times the mid-span displacement for both RCCV and PCCV. This behavior indicates that rapid pressure release limits the effective blast demand to the central region and causes a rapid attenuation of structural response with distance. In contrast, in the enclosed structures, the displacement at the same location reaches approximately 0.7 to 0.8 times the mid-span displacement, reflecting sustained internal pressure and repeated shock-wave reflections acting over a wider region of the internal surface.

These observations demonstrate that, although the maximum displacement consistently occurs at the mid-span for both boundary conditions, the spatial extent and duration of blast-induced deformation differ markedly. Rapid venting in the open-ended configuration limits both pressure duration and spatial influence, whereas pressure accumulation and repeated reflections in the enclosed configuration impose a more severe overall structural demand, even in cases where the instantaneous local peak reflected pressure is lower than that observed in the open-ended configuration.

4.3. Comparison of Displacement for RCCV and PCCV at Equal Blast Charge

As shown in Table 8, the maximum displacement responses of reinforced concrete containment vessels (RCCVs) and prestressed concrete containment vessels (PCCVs) under identical internal ANFO blast charges are compared based on the numerical simulation results. The comparison focuses on two representative locations, namely the mid-span (0°) and a position 1000 mm away from the center, for both open-ended and enclosed configurations. In the open-ended configuration, the mid-span displacements of RC50 and RC55 reach 12.2 mm and 13.7 mm, respectively, while the corresponding displacements at 1000 mm reduce to 5.27 mm and 7.21 mm. For the PSC specimens, the mid-span displacements of PSC50 and PSC55 reach 12.2 mm and 14.3 mm, and the displacements at 1000 mm reduce further to 4.96 mm and 5.41 mm, respectively. In the enclosed configuration, RC50 and RC55 exhibit mid-span displacements of 13.0 mm and 14.2 mm, with corresponding displacements of 5.65 mm and 6.14 mm at 1000 mm. The PSC specimens show mid-span displacements of 11.9 mm and 14.9 mm, while the displacements at 1000 mm reach 5.43 mm and 5.83 mm. As summarized in Table 8, the differences in displacement between RCCV and PCCV at the mid-span remain relatively small for both boundary configurations.

At this location, the structural response is governed primarily by direct interaction between the wall and the reflected blast pressure, and the contribution of prestressing to the peak displacement response remains limited. In contrast, at the location 1000 mm away from the center, the PSC specimens consistently exhibit smaller displacement responses than the RC specimens for comparable blast charges. This trend indicates that the prestressing effect contributes more effectively to displacement control in regions where the blast demand is spatially attenuated. To interpret the displacement responses in terms of structural damage severity, displacement-based damage criteria proposed by ASCE are referenced, as summarized in Table 9. These criteria are originally established for conventional reinforced concrete structural elements such as beams, slabs, columns, and shear walls, and classify damage levels based on the ratio of maximum deflection to the overall structural length (δ/L). Because explicit damage criteria for reinforced concrete tubular or vessel-type structures subjected to internal blast loading are not currently available, these criteria are adopted in this study as a methodological reference. Based on the average bending- and shear-dominated failure criteria for beams and slabs summarized in Table 10, a modified damage classification is applied by assuming a bending/shear contribution ratio of 35%. The maximum displacement obtained from the numerical simulations is normalized by a blast-affected length of 2000 mm, and the resulting displacement-to-length ratios are presented in Table 11.

Table 9. Typical failure criteria for structural elements of reinforced concrete structures (ASCE, 1999).

Element Type	Material Properties	Failure Type	Criteria	Light Damage	Moderate Damage	Severe Damage
Beam	Reinforced Concrete ($\rho > 0.5\%/face$)	Global bending/ Membrane response	Ratio of center-line deflection to span, (δ/L)	4%	8%	15%
		Shear	Average shear strain across section, γ_v	1%	2%	3%
Slab		Bending/Membrane	δ/L	4%	8%	15%
Shear		γ_v	1%	2%	3%	
Column		Compression	Shortening/height	1%	2%	4%
Load-bearing wall		Compression	Shortening/height	1%	2%	4%
Shear wall		Shear	Average shear strain across section	1%	2%	3%

Table 10. Average of beam and slab failure criteria for structural elements of reinforced concrete structures.

