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Numerical Study on the Behavior of Reinforced Concrete Sandwich Panels (RCSPs) under Blast Load

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Abstract

This paper presents the numerically modelled behavior of reinforced concrete sandwich panels (RCSPs) subjected to blast loads. The RCSPs are made up of expanded polystyrene (EPS) foam core having a spray-on reinforced concrete skins, like a ferrocement overlay, on both sides. The experimental study on RCSPs under blast load was previously conducted by Abbas et al. (2019). The authors investigated the physical damage intensity at different scaled distances by using qualitative fragility curves developed through visual inspection. The study was limited to the specific test plan and there is no quantitative record of the research. Experimental work on RCSPs under blasts requires extensive resources and finances. In order to overcome these limitations, the present study investigates the behavior of RCSPs subjected to blast load by utilizing Abaqus software package. The study aims at developing a numerical simulation through finite element analysis (FEA) which can be extended to other geometries and/or loads, facilitating quantitative research for the future. For simulating explosions in Abaqus, the conventional weapons effects programme (CONWEP) model has been used and the analysis is carried out by manipulating explicit dynamic solver. Damage assessment of RCSPs is done by utilizing concrete damage plasticity model. A satisfactory comparison has been drawn between the results of practical damage assessment and that obtained by the FEA. The numerical study is further extended to an RCSPs' structure which showed satisfactory strength against blast load. It is recommended that RCSPs can be used in buildings as infill walls which provide significant capabilities to absorb and dissipate energy produced by explosions.

Keywords: Reinforced Concrete Sandwich Panel (RCSP), Blast, FEM.

1. Introduction

Accidental explosions and terrorist attacks as a life-threatening issue for community are also related to damaging infrastructure thus, causing huge economic losses. The resistance of military as well as civilian structures against terrorist attacks and accidental explosions is an important facet of structural engineering in the modern times. Casualties in large numbers may be resulted when vulnerable structures are faced with blasts, sometimes leading to total collapse. Events such as the New York World Trade Centre (1993), the Oklahoma City Federal Building (1995), the Saudi

Arabian Khobar Tower (1996), the US Embassy in Nairobi, Tanzania (1998), 2002 attack in Indonesia, Canal Hotel Iraq attack (2003), bomb attacks on trains in Spain (2004) and India (2006), blasts by a gunpowder production facility (2008) in Turkey, Oslo (2011) bombing and Peshawar Church (2013) and Army Public School (2014) attacks in Peshawar, Pakistan, highlighted the requirement to investigate the structural response and design against blast load.

A number of approaches have been suggested by several researchers to ensure the safety and serviceability of structures against explosions. Such as designing new structures or altering the old ones or using combination of different materials which have ability to absorb and dissipate energy imparted by blasts. The reinforced concrete structures are incapable to absorb energy produced by blast which led to disintegration of the materials (Javed 2009 and Xueying Wei, 2010). Over the last few decades, researchers recommended that sandwich composite structures have capability to absorb and dissipate energy. With the fast development of modern military technology, the sandwich structures with cellular solid cores, such as metallic foams and honeycomb structures, have shown superior weight specific stiffness and strength properties compared to their monolithic counterparts in blast resistant structural applications. Their cellular microstructure allows them to undergo large deformation at nearly constant nominal stress and thus absorb more energy (Zhu & Lu, 2022). The metallic sandwich soft cores reduce the momentum transferred, thus providing better mitigation for blast loading (Wei et al., 2007). Sandwich structures have various energy dissipation mechanisms, such as bending and stretching of the face sheet as well as compression and shear of the core (Cui et al., 2012). Composite sandwich structure shows least damages when subjected to blast loading (Dear et al., 2017). Santosa et al., 2017 studied response analysis of blast impact loading of metal-foam sandwich panels which showed high structural efficiency.

A structural sandwich, generally, is a three-layered panel having a soft core (polyurethane foam, metallic honeycomb, balsa wood etc.) sandwiched between two facings of relatively stiffer material (steel, aluminum, concrete etc.). The use of pre-cast sandwich panels is a concept older than half a century (Losch et al., 1997), started in North America, the base of interest for which was to build high rise structures. Reinforced Concrete Sandwich Panel (RCSP) is also a structural composite whose building system has the required capabilities. An RCSP typically consists of expanded polystyrene (EPS) foam core surrounded with concrete. The concrete on both sides is reinforced with steel mesh. Abbas et al. (2019) examined the response of RCSPs to blasts and recommended their use at critical locations as a boundary wall. Impact penetration and hole execution of square sandwich panels with EPS core have been evaluated by Caliskan & Apalak (2020). It was concluded that the energy absorption of the panels got elevated by increasing the core thickness while the penetration resistance was mainly offered by the leaves.

Lightweight structures are entirely attractive in blast protection (Nwankwo, 2014) and RCSPs can be a good choice to implant the desired properties in structures vulnerable to explosions. But it needs development of proper design codes. Testing with explosions has many limitations (Draganić et al., 2018). The process includes a large number of experiments consisting of, but not limited to, making of prototypes, application of blast loads, dynamic analyses, trials, and recording of the data. The conduct of blast load experiments are very costly and carry a lot of safety concerns (Zakrisson et al., 2010). An alternative to experimental studies is to utilize the previous small-scale research data for nonlinear dynamic analysis through finite element modeling (FEM) tools.

Numerical studies on sandwich panels are very limited and most of the literature was found focused on static analysis of composites. A comprehensive study on the behavior of Precast

Concrete Sandwich Panels (PCSPs) under different static conditions and corresponding Finite Element Analyses (FEA) can be found in (Benayoune et al., 2006, 2007, 2008), where the FEA and practical results were in close approximation. Zakrisson (2010) investigated the metal plate behavior subjected to blast effects through numerical techniques. Gara et al. (2012) numerically simulated sandwich walls under static compression. Pozorska & Pozorski, (2018) studied the effect of core non-continuity and progressive damage and developed different numerical models for structural sandwiches and explored their stress redistribution and core compression under static tests. Thiagarajan & Munusamy, 2020 performed explicit FEA for structural composites with the help of LS-DYNA. Very limited studies are available in literature on the behavior of RCSPs under blast load via numerical techniques. The present study focuses on the development of numerical model for RCSPs followed by its behavior under blast load using Abaqus explicit dynamic solver. There are many different numerical schemes to simulate explosions but the most common (Castellano et al., 1982) are JWL (Jones-Wilkins-Lee), TM 5-1300, and CONWEP. A comparison of these three can be found in Masi et al., (2017). The JWL and CONWEP concepts are implemented in commercial programming for FEA. AUTODYN hydrocode (Chapman et al., 1995) uses the JWL model while ABAQUS uses CONWEP model. CONWEP is widely used for blast simulation (Prueter, 2014; Temsah et al., 2018). It takes the negative phase also into account and it can model hemispherical blasts (Masi et al., 2017). In the present study, blasts are simulated through CONWEP model using experimental data from Abbas et al. (2019). The damage observed in the numerical model RCSPs is in close approximation with that recorded by the practical experiment. The numerical model has been further extended to a 3-dimensional frame structure of plain cement concrete having RCSPs as slab and walls.

2. Experimental Setup

2.1. Reinforced concrete sandwich panel (RCSP)

An RCSP has an EPS foam core wrapped on both sides by spray-on reinforced concrete skins. The RCSP construction is a more than 50-year-old technique that has received little attention until lately as earthquake, energy, and blast resistant structural requirements have arisen as one of the most basic criteria for modern buildings. Adil's (2010) research has demonstrated that RCSP performs well under a variety of loads, including shear, axial, and flexure, and that it has the potential to be employed as a shock-resistant structural member.

2.2. Geometric Details

The prototype consists of four 3.04 m high RCSPs arranged in a rectangular fashion as shown in Figure 1. Figure 2 shows dimensions of a typical RCSP wall. The assembly has two pairs of different wall thicknesses whose details are given in Table 1. The first pair of walls is made up of 38 mm reinforced concrete on both sides and a sandwiched EPS foam of 101.6 mm thickness, i.e. P17 and P27. The second pair of walls is made up of 38 mm reinforced concrete on both sides and sandwiched EPS foam with thickness of 50.8 mm (P35 & P45).

