Numerical evaluation of the performance of two-way RC panels under blast loads

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Abstract

Nonlinear dynamic numerical modelling and analysis of concrete panels subjected to blast loads is presented in this paper. Reinforced concrete panels of dimension 1.0×1.0 m and different thicknesses and supported on four sides are subjected to blast loads produced by the detonation of high explosive charges. The modelling and analysis was conducted using ANSYS AUTODYN solver. The accuracy of the model is verified against experimental results of blast load tests on reinforced concrete horizontal slabs subjected to the detonation of high explosive charges above them. The model was capable of simulating the observed damage and displacement with reasonable accuracy. The verified model is used for extensive parametric study to examine the effect of different design parameters on the performance of reinforced concrete slabs under blast loads. The design parameters considered in this study include the effect of concrete compressive strength, panel thickness, reinforcement steel ratio, arrangement of reinforcement steel, and boundary condition on the behaviour of RC panel under blast load. The performance was evaluated in terms of maximum displacement, extent of damage and energy absorbed.
Keywords: blast load, nonlinear numerical modelling, dynamic, ANSYS.

1 Introduction

The blast load effect on buildings and building components has received considerable attention in recent years [1, 2]. This is mainly due to the increase in blast events resulting from different accidents and terrorism activities targeting important structures in different parts of the world. This paper is concerned with the effect of dynamic loading produced by the detonation of high explosives on concrete panels representing slab or wall elements. Studies have shown that blast

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loads with short duration and high magnitude influence the response of the structure and modify the material behaviour [3–5]. Results of experimental research on steel [6, 7], concrete [8, 9] and FRP [10] panels subjected to blast loads are reported in the literature. Beams, slabs and shells under blast loads are mostly studied with limit analysis theory, which assumes rigid-plastic behaviour for the material.

With the rapid development of computer hardware over the last few decades, it has become possible to make detailed numerical simulations of blast loads on personal computers, significantly increasing the availability of these methods. In this paper the parametric study is conducted using ANSYS AUTODYN Solver provided by Workbench explicit dynamic modules in ANSYS V14.5.

2 Numerical model verification

The numerical model developed in this paper is verified against the experimental results of published research on blast loading response of reinforced concrete panels by Razaqpur *et al.* [11]. The control specimens of the experimental program are modelled and its nonlinear blast response is simulated and compared to the test results.

Razaqpur *et al.* [11] conducted an experimental program which consist of eight 1000×1000×70 mm panels made of reinforced concrete. Five of the panels were used as control while the remaining four were retrofitted with adhesively bonded 500 mm wide GFRP laminate strips on both faces, one in each direction parallel to the panel edges. The panels were subjected to blast loads generated by the detonation of either 22.4 kg or 33.4 kg ANFO explosive charge located at a 3 m standoff distance. Blast wave characteristics, including incident and reflected pressures and impulses, as well as panel central deflection and strain in steel and on concrete/FRP surfaces were recorded [11].

The concrete panels were doubly reinforced with welded steel mesh of designation MW25.8, which has bar cross-sectional area of 25.8 mm^2, mass per unit area of 2.91 kg/m^2 and center-to-center spacing of 152 mm in each direction. The bar yield stress and ultimate strength are 480 MPa and 600 MPa, respectively. The concrete had an average 28 days compressive strength of 40 MPa, with its average strength at the age of testing the panels being 42 MPa.

ANSYS V.14.5 – Explicit dynamics system is used for simulating the response of the control specimen to blast. The model is used to evaluate the central deflection, the strain in the reinforcing steel, the strain in the surface of the concrete panel, the kinetic energy, the internal energy, and the total work. The mathematical model is composed of three parts. The first part is the concrete body that has a 1000×1000×70 mm dimensions as per the experimental specimen CS2 [11]. The second and third parts are the reinforcing steel, which has bar cross-sectional area of 25.8 mm^2 as illustrated in Figure 1.

Solid element was used to simulate both the concrete body and the reinforcing steel bars. The characteristics of the solid element are governed by the mesh type and characteristics. The mesh physics preference is set to explicit with coarse relevance centre and triangle surface mesher program controlled which leads to tetrahedrons element as shown in Figure 1. For the simulation of the concrete panel the CONC-40MPA material is used. Furthermore the STEEL 4340 is assigned to simulate the steel reinforcing bars. To imitate the same boundary condition the mathematical model is assigned to have fixed supports all over the panel edges. The assigned load is chosen to be pressure type and equal to 5.059 MPa in time duration equal to 7.7×10^{-4} sec.