Element Type	Damage Type	Bending/Shear			
		50%	35%	25%	15%
Beam & Slab	Light damage (δ/L)	2.5%	1.75%	1.25%	0.75%
	Moderate damage (δ/L)	5%	3.5%	2.5%	1.5%
	Severe damage (δ/L)	9%	6.3%	4.5%	2.7%

Table 11. Comparison of maximum displacement-to-length (δ/L) for open-ended and enclosed structures.

Blast Mass	Maximum Displacement/Length (δ/L)	
	Open-Ended Structure	Enclosed Structure
ANFO (lb)		
RC35	0.46%	0.48%
RC45	0.58%	0.58%
RC50	0.61%	0.65%
RC55	0.68%	0.71%
PSC50	0.61%	0.63%
PSC55	0.71%	0.74%
PSC60	0.73%	0.74%
PSC65	0.78%	0.83%
PSC70	0.94%	0.96%

As shown in Table 11, all analyzed cases for both open-ended and enclosed configurations remain below the light-damage threshold of 1.75%. Specifically, the maximum δ/L ratios are less than 1.0% even for the highest internal blast charges considered in this study. These results indicate that, according to the referenced ASCE-based displacement criteria, the structural response corresponds to a no-damage or very low damage state under the examined loading conditions. It should be emphasized that the applied damage classification does not represent definitive failure limits for reinforced concrete containment or vessel-type structures. Rather, it is introduced to provide a consistent and rational framework for interpreting displacement-based response levels under internal blast loading, in the absence of established damage criteria specifically developed for reinforced concrete tubular or vessel-type structural systems.

5. Stiffness Analysis of RCCV and PCCV Test Results Based on F-U Curves [27,28]

Structural stiffness of reinforced concrete and prestressed concrete walls under internal blast loading is difficult to evaluate directly because the load is highly transient and the structural response is strongly dynamic and nonlinear. Previous studies addressed this limitation by constructing force–displacement (F–U) relationships from measured pressure and displacement responses and by extracting elastic and plastic stiffness parameters from regression slopes of the F–U curves. Following the same framework, this study constructed F–U curves using internal pressure and displacement time histories

obtained from displacement-calibrated numerical simulations for both open-ended and enclosed RCCV and PCCV configurations. The resulting stiffness parameters are then used to estimate failure-inducing internal blast loads for real-scale containment vessels. Unlike conventional blast assessments that primarily report demand and damage using peak overpressure, impulse, or empirical pressure-based thresholds, the present work adopts a response-based formulation that connects internal blast loading to structural resistance through displacement time histories and the corresponding F–U relationship. This formulation is particularly suitable for semi-confined internal explosion conditions where internal pressure measurements are impractical and where pressure time histories are strongly affected by venting and wave reflections.

6. Stiffness Analysis Method and Definition of the F-U Relationship [27,28]

It is difficult to theoretically calculate or experimentally determine the structural stiffness coefficients of reinforced concrete and prestressed reinforced concrete members subjected to internal blast loading because the response is highly transient and nonlinear. Previous studies addressed this limitation by constructing a case-wise force–displacement relationship across multiple blast cases and deriving stiffness parameters from the regression slopes of a bilinear representation. In the present study, the force–displacement relationship is not defined as a time-domain response obtained from a single blast event. Instead, a case-wise force–displacement relationship is constructed by extracting one representative value of data from each charge case. For each internal blast case, the maximum internal blast load F_{max} and the corresponding maximum wall displacement U_{max} are identified. The resulting (U_{max}, F_{max}) data obtained from multiple charge levels are plotted as discrete points and connected to form a bilinear force–displacement relationship. The initial segment extending from the origin to the first approximately linear range of points represents the elastic-dominant response, and the slope of this segment defines the elastic stiffness K_{el} . Beyond this range, a clear reduction in slope is observed, indicating a transition in structural behavior. The subsequent segment represents the post-yield response, and its slope defines the plastic stiffness K_{pl} . The stiffness transition point is defined as the first charge level at which a distinct change in slope appeared in the case-wise relationship between F_{max} and U_{max} . This transition occurred at RC35 for the reinforced concrete vessel and at PSC50 for the prestressed reinforced concrete vessel, as shown in Figure 16.