There is no connection between any two of the panels. Foundation details indicated fixed support between each panel and the ground as a result, each panel is assumed functioning cantilevered.

Figure 1: RCSP Walls Layout (Abbas et al., 2019)

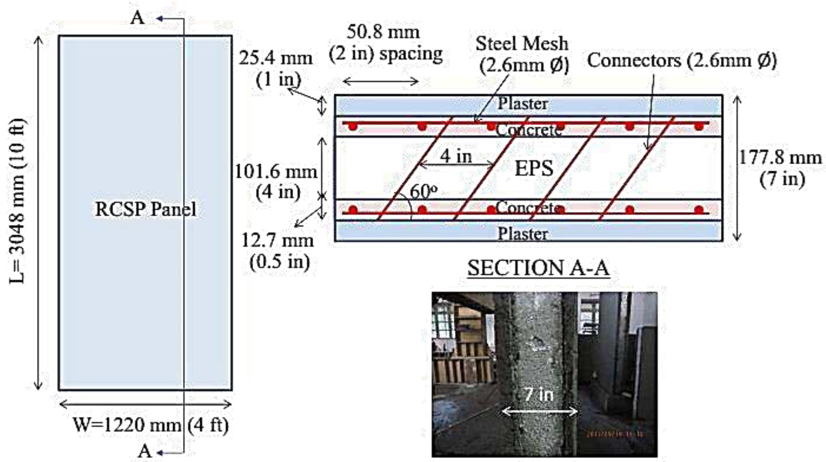

Figure 2: RCSP Wall Details (Abbas et al., 2019)

Table 1: RCSPs Dimensional Details (Abbas et al., 2019)

Panel ID	Plaster Layer Top & Bottom (mm)	Concrete Layer Top & Bottom (mm)	Polystyrene Layer Middle (mm)	Overall Thickness (mm)
P17	25.4	50.8	101.6	177.8
P27	25.4	50.8	101.6	177.8
P35	25.4	50.8	50.8	127
P45	25.4	50.8	50.8	127

2.2. Material Properties

Concrete (M15) is used in RCSPs having properties as given in Table 2 as per ASTM-C39/C39M-2003. It was reinforced with a steel mesh which, according to ASTM 370-03, had the properties of Table 3. Table 4 enlists the mechanical properties for EPS.

Table 2: Material Properties of Concrete base on Cylindrical Samples

Sample	Load (kN)	Surface Area (mm^2)	Strength (MPa)	Average Strength (MPa)
1	33.12	2026.82	16.34	
2	24.21	2026.82	11.94	15.00
3	33.91	2026.82	16.34	

Table 3: Material Properties of Steel

Sample	Nominal Diameter (mm)	Yield Strength (MPa)	Ultimate Strength (MPa)	Percent Elongation	Effective Diameter (mm)
1	2	184	272	12.5	2.794
2	2	200	232	15.6	2.794
3	2	192	264	15.6	2.794
Average	2	192	264.48	14.6	2.794

Table 4: Material Properties of EPS

Property	Value (MPa)
Elastic Modulus	0.35772
Shear Modulus	0.17886
Tensile Strength	0.01595
Compressive Strength	0.00736
Flexural Strength	0.01371
Shear Strength	0.01647

2.3. Blast Tests Details

Explosive consumed in the experiment was trinitro toluene from WA Box, a product of WA Nobel PVT. Limited. It has a density of 1.4-1.45g/cc, gas volume of 860 m^3/kg, velocity of +5000m/s and TNT equivalence of 1.1. Several blasts of different weights were performed at various locations on the RCSPs as shown in Figure 3 and Table 2.

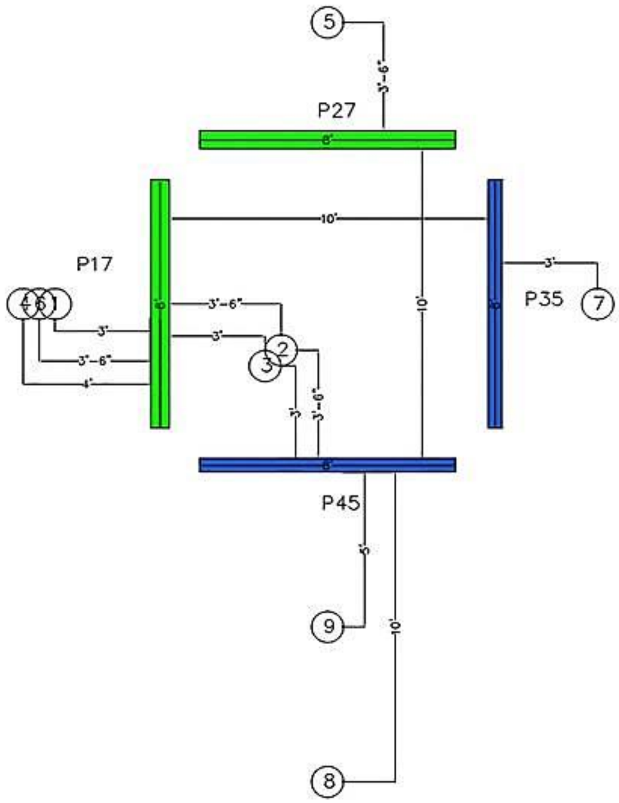

Figure 3: Progressive Blast Load Locations (Abbas et al., 2019)

Table 5: Details of Blasts (Abbas et al., 2019)

Blast No.	Quantity (kg)	Distance (m)	Location (Relative to Enclosure)
1	0.5	0.91 from thick panel	Outside
2	1	1.1 from thick & thin panel	Inside
3	1.5	0.91 from thick & thin panel	Inside
4	2	1.2 from thick panel	Outside
5	2.5	1.1 from thick panel	Outside
6	3	1.1 from thick panel	Outside
7	3.5	0.91 from thin panel	Outside
8	4	3 from thin panel	Outside
9	5	1.1 from thin panel	Outside
10	2	In contact with thick panel	Outside

3. Finite Element Modeling with Abaqus

Abaqus is a nonlinear FEA tool with a wide range of applications. It has a large library of material models, elements, and procedures, and it can model almost any geometry. It offers models which can be used to establish elastic and plastic properties of metals, foams, and concrete.

3.1. RCSPs' Numerical Model Development

For modelling EPS foam and concrete leaves, solid continuum elements can be used thus their geometry was modelled by utilizing C3D8R, which is an 8-node linear brick element. The friction surface contact approach was used to attach these elements to the continuum shell elements. A perfect plasticity method was established for the steel material definition and it was modelled through T3D2 geometry. To represent the relation/connection between the concrete and steel, the embedded element technique was used where the reinforcing components "the guests" were embedded in the concrete elements "the host." The node of the guest elements eliminates its translational degrees of freedom, and it becomes an "embedded node". The embedded node's translational degrees of freedom are limited (*Abaqus/CAE User's Guide*, 2002) by interpolated values of the host element's corresponding degrees of freedom. With a diameter of 2.5 mm, the longitudinal and transverse reinforcements were modelled with a center-to-center spacing of 50.8 mm. A mesh size of 40mm was used.