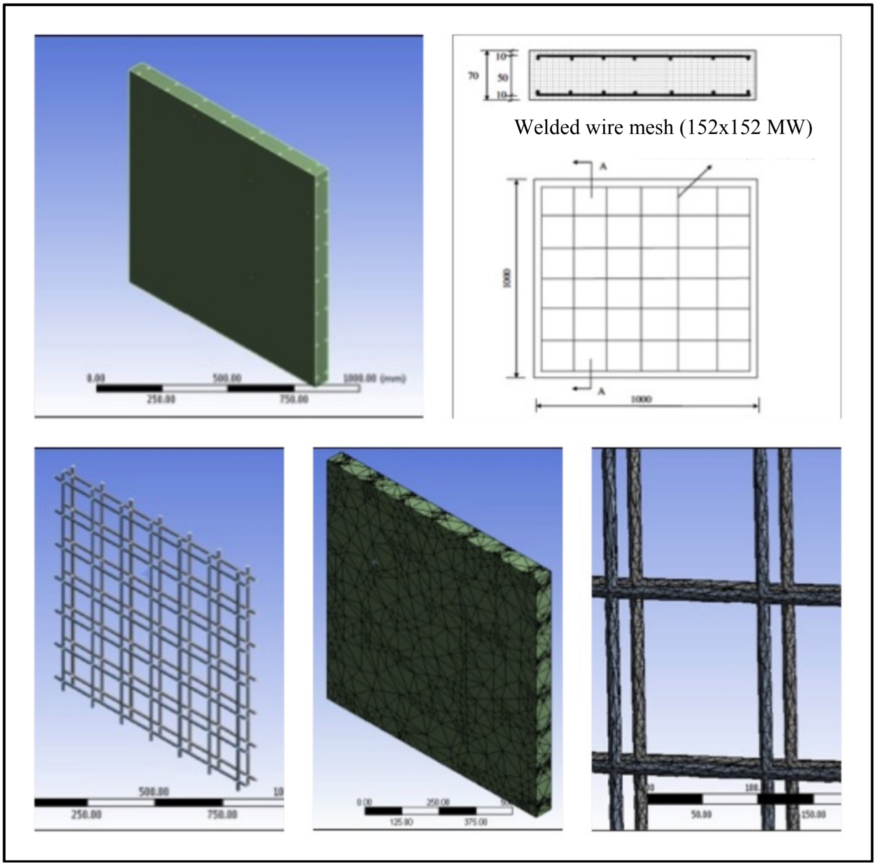

Figure 1: Mathematical model geometry and mesh details.

To verify the mathematical model, one of the control specimens, CS2 [11], was chosen for correlation with the mathematical model. The control specimen had maximum central deflection of 13.12 mm, while the mathematical model evaluated the maximum central deflection as 16 mm (as shown in Figure 2). Correlating numerical to experimental results reveals that the mathematical model gives reasonably accurate results related to maximum central deflection with accepted error in the order of 20%.

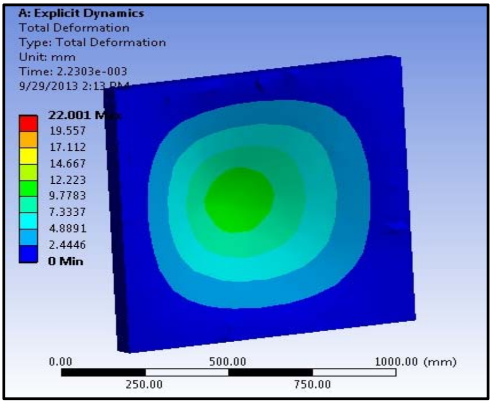

Figure 2: Total deformation of the RC panel.

3 Parametric study through numerical simulation

The developed and verified numerical model is now used for further parametric study using numerical simulation. The parameters considered in this research are the concrete compressive strength, concrete panel thickness, boundary conditions, reinforcement steel quantity, and steel reinforcement arrangement.

A total of 32 panel models 1000×1000 mm were used with two different thicknesses 100 mm and 160 mm. Two different steel bar sizes 12 mm and 10 mm and two reinforcement spacing, 150 mm and 100 mm were used in this study. Two concrete compressive strengths were also used: 35 MPa as normal strength concrete and 140 MPa as high strength concrete. For all models steel reinforcement had yield stress equal to 400 MPa. Details of the models used in this parametric study is summarized in Table 1.

Solid element was used to simulate both the concrete body as well as the reinforcing steel bars. Furthermore the configurations of the solid element are governed by the mesh type and characteristics. The mesh physics preference is set to explicit with coarse relevance centre and triangle surface mesher is program controlled which leads to tetrahedrons element as used in the verification process and shown in Figure 1. For the simulation of the concrete panel the CONC-35MPA and CONC-140MPA material are used. Furthermore, the STEEL 4340 is assigned to simulate the steel reinforcing bars.

The parametric study examines the effect of the different parameters mentioned above on the response measures represented by maximum deformation, strain in concrete body, strain in the reinforcement mesh, internal energy of both concrete body and reinforcement mesh, and plastic work-done by both the concrete body and the reinforcement mesh.