$$F_{max} = (K \times U_{max}) \quad (3)$$

Accordingly, the bilinear relationship between F_{max} and U_{max} reflects the progressive transition from elastic-dominant deformation to plastic-dominant deformation as the internal blast demand increases. Under this definition, the relationship between the maximum internal blast load F_{max} , the wall stiffness K , and the maximum displacement U_{max} is expressed by Equation (3). The wall stiffness is decomposed into elastic and plastic components such that K is represented by the combined contribution of K_{el} and K_{pl} , while the maximum displacement U_{max} is similarly decomposed into elastic and plastic components. In practice, the elastic stiffness K_{el} is obtained from the regression slope of the initial segment of the case-wise F_{max} – U_{max} points, and the plastic stiffness K_{pl} is obtained from the regression slope of the subsequent segment following the stiffness transition. The resulting bilinear stiffness parameters are determined consistently for both reinforced concrete and prestressed reinforced concrete vessel configurations and are used as the basis for stiffness comparison and subsequent estimation of failure-inducing loads at real scale.

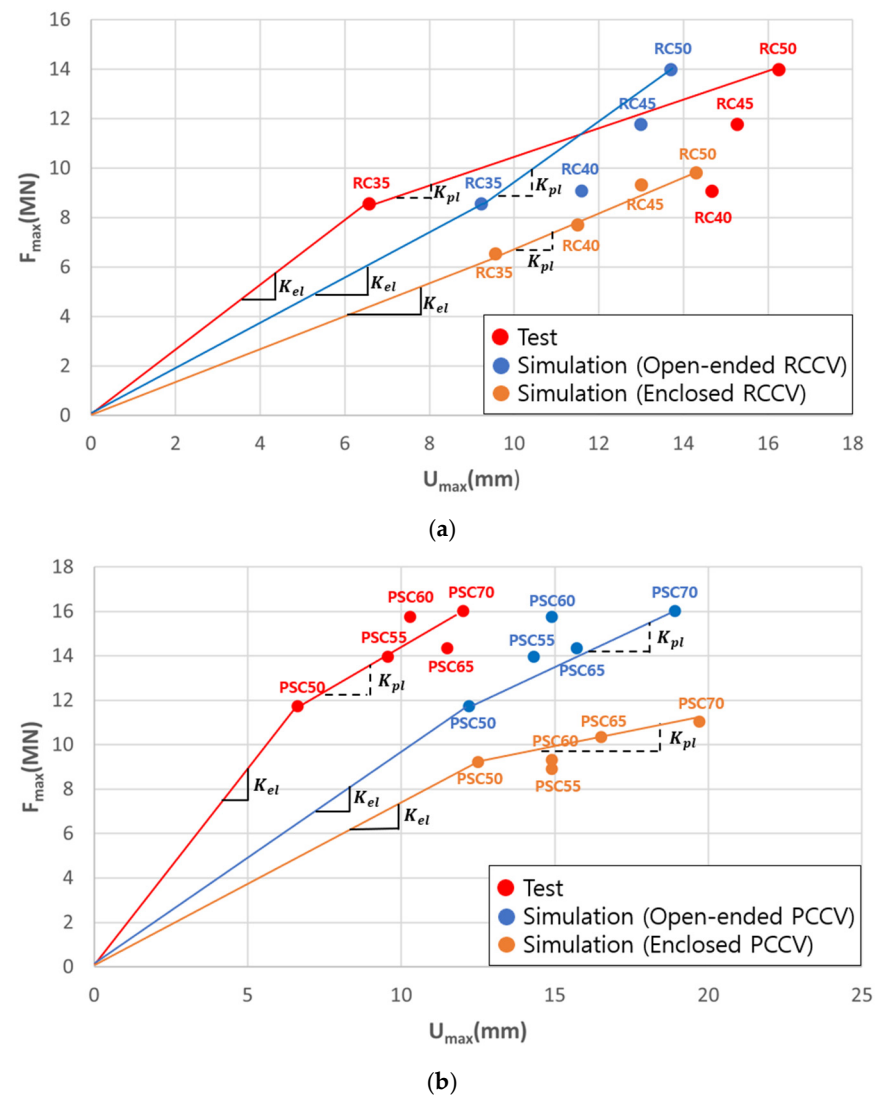


Figure 16. Internal wall structural stiffness of RCCV and PCCV. (a) RCCV; (b) PCCV.