The nonlinear material behavior was described by a perfectly plastic model using the data given in Table 6. The EPS foam was modelled as an elastic material. Poisson's ratio for modelling EPS was taken zero. Tension stiffening was considered in material definition to simulate load transfer, across cracks in concrete, through steel. For the start of yielding, the tension stiffening parameters were defined as follows:

$$\frac{\sigma_{remaining}}{\sigma_{cracking}} = 1 \tag{1}$$

And,

$$\left|\varepsilon_{direct} - \varepsilon_{cracking}\right| = 0 \tag{2}$$

While for full plasticization, the tension stiffening parameters were:

$$\frac{\sigma_{remaining}}{\sigma_{cracking}} = 0 \tag{3}$$

And,

$$\left|\varepsilon_{direct} - \varepsilon_{cracking}\right| = 0.0005 \tag{4}$$

The following failure ratios from the Abaqus (2002) manual define the failure criteria for concrete:

$$\frac{Biaxial\ compression\ failure\ stress}{Uniaxial\ compression\ failure\ stress} = 1.16 \tag{5}$$

And,

$$\frac{Biaxial\ tension\ failure\ stress}{Uniaxial\ tension\ failure\ stress} = 0.0836 \tag{6}$$

Table 6: Material Inputs for the RCSP Model

Material Property	Concrete	Steel	EPS
Modulus of Elasticity (N/mm^2)	15427.2973	2.04E+05	7.0809
Ultimate Strength (MPa)	52	700	-
Yield stress (MPa)	-	450	-
Yield stress (MPa)	-	0.002	-
Density (ton/mm^3)	2.30E-09	7.83E-09	2E-11
Poisson's ratio	0.12	0.3	0

There are two models implemented in Abaqus for analyzing the concrete cracking behavior in order to conduct damage investigations i.e. Smeared Cracking (SC) model and Concrete Damage Plasticity (CDP) model. SC model uses crack detection techniques where the compression failure is described by failure ratios. These failure ratios represent the geometry of the failure surface of the concrete model. While CDP is a continuum, plasticity-based model. CDP is the most widely used model for plain as well as reinforced concrete damage studies. Theoretically conceptualized by Lubliner et al. and Lee & Fenves (Szczecina & Winnicki, 2017), CDP is based on the Drucker-Prager plasticity model (*Abaqus/CAE User's Guide*, 2002), which accounts for plastic stresses and cracking in a wide range of problems under monotonic, cyclic, and dynamic loadings. CDP model takes the effects of bond-slip and dowel action at the rebar-concrete contact under consideration but it needs some material constants as inputs. It is an opened scientific issue (Szczecina & Winnicki, 2017) to choose suitable values for these constants. As demonstrated in Figure 4, this model allows for concrete interaction with reinforcement and models concrete failure in tension and compression using a non-associated flow law.

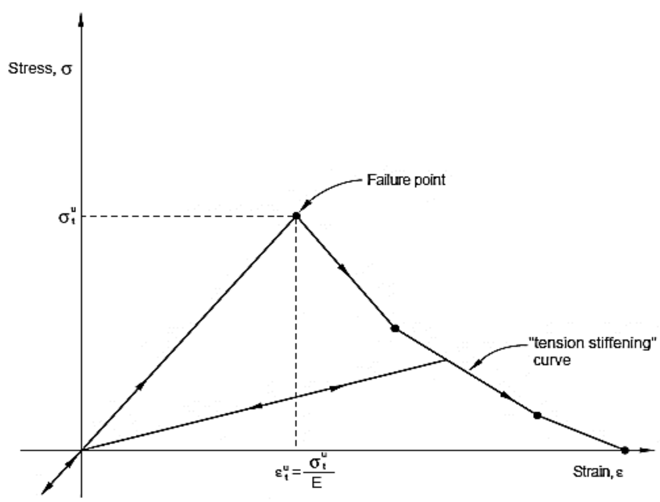

Figure 4: Concrete Tension Stiffening (Hafezolghorani et al., 2017)

The compression hardening of a material is accounted for by the dilation angle (Ψ), which ranges from 0° to 56°. The dilation angle of concrete has a wide range of values in the literature; the Abaqus Verification manual and the Abaqus Examples manual use 15° and 36.36° respectively.

The rate at which the Drucker-Prager function approaches the asymptote is known as the flow potential eccentricity (ε). When eccentricity approaches zero, the plastic flow tends to a straight line. The eccentricity in the FE calculations was set to 0.1. This value is chosen to achieve a soft curvature of the potential flow and to produce almost the same dilation angles for a wide range of confining pressure values. The ratio of the second stress invariant on the tensile meridian (T.M. in Figure 5) to the second stress invariant on the compressive meridian is termed as "K_c". The crack detection surface for different values of the K_c parameter is shown in Figure 6. The viscosity parameter (μ) is used to tackle convergence issues caused by elasticity degradation and softening behavior. Values used for these parameters are listed in Table 7.

Several analyses, run with different values of tension stiffening profiles, failure ratios and dilation angles, were used to compare the SC and CDP models. The CDP model was found to provide more consistent results for different mesh sizes and to better predict the panel's post-failure behavior as was the case with Adil (2010). The damage plasticity model was used for further analysis. Nonlinear properties of concrete defined for the CDP are illustrated in Figures 6 and 7.

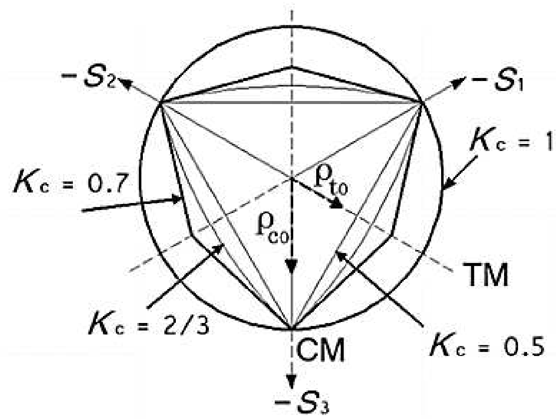

Figure 5: Druker-Prager Yield Surface Showing the Kc Parameter in Deviatoric Stress Plane (Alfarah et al., 2017)