Two types of support conditions are considered for the models, either fixed supports all over the panel edges or hinged supports. The assigned load assumed to be uniform pressure of 23.5 MPa decreasing to zero in duration equal to 6.4×10^{-4} seconds representing a 100 Kg TNT charge at a distance of 3 m.

Table 1: Parametric study specimen's details.

Specimen No.	Conc. strength (MPa)	Panel Thickness (mm)	Yield stress (MPa)	Reinforcement Ratio	Reinforcement bar Size (mm)	Reinforcement Spacing (mm)	Edge support
001	35	100	400	0.0160	12	150	Fixed
002	35	100	400	0.0160	12	150	Hinged
003	35	100	400	0.0230	12	100	Fixed
004	35	100	400	0.0230	12	100	Hinged
005	35	100	400	0.0110	10	150	Fixed
006	35	100	400	0.0110	10	150	Hinged
007	35	100	400	0.0160	10	100	Fixed
008	35	100	400	0.0160	10	100	Hinged
009	35	160	400	0.0099	12	150	Fixed
010	35	160	400	0.0099	12	150	Hinged
011	35	160	400	0.0140	12	100	Fixed
012	35	160	400	0.0140	12	100	Hinged
013	35	160	400	0.0069	10	150	Fixed
014	35	160	400	0.0069	10	150	Hinged
015	35	160	400	0.0099	10	100	Fixed
016	35	160	400	0.0099	10	100	Hinged
017	140	100	400	0.0160	12	150	Fixed
018	140	100	400	0.0160	12	150	Hinged
019	140	100	400	0.0230	12	100	Fixed
020	140	100	400	0.0230	12	100	Hinged
021	140	100	400	0.0110	10	150	Fixed
022	140	100	400	0.0110	10	150	Hinged
023	140	100	400	0.0160	10	100	Fixed
024	140	100	400	0.0160	10	100	Hinged
025	140	160	400	0.0099	12	150	Fixed
026	140	160	400	0.0099	12	150	Hinged
027	140	160	400	0.0140	12	100	Fixed
028	140	160	400	0.0140	12	100	Hinged
029	140	160	400	0.0069	10	150	Fixed
030	140	160	400	0.0069	10	150	Hinged
031	140	160	400	0.0099	10	100	Fixed
032	140	160	400	0.0099	10	100	Hinged

4 Results of parametric study

The failure in concrete panel is represented by the percentage of nodes with strain level exceeding the maximum assigned strain. To define the contribution of concrete panel and the reinforcement steel, both internal energy and plastic work had been measured. In this paper, the term upper reinforcement steel is refers to the reinforcement steel near to the detonation surface, while the term lower reinforcement steel refers to the reinforcement steel on the other side.

4.1 Effect of concrete compressive strength

By examining panel 001 and panel 017 both had the same properties except the concrete strength which is 35 MPa for panel 001 and 140 MPa for panel 0017. Panel 017 had maximum central deformation equal to 57.6 mm which is less by 15.4% than panel 001 which had 68.1 mm as a maximum central deformation. Moreover panel 017 achieved lower percentage of concrete failure (65.3%) than panel 001 (82.9%) by 21.2%. On the other hand reinforcement steel of panel 001 had a higher percentage of failure than panel 017 by 34%. And by comparing the other panels to see the effect of concrete compressive strength, all panels exhibit the same behavior as shown in Figure 3 which shows the maximum central deflection for the other compared panels. The effect of concrete strength on percentage of concrete failure and steel failure is presented in Figures 4 and 5, respectively. All high strength concrete panels exhibited higher internal energy of concrete when compared to the normal concrete panels. On the other hand all high strength concrete panels exhibited lower internal energy of reinforcement steel when compared to normal strength concrete panels.

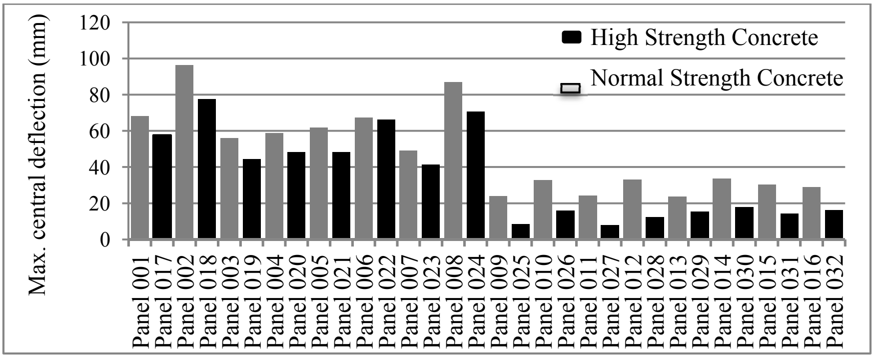

Figure 3: Effect of concrete strength on maximum central deflection.