6.1. Assumption of Internal Blast Load Distribution and Load Calculation

Because direct determination of the spatial distribution of internal blast pressure is difficult, a simplified and conservative load distribution assumption is adopted following previous studies. The internal blast load is assumed to act predominantly on the wall segments near the mid-span. As shown in Figure 17, the majority of the internal blast pressure is assumed to be applied within a distance of $r_{internal}$ on both sides of the mid-span, corresponding to a total influence length of $2r_{internal}$. In the numerical simulations, the internal blast pressure is applied only to this region to maintain consistency with the stiffness evaluation framework. Under this assumption, the maximum internal blast load F_{max} is calculated from the maximum internal pressure P_{max} using Equation (4) [27,28].

$$F_{max} = 2\pi r_{internal} \times 2r_{internal} \times P_{max} \quad (4)$$

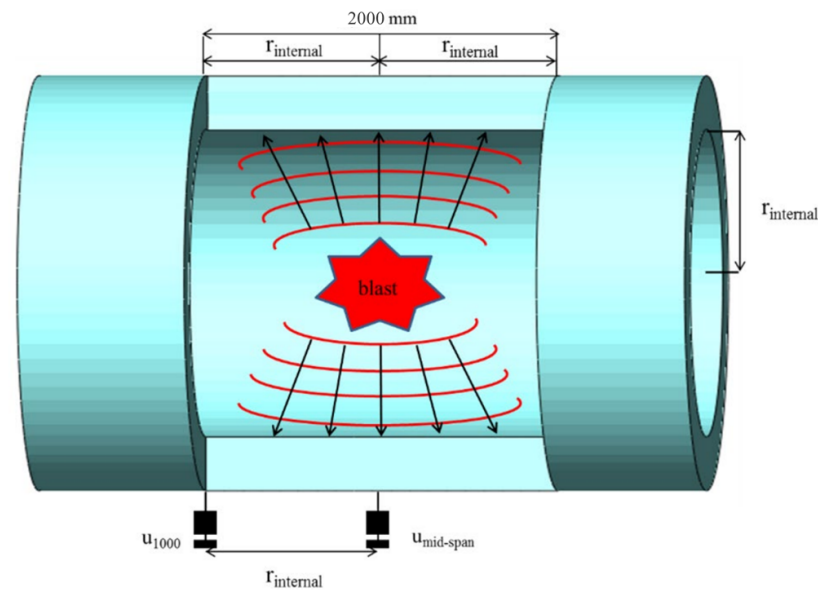


Figure 17. Internal blast simple model [27,28].

6.2. Comparison of Test and Simulation Stiffness Results for Open-Ended Vessels

For the open-ended configurations, force–displacement (F–U) curves are constructed, and the corresponding elastic and plastic stiffness parameters are evaluated using Equations (3) and (4). It should be noted that the F–U relationships labeled as “test-based” do not represent purely experimental results. In the experiments, displacement time histories are directly measured. However, the internal blast pressure acting on the vessel wall could not be obtained experimentally. Consequently, the maximum internal blast load F_{max} required for the F–U construction is derived from numerical simulations that are calibrated to reproduce the measured displacement responses. Under this hybrid procedure, the “test-based” F–U curves are constructed by combining experimentally measured displacements with simulation-derived internal blast loads, allowing a consistent stiffness evaluation framework to be applied in the absence of direct pressure measurements. As summarized in Table 12, for the open-ended RCCV, the elastic and plastic stiffness values obtained from the test-based F–U construction are 1.294 and 0.560, respectively, whereas the corresponding simulation-derived values are 0.929 and 1.209. For the open-ended PCCV, the test-based elastic and plastic stiffness values are 1.775 and 0.795, respectively, while the simulation yielded values of 0.963 and 0.639. The discrepancies between the test-based and simulation-derived stiffness values should therefore be interpreted in the context of the hybrid nature of the test-based evaluation, in which the load component is obtained from displacement-calibrated numerical pressure histories rather than direct experimental measurements. Despite these quantitative differences, both the test-based and simulation results consistently indicate that PSC structures exhibit higher structural stiffness than RC structures under internal blast loading conditions.

Table 12. Structural stiffness of open-ended RCCV and PCCV.

Type			K_{el}	K_{pl}
RCCV	Open-ended (mid-span (0°))	Test	1.294	0.560
		Simulation	0.929	1.209
	Enclosed (mid-span (0°))	Simulation	0.686	0.689

Table 12. Cont.