Table 7: Input Parameters for Concrete Damage Plasticity Model

Parameter	Value
Ψ	35
ε	0.1
K_c	0.667
μ	0.007985

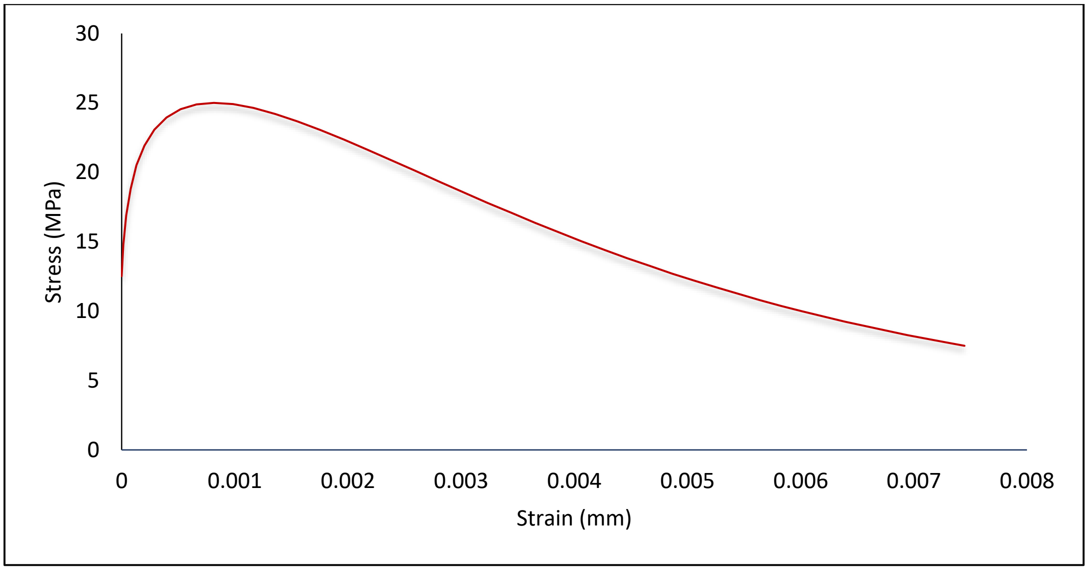

Figure 6: Concrete Material Definition for CDPM

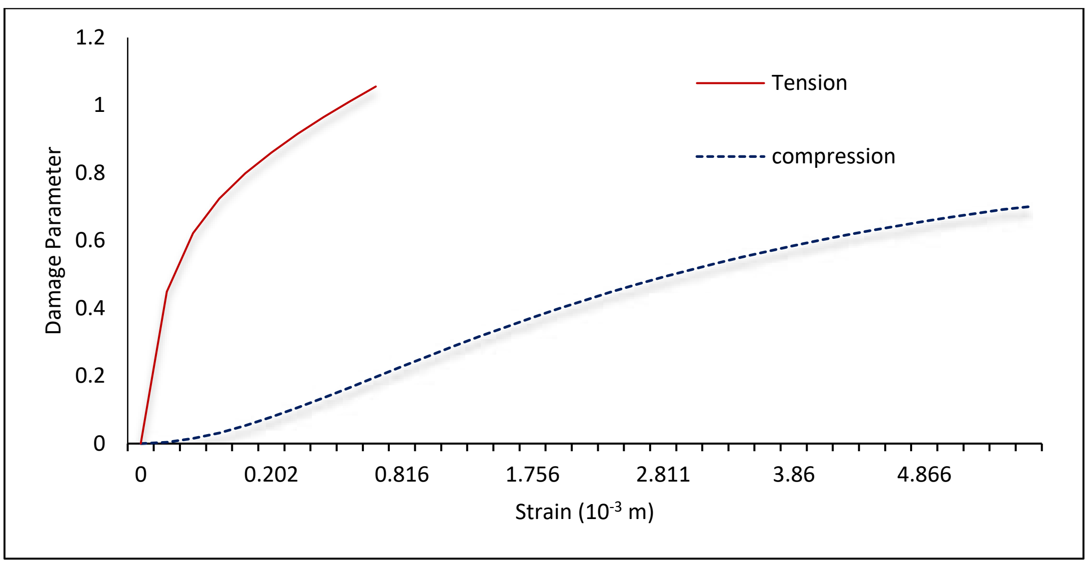

Figure 7: Concrete Damage Ratio for CDPM

3.3. Modeling Blast Action

The loading consequences of an explosion in the air can be defined using empirical data provided by the CONWEP model in conjunction with the incident wave loading definition for spherical incident waves (air blast) or hemispherical incident waves (surface blast). A study conducted by the United States Army Corps of Engineers (USACE, 1986) (Mendonça et al., 2021), CONWEP model provides an effective and convenient functionality to scaled distance and the amount of material exploded, the CONWEP model provides empirical data such as maximum overpressure, arrival time, positive phase duration and exponential decay coefficient for both incident and reflected pressures. The function generates non-uniform loads on the structure's exposed surface and accounts for surface reflection before applying total blast pressure, which is calculated using the equation:

$$P(t) = P_r cos^2\theta + P_i(1 + cos^2\theta - 2cos\theta) \quad (13)$$

Where, P_r is the reflected pressure, θ is the angle of incidence defined by the tangent to the wave front and the target's surface and P_i is the incident pressure. Input data for blast simulation is given in Table 8 where the scaled distance has been calculated using equation (3).

Table 8: Input Data for CONWEP model

Blast No.	Charge Weight (kg)	Stand-off Distance (m)	Scaled Distance (m/kg$^{1/3}$)
1	0.5	0.91	1.15
2	1	1.1	1.1
3	1.5	0.91	0.79
4	2	1.2	0.95
5	2.5	1.1	0.81
6	3	0.91	0.63
7	3.5	0.91	0.60
8	4	3.0	1.89
9	5	1.5	0.88
10	2	0	0

4 Results and Discussion

Two models were prepared; one was 7 in thick and the other was having a total thickness of 5 in. The dimensional details were those given in Table 1. Each of the two models was replicated once to get a copy thus making a total of four samples. Only those blasts were applied on a panel which had practically direct and largest impact on the panel.

4.1. Panel P17

The P17 panel was subjected to 1st (0.5 kg), 2nd (1 kg), 4th (2 kg), 6th (3 kg) and 10th (2 kg) blasts progressively. Practically, the first blast did not affect the panel and there were no signs of damage but the numerical model showed some negligible damage and maximum Mises stress of about 1.74 MPa. This is demonstrated in Figure 11. After the first blast, a 1 kg second blast was launched inside the enclosure, the pressure reflected from the entire panel and needed a passage through which it could proceed into the low-pressure zone. The concrete peeling from the inside borders of the corner at the joint and mid-center portion of the P17 panel is a sign of the pressure confinement because the maximum stress was 1.48 MPa, lower than the previous. Moreover, revealed were the minor cracks categorized as compression cracks and slight scabbing of concrete as shown in Figure 12. The 4th blast of 2 kg caused minor detaching of plaster and tiny cracks at the joint between the foundation and panel. In Figure 13, it can be observed that slightly more damage was noted in the numerical model, where the grey area shows the damaged elements, of P17. The affected area was the same in both the experimental and numerical models and the maximum stress was 2.36 MPa. The 6th blast had a charge weight of 3 kg which caused complete falling off of plaster and crushing at the lower portion, although, plaster remained in contact at the upper part of the panel (Figure 14). The foundation dowel steel got exposed entirely which weakened the foundation connection. The failure that occurred was categorized as the flexural crushing of P17 concrete due to a maximum stress of 2.78 MPa.

The tenth was a contact blast in which the explosive material of 2 kg was placed at the center of the fallen P17 panel and applied a maximum stress of nearly 3 MPa. It was detected that only the portion where the blast material was placed got effected. The blast made a cavity-like-hole in the panel causing severe damage and exposure of reinforcement mesh. The results from the numerical model also simulated similar visualization as shown in Figure 15.

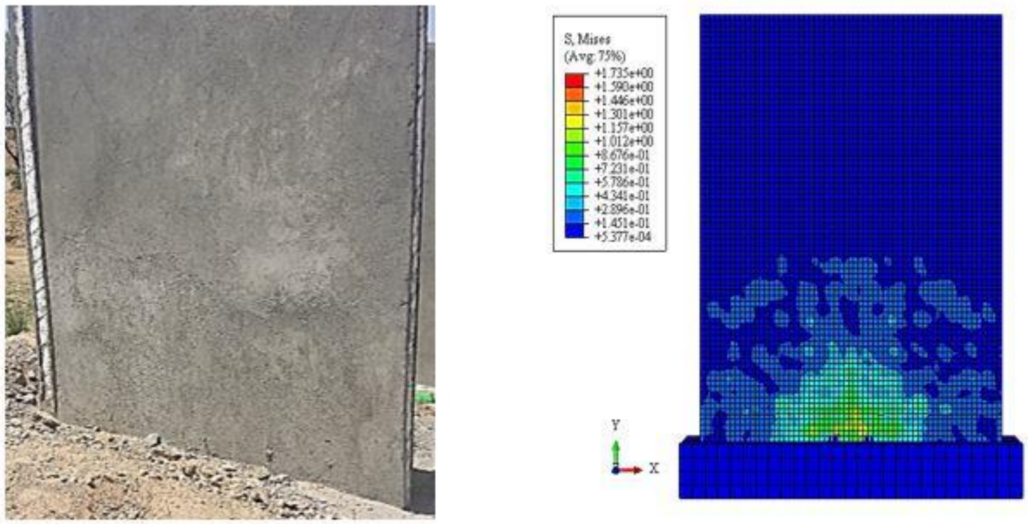

Figure 8: Observation of P17 after 1st Blast

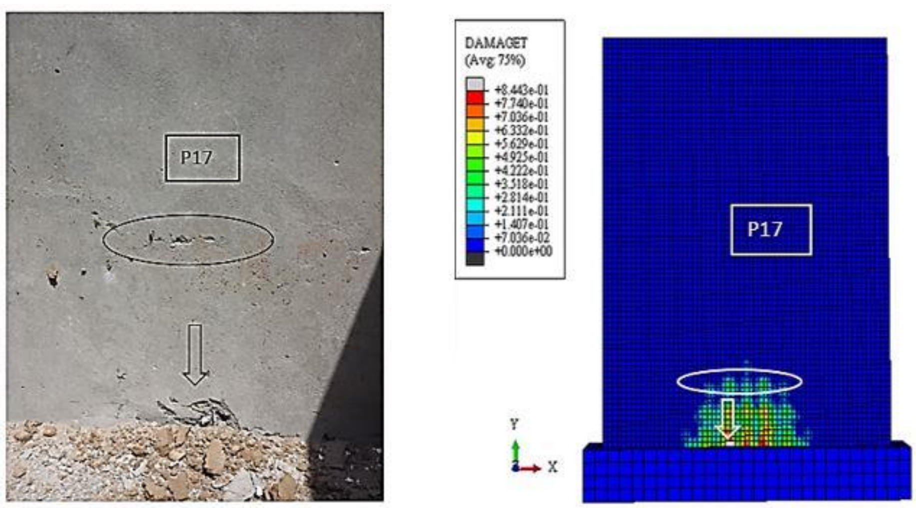

Figure 9: Observation of P17 after 2nd Blast

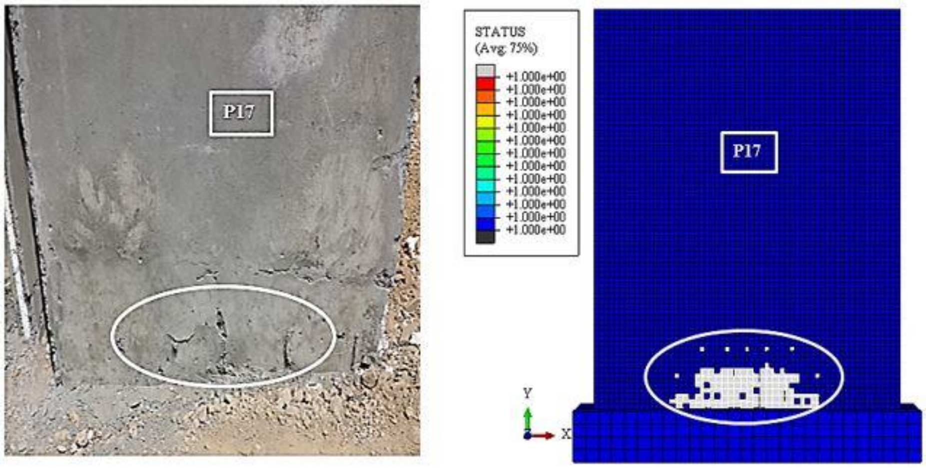

Figure 10: Observation of P17 after 4th Blast

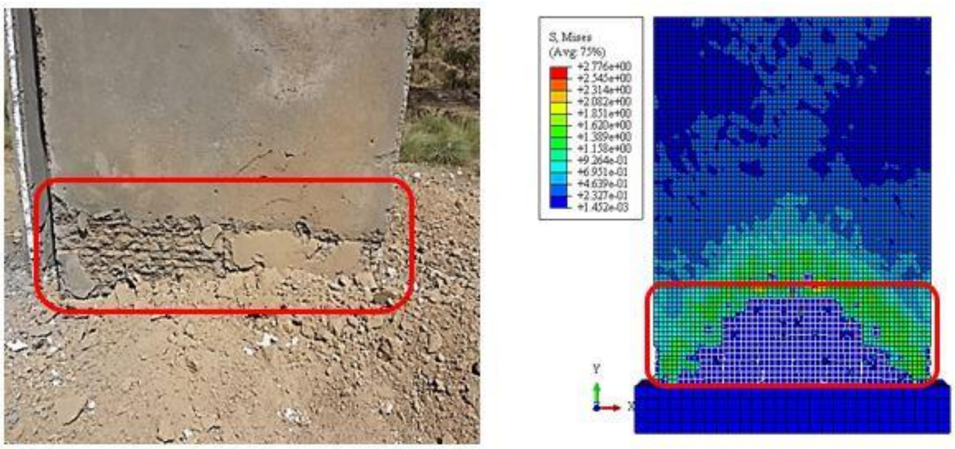

Figure 11: Observation of P17 after 6th Blast

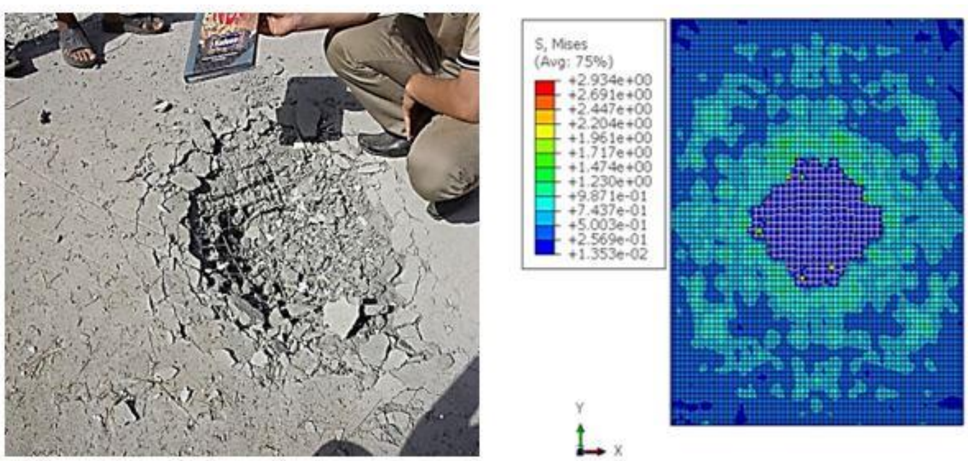

Figure 12: Observation of P17 after 10th Blast

The physical and numerical damage assessments, done cumulatively, are compared in Figure 16. From the figure, it is worthy to note that the numerical model slightly underestimates RCSPs' opposition to explosions as compared to the experimental specimen.

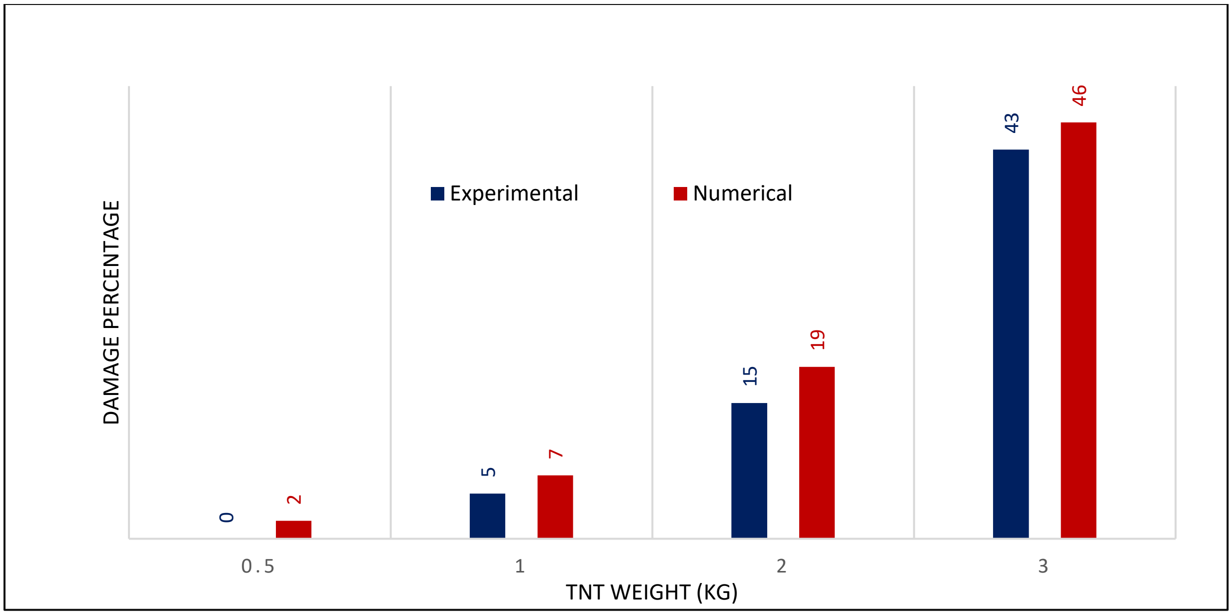

Figure 13: Comparative Damage Assessment of Panel P17

4.2. Panel P27

Panel P27 was exposed to the 5th blast having a charge weight of 2.5 kg. The blast caused severe peeling off of plaster and concrete at the base, up to about 1/5 of the height from the bottom and the steel mesh was exposed in the affected area. Numerically depicted damages also exhibited the same nature as shown in Figure 17 where the maximum stress recorded was approximately 3.88 MPa.