Type		K_{el}	K_{pl}
PCCV	Open-ended (mid-span (0°))	Test	1.775
		Simulation	0.963
	Enclosed (mid-span (0°))	Simulation	0.738
			0.253

6.3. Comparison of Simulation Stiffness Results for Open-Ended and Enclosed Vessels

For enclosed containment structures, the internal blast pressure generated by an internal explosion remains trapped within the structure and undergoes complex reflection and superposition processes. As a result, direct experimental measurement of internal pressure histories is practically infeasible. Owing to this limitation, most previous studies have focused primarily on experimental investigations of open-ended or partially vented configurations, while quantitative evaluations of the structural stiffness of fully enclosed containment vessels under internal blast loading remain limited. To address this research gap, the present study numerically investigates the internal blast response of enclosed RCCV and PCCV configurations and quantitatively evaluates their structural stiffness using a consistent force–displacement (F–U) framework. This approach constitutes a key contribution of this study. As summarized in Table 12, the elastic and plastic stiffness values of the enclosed RCCV are evaluated as 0.686 and 0.689, respectively, whereas those of the enclosed PCCV are 0.738 and 0.253. Compared with the open-ended configurations, both RCCV and PCCV exhibited reduced elastic stiffness under enclosed conditions. This reduction can be attributed to the fundamental change in the load transfer and structural response mechanisms caused by pressure trapping, whereby the internal blast pressure is not released but instead persists and interacts with the structure over a longer duration.

Despite this overall reduction, the enclosed PCCV consistently exhibited higher elastic stiffness than the enclosed RCCV. This observation is noteworthy and can be explained by the presence of prestressing-induced compressive stresses in the PSC structure, which effectively delay the initiation and propagation of tensile cracking under internal blast loading, thereby contributing to the preservation of initial structural stiffness. In contrast, the plastic stiffness of the PCCV is lower than that of the RCCV, suggesting a more localized deformation concentration after cracking in the prestressed structure. These findings demonstrate that, for enclosed containment structures, internal blast performance cannot be adequately characterized solely by peak displacement or damage indices. Instead, a stiffness-based evaluation provides additional insight into the structural response characteristics across different deformation stages. Moreover, by proposing a numerically based and internally consistent stiffness evaluation framework for enclosed conditions that are experimentally inaccessible, this study provides a valuable analytical basis for assessing the internal blast resistance of safety-critical structures, such as nuclear containment vessels, and for supporting future developments in performance-based design and evaluation criteria.

6.4. Prediction of Failure-Inducing Internal Blast Load F_{max} for Real-Scale Containment Vessels

In this study, the failure criterion of the containment vessel is defined in terms of the maximum internal blast load F_{max} , rather than the internal blast pressure. To extrapolate the stiffness obtained from the simulation model to the real-scale containment vessels, the sectional moment of inertia of the wall is first calculated using Equation (5). The corresponding elastic wall stiffness of the real-scale structure is then evaluated using the

stiffness relationship expressed in Equation (6), which relates the elastic modulus, sectional moment of inertia, and characteristic length of the wall.

$$I = \frac{bt^3}{12} \quad (5)$$

$$K_{el} = \frac{EI}{L^3} \quad (6)$$

To establish a consistent failure criterion between the simulation model and the real-scale structure, a curvature-based formulation is adopted. The relationship between curvature and bending moment is expressed by Equation (7), while the bending moment induced by the internal blast load is represented by Equation (8). This formulation is based on a simplified representation of the load distribution and boundary conditions and is intended to provide a conservative estimate of the structural response.

$$\frac{M}{EI} = \varnothing \quad (7)$$

$$M = \frac{F_{max}}{2} \cdot \frac{L}{2} \quad (8)$$

Because failure in the elastic regime is governed by a critical curvature, it is assumed that the curvature at failure of the simulation model is equivalent to that of the corresponding real-scale wall segment. This curvature equivalence condition is expressed by Equation (9). Under the assumption that the simulation model and the real-scale containment vessels share identical material properties and wall configuration in the elastic range, this relationship enables scaling of the failure-inducing internal blast load.

$$\varnothing_{simulation}^{fail} = \varnothing_{real}^{fail} \quad (9)$$

Accordingly, the failure-inducing internal blast load of the real-scale structure, F_{max} , is calculated using the scaling relationship given in Equation (10), which incorporates the ratios of characteristic length, sectional moment of inertia, and internal radius between the simulation model and the real-scale vessel. The calculation procedure for the sectional moment of inertia and the selected wall region is illustrated in Figure 18.