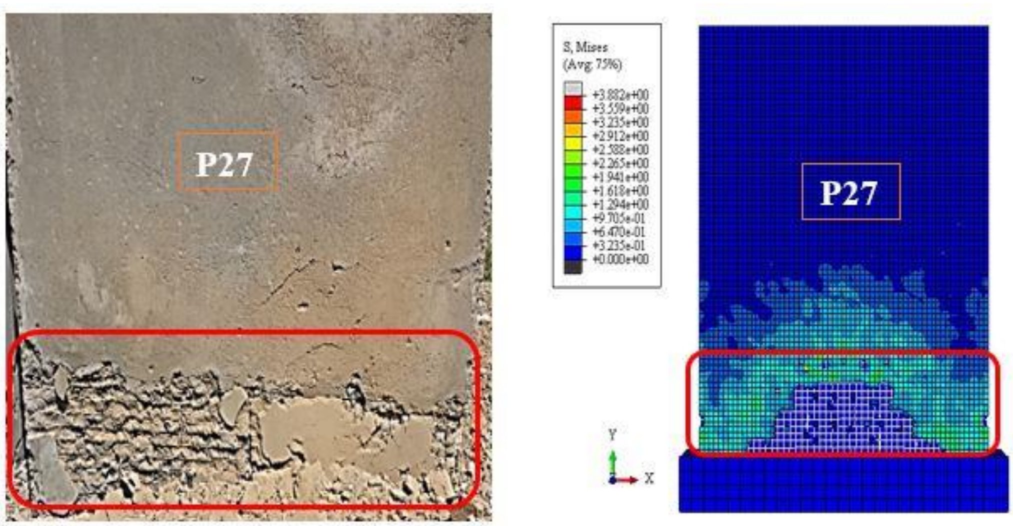

Figure 14: Observation of P27 after 5th Blast

4.3. Panel P35

P35 panel was subjugated by the 7th (3.5 kg) blast. It was the only blast loaded to P35. The larger TNT weight, the smaller stand-off distance and of course the smaller thickness of the wall's core were sufficient reasons to fail the panel leading it to get completely collapsed from a maximum stress of 10.56 MPa. It was analyzed that the failure occurred due to the pulling out of dowels from the panel at the foundation level. Also, it should be kept in mind that the blasts were progressive and the bursts previously done had, no doubt, already weakened the joint between the panel and foundation. Alike were the consequences, shown in Figure 18, procured by simulation.

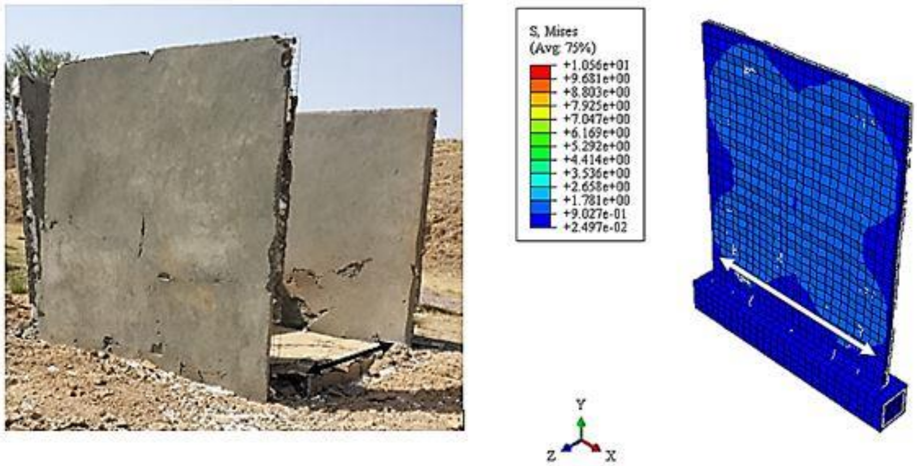

Figure 15: Failure of P35 after 7th Blast

4.4. Panel P45

P45 panel was treated with blasts 2 (1 kg), 3 (1.5 kg), 8 (4 kg) and 9 (5 kg). As the 2nd blast was launched inside the enclosure, it was nearer to P17 and P45 as compared to the other two panels. Panel P45 had greater scabbing at the foundation connection and in the middle of the panel than panel P17. The reason possibly behind this is the smaller thickness of EPS core in P45. P45 had a smaller energy dissipation capacity but it stood, still (See Figure 19). The third (1.5 kg) blast was also detonated inside the room, applying 2.34 MPa maximum stress. After the third explosion, there was severe peeling-off of the plaster and concrete at the foundation joint of P45. The numerical observations showed almost the same damage as the experimental visuals that are depicted in Figure 20.

Eighth blast caused plaster from the lower portion to get off completely from the panel and the connection between the panel and its foundation almost detached, only a little portion remained intact due to which the panel was free-standing. The numerical assessment in Figure 21 reveals the same details showing maximum stress of 2.93 MPa. The 9th blast of 5 kg was the heaviest among all the blasts occurred causing a maximum stress of 9.64MPa. After the occurrence of this blast at a distance of 5 ft from P45, the whole assembly collapsed. Complete failure of P45 has also been accurately predicted by the numerical model and shown in Figure 22.

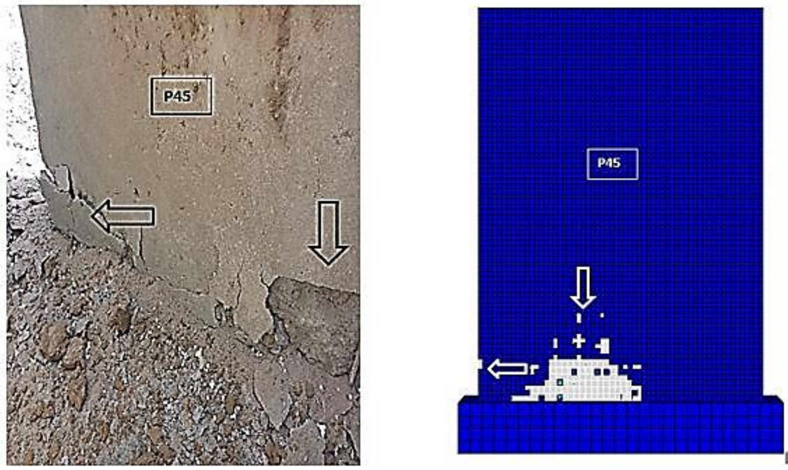

Figure 16: Observation of P45 after 2nd Blast

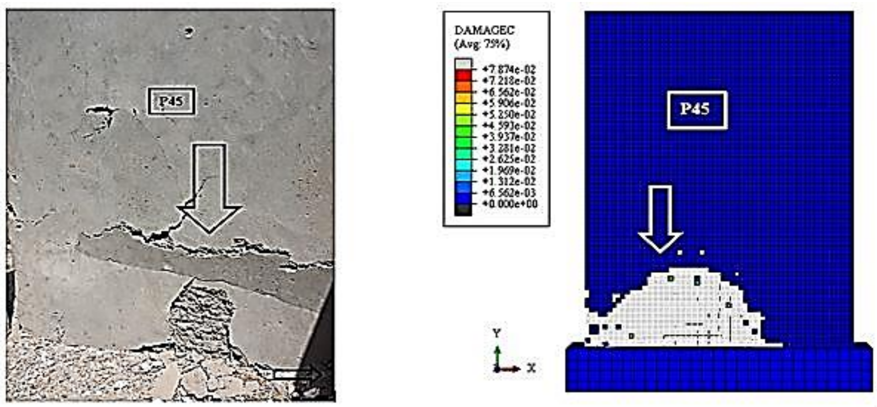

Figure 17: Observation of P45 after 3rd Blast

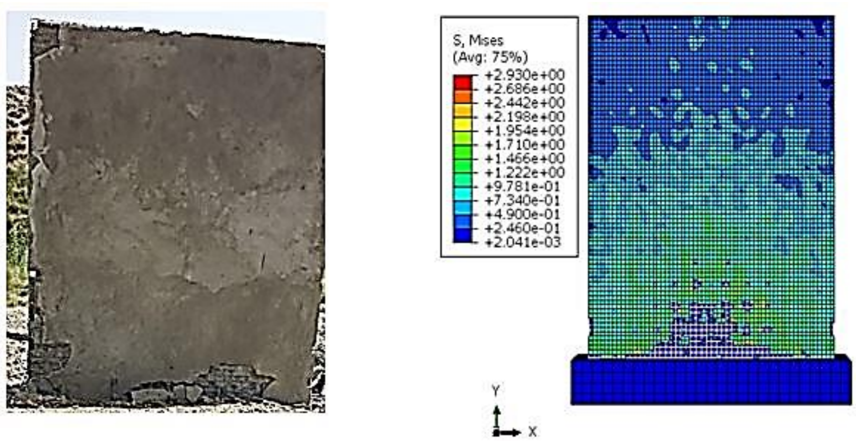

Figure 18: Observation of P45 after 8th Blast

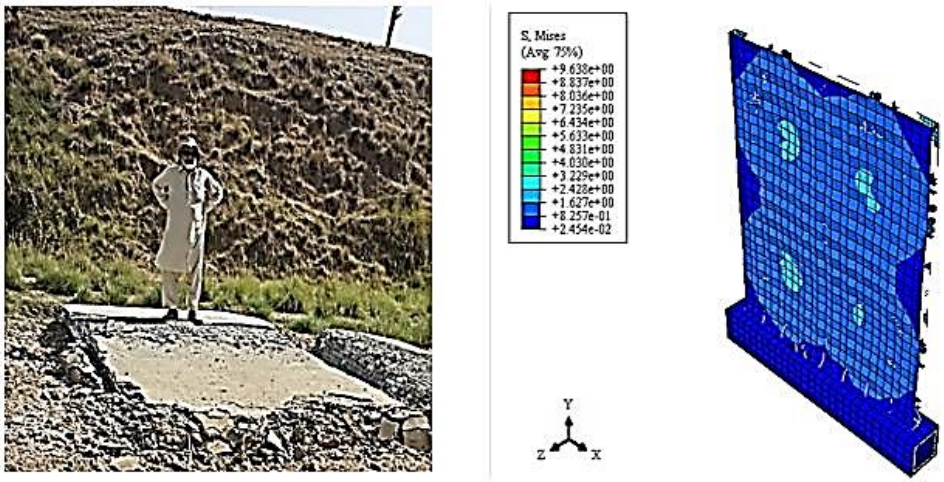

Figure 19: Observation of P45 after 9th Blast

The comparison of damage physically observed and that numerically predicted is done through the bar chart illustration in Figure 23. It is worthy to mention that for the 3rd blast, which was 2nd for P45, the simulation gave a damage lesser by 2%, contrary to all the other cases where the damage predicted by the numerical model had always been greater than actually observed.

Difference in percent damage observed in practical test and numerical modeling for all the panels is summarized in Table 9.

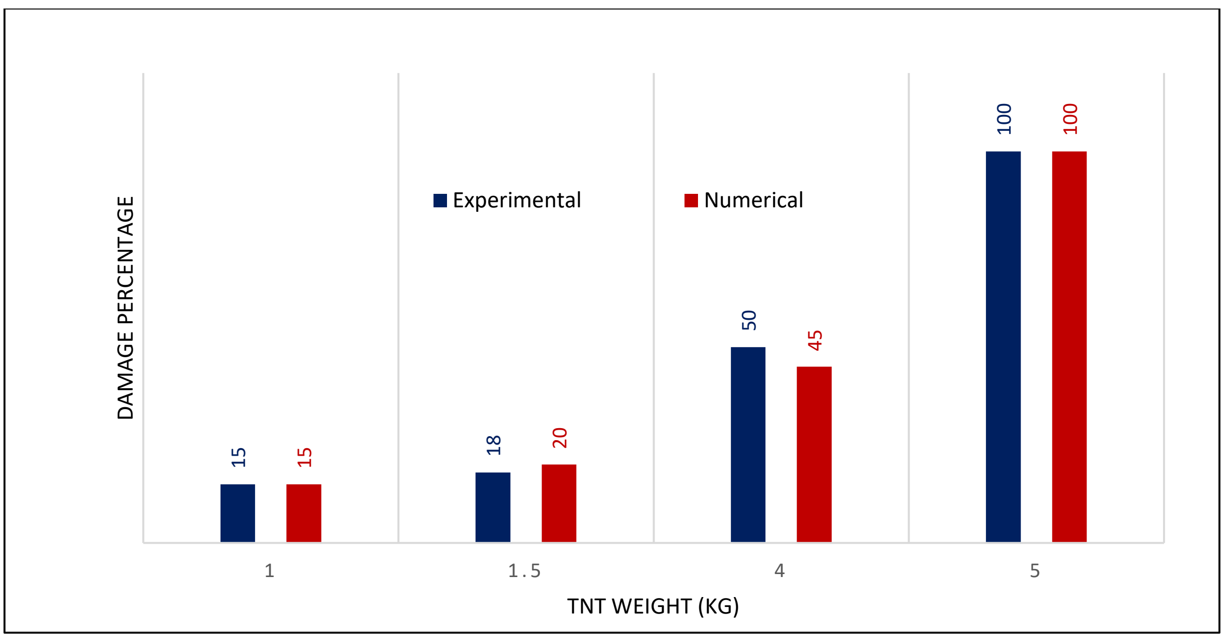

Figure 20: Comparative Damage Assessment of Panel P45

Table 9: Difference in Practical and Numerical Percent Damages

Blast No.	Percent Damage Difference			
	P17	P27	P35	P45
1	2	-	-	-
2	2	-	-	0
3	-	-	-	2
4	4	-	-	-
5	-	0	-	-
6	3	-	-	-
7	-	-	0	-
8	-	-	-	5
9	-	-	-	0
10	0	-	-	-
Mean	2.75	0	0	1.75

4. Numerical Analysis of RCSPs Structure

The experimental work to check the behavior of RCSPs under blast load was only limited to the cantilever panels. It is necessary to evaluate the response of such panels within the buildings by analyzing the behavior of a single panel acting in combination as well as the behavior of their combination. This will clear the idea of using RCSPs as resistive structures against blast load. To extend the study of RCSPs behavior under blast load, a room of 18 ft × 18 ft was modelled and the damage of RCSPs was assessed under larger blast charges. The walls and slab of the room were made up of RCSPs and the structure was fixed supported on the foundation through dowel connection.

The structure was subjected to three blasts progressively. For the first blast of 10 kg TNT, the point of detonation was defined at 10ft from the front wall outside the structure. The visuals (Figure 26) from the results showed that the confined RCSP wall had a greater capacity to resist the blast shock.

The second blast was having a 50 kg weight applied on the model from 10 feet distance at its front. The room was still stable and didn't undergo any serious damage although some slight spalling was noticed in the lower portion of the wall at front, as shown in Figure 27.

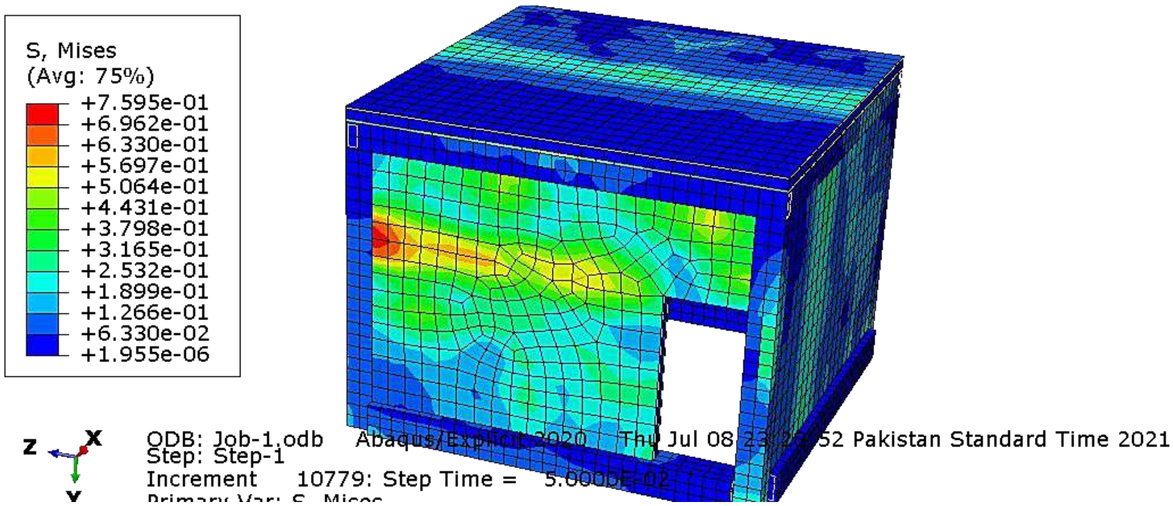

Figure 21: Numerical visuals of stresses (10kg blast, Outside)