$$F_{max}^{real} = \left(\frac{L_{real}}{L_{simulation}} \right) \cdot \left(\frac{I_{real}}{I_{simulation}} \right) \cdot \left(\frac{r_{internal}^{real}}{r_{internal}^{simulation}} \right) \cdot F_{max}^{simulation} \quad (10)$$

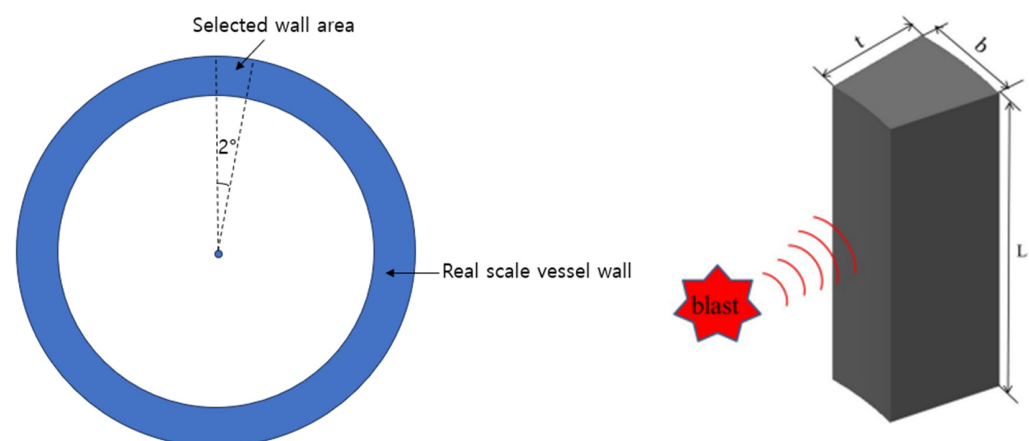


Figure 18. Moment of inertia calculation model [27,28].

The scaling procedure adopted in this study is based on stiffness and curvature consistency in the elastic range. The approach assumes geometric similarity of the selected wall segment, comparable reinforcement configuration in terms of longitudinal reinforcement ratio, and identical constitutive properties for concrete and steel within the elastic regime. Under these assumptions, the failure state is defined by reaching an equivalent critical curvature, and the failure-inducing internal blast load is scaled using the ratios of characteristic length, sectional moment of inertia, and internal radius. The applicability of this scaling is limited when strong size effects govern cracking and localization, when the response is dominated by post-peak softening, or when boundary conditions and load distribution in the real-scale vessel deviate from the simplified wall-segment representation. These limitations are explicitly acknowledged, and the derived real-scale failure-inducing loads should be interpreted as stiffness-based estimates within the scope of the stated assumptions.

Based on this scaling procedure, the failure-inducing internal blast loads of real-scale RCCV and PCCV structures are derived from the simulation results, and the calculated values are summarized in Table 13. For the open-ended condition, the failure-inducing internal blast load of the real-scale RCCV is estimated as 2.66×10^{12} N, whereas the corresponding value for the PCCV is 3.65×10^{12} N, representing an increase of approximately 37% due to prestressing. Under the enclosed condition, the failure-inducing internal blast load decreased to 1.17×10^{12} N for the RCCV and 2.86×10^{12} N for the PCCV. Despite the overall reduction associated with the enclosed boundary condition, the PCCV still exhibited a failure-inducing load more than twice that of the RCCV. These quantitative comparisons indicate that prestressing enhances the elastic stiffness of the containment wall, thereby increasing the internal blast load required to reach the failure curvature. In addition, the consistently lower F_{max} values observed under enclosed conditions for both RCCV and PCCV reflect a reduction in effective structural stiffness associated with confinement and pressure accumulation effects. Overall, the results demonstrate that the proposed stiffness-based scaling approach enables a quantitative assessment of the failure-inducing internal blast load of real-scale containment vessels based on simulation-derived structural stiffness parameters.

Table 13. Calculated failure pressure of the simulation and real scale vessels.