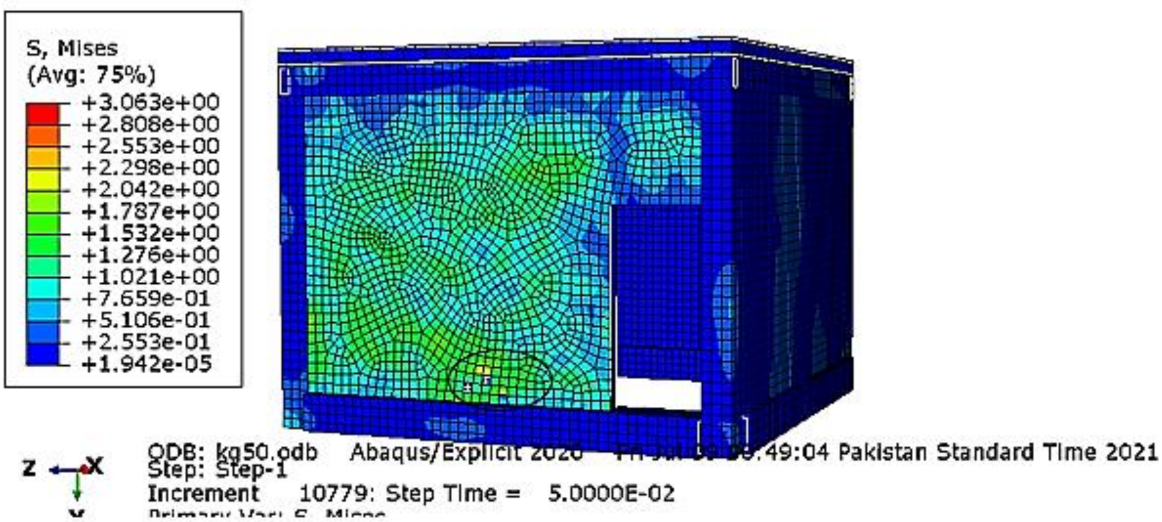

Figure 22: Damage assessment (50kg blast, outside)

To study the response of RCSPs in near field explosions, a blast of 20 kg TNT was detonated inside the room. The results from the numerical simulations were still satisfactory and the panel didn't show total distortion, however the slab panel was under larger effects of the shock wave and showed greater deformation as compared to wall panels. This is because the wall panels were much confined by columns and beams as compared to the slab panel. The beams and columns were badly affected by the shock and the serviceability of the beam and columns was diminished. This reveals that the response of RCSP is attractive as compared to the ordinary concrete structures due to its ability of dissipating energy during extreme loading conditions. Figure 28 evinces the damage assessment.

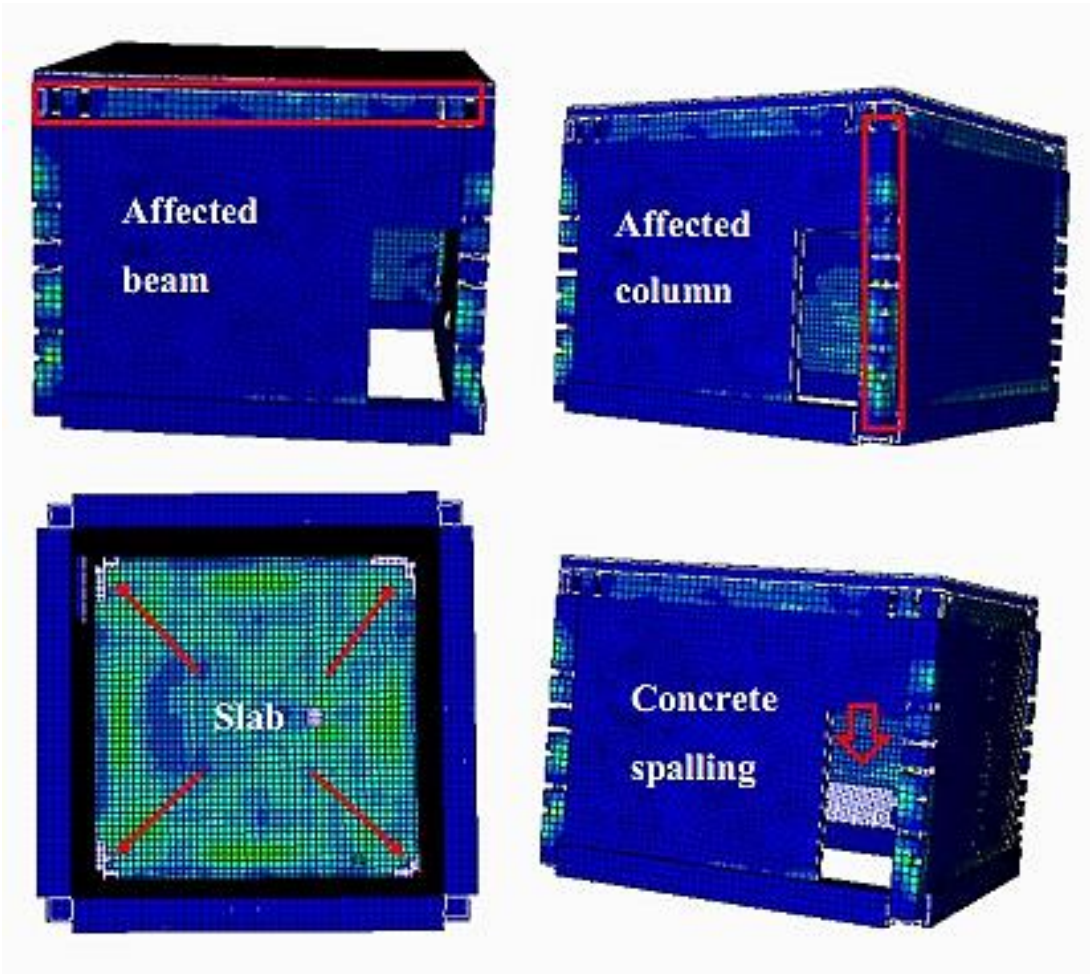

Figure 23: Damage assessment (20 kg blast, Inside)

5. Conclusion

In this study, the behavior of RCSPs subjected to explosions has been simulated with finite element modeling technique. A material model was developed for RCSPs and was then loaded with blasts progressively. Explicit/Dynamic analysis was done for blasts simulated with CONWEP model. Damage assessment of RCSPs was carried out by utilizing concrete damage plasticity model and the numerical visuals were compared with physically observed behavior. The mean difference in percent damage was found to be 1.12%. The assembly was extended to predict the response of RCSPs structure under blasts. The following conclusions are drawn from the study:

1) RCSPs are capable of absorbing and dissipating impact and can be used at locations critical to intentional and/or accidental bursts. No fragmentation has been observed, thus, using RCSPs at these locations will reduce the chances of nonfatal injuries caused by flying debris.
2) Increasing thickness of EPS core in RCSPs increases their potential damage caused by blasts. Also, RCSPs in combination display more resistance against dynamic loads as compared to RCSPs standing alone.

3) Inside blasts cause larger damage as compared to outside blasts because in case of inside blasts, the whole hemispherical explosion is enforced on the structure in all directions, unlike outside ones, where the structure gets only a part of the hemisphere.
4) Slabs are more vulnerable to damage than walls specially when blast occurs inside the enclosure. It can be justified by the fact that the walls only get hemisphere of the shock wave while the slab gets a total circular shock. Moreover, waves reflected from walls also try rising up, thus, ultimately reaching the slab.
5) Lower parts, approximately 1/4th of height, of walls are more exposed as long as explosion occurs at ground level, leading primarily to dowel failure.

6. Research Recommendations

1) The Abaqus numerical model for RCSPs presented herein can be compared with models offered by other commercial nonlinear FEM software packages and/or the model can be subjected to blasts simulated through concepts other than CONWEP for example Euler, JWL etc.
2) The numerical model can be utilized for dynamic implicit analysis and comparison with the Abaqus/dynamic explicit analysis can be drawn to examine the behavior of the model subjected to same dynamic loads with different approaches.
3) A study can be conducted to assess the effects of stress concentration caused by structural discontinuities, such as doors and windows etc., on the behavior of RCSPs when exposed to dynamic loads.

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Data Availability Statement

On reasonable request, the corresponding author will provide any or all of the data, models, or code that support the findings of this study.

Declaration of competing interest

The authors state that they have no known competing financial interests or personal ties that could appear to have influenced the work described in this paper.

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