Type	Simulation				Real Scale Vessel			
	Open-Ended		Enclosed		Open-Ended		Enclosed	
	RCCV	PCCV	RCCV	PCCV	RCCV	PCCV	RCCV	PCCV
$I [mm^4]$	1.786×10^8	1.786×10^8	1.786×10^8	1.786×10^8	1.633×10^{11}	1.633×10^{11}	1.633×10^{11}	1.633×10^{11}
$K_{el} [N/mm]$	0.929	0.963	0.686	0.738	87.94	91.16	64.94	69.86
$E [N/mm^2]$	242.68	251.57	179.21	192.79	242.68	251.57	179.21	192.79
$F_{max} [N]$	8.568×10^6	11.750×10^6	6.552×10^6	9.230×10^6	3.61×10^{12}	2.76×10^{12}	4.95×10^{12}	3.89×10^{12}

7. Conclusions

This study presents a displacement-calibrated LS-DYNA simulation framework for evaluating the transient structural response of reinforced concrete vessels (RCCVs) and prestressed reinforced concrete vessels (PCCVs) subjected to internal blast loading. The numerical model is calibrated and validated using displacement–time histories obtained from reduced-scale open-ended experiments, and subsequently applied to investigate both open-ended and enclosed boundary configurations. Based on the simulation results, case-wise force–displacement relationships are constructed using (F_{max} , U_{max}) data, stiffness

indices are quantified, and failure-inducing internal blast loads at real scale are estimated using a stiffness-based scaling procedure.

1. The displacement-calibrated simulations successfully reproduced key temporal response characteristics observed in the experiments, including the onset of deformation, the time to reach peak displacement, and the post-peak decay trend. Because internal reflected wall pressures are not directly measurable in the experiments and free-field pressure measurements are not directly comparable to simulated internal pressures, displacement–time histories are identified as the most physically consistent validation metric for internal blast response assessment.
2. The case-wise $F_{\max}$ – $U_{\max}$ relationships enabled consistent quantification of elastic and post-transition stiffness indices for both RCCV and PCCV specimens. The stiffness transition observed in the bilinear representation provided a response-based criterion for distinguishing elastic-dominant and plastic-dominant deformation regimes under increasing internal blast demand, allowing systematic comparison of structural response characteristics between vessel types and boundary conditions.
3. Based on the stiffness-based scaling procedure, failure-inducing internal blast loads at real scale are estimated for both vessel types under open-ended and enclosed conditions. For the open-ended configuration, the real-scale failure-inducing load is estimated as 3.61×10^{12} N for the RCCV and 2.76×10^{12} N for the PCCV. For the enclosed configuration, the corresponding estimates are 4.95×10^{12} N and 3.89×10^{12} N, respectively. The enclosed configuration yielded higher failure-inducing load estimates than the open-ended configuration due to the absence of pressure relief and the accumulation of reflected pressure waves acting on the wall segment.

These failure-inducing loads should be interpreted as response-based estimates conditional on the adopted modeling assumptions, including the blast influence length, stiffness-transition definition, scaling formulation, and TNT-equivalency-based blast-loading representation. The robustness of the numerical predictions with respect to mesh resolution, concrete compressive strength, and blast influence length is demonstrated through the sensitivity analyses presented in the revised Section 3.5, which showed limited variation in peak response quantities within the examined parameter ranges. Accordingly, the results provide a rational and internally consistent basis for comparative assessment of internal blast response in vessel-type structures, while acknowledging inherent modeling uncertainty.

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Abbreviations

The following abbreviations are used in this manuscript:

RC	Reinforced Concrete
PSC	Prestressed Concrete
RCCV	Reinforced Concrete Containment Vessel
PCCV	Prestressed Concrete Containment Vessel

ANFO	Ammonium Nitrate Fuel Oil
TNT	Trinitrotoluene
F-U	Force–Displacement

Nomenclature

Symbol	Description	Unit
F	Internal blast load acting on the wall	N
F_{max}	Maximum internal blast load inducing failure	N
U	Wall displacement	mm
U_{max}	Maximum wall displacement	mm
K	Structural wall stiffness	N/mm
K_{el}	Elastic stiffness of the wall	N/mm
K_{pl}	Plastic stiffness of the wall	N/mm
P_{max}	Maximum internal blast pressure	MPa
E	Elastic modulus of concrete	N/mm^2
I	Second moment of area of wall section	mm^4
$r_{internal}$	Effective internal radius of blast load application zone	mm
M	Bending moment induced by internal blast load	$N \cdot mm$
$\varnothing$	Curvature of wall section	$\frac{1}{mm}$
$\varnothing_{fail}$	Critical curvature at failure	$\frac{1}{mm}$
ρ	Reinforcement ratio per wall face	—

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