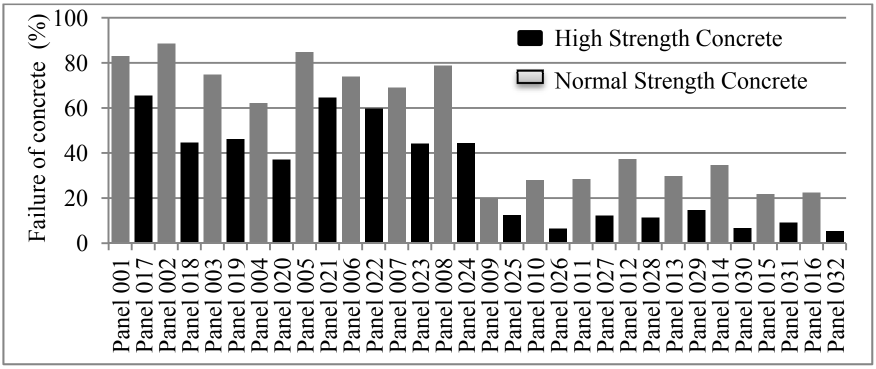

Figure 4: Effect of concrete strength on failure percentages of concrete panels.

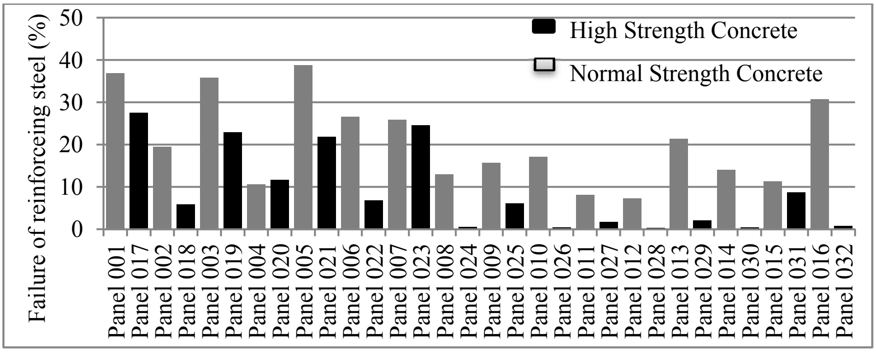

Figure 5: Effect of concrete strength on failure percentages of reinforcing steel.

4.2 Effect of panel thickness

Both panels 001 and 009 have similar characteristics except for the thickness which is 100 mm for panel 001 and 160 mm for panel 009. Panel 009 experienced maximum central deformation of 23.81 mm which is less by 65% than that of panel 001 which was 68.1 mm. Moreover panel 009 achieved lower percentage of concrete failure (20.14%) than panel 001 (82.9%) by 75.72%. On the other hand reinforcement steel of panel 001 had higher percentage of failure than panel 009 by 136.5%. Furthermore the concrete body of panel 001 exhibited higher internal energy (4.2×10^{7} MJ) than panel 009 (3.3×10^{7} MJ) by 27.3%. Additionally the upper reinforcement steel in panel 001 exhibited higher internal energy (7×10^{6} MJ) than panel 009 (1.8×10^{6} MJ) by 288% due to the higher thickness of panel 009. Likewise the lower reinforcement of panel 001 exhibited higher internal energy (5.9×10^{6} MJ) than panel 009 (0.7×10^{6} MJ) by 743%. All other panels showed similar behaviour when examining the effect of the thickness of the concrete body on the maximum central deflection and percentage of failure as shown in Figures 6–8.

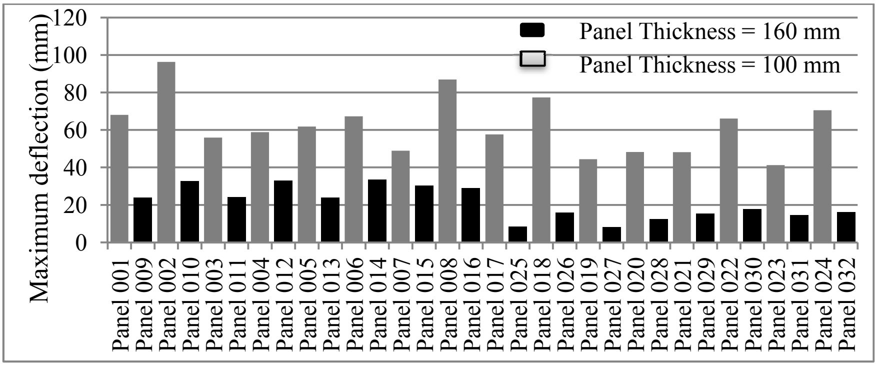

Figure 6: Effect of panel thickness on maximum central deflection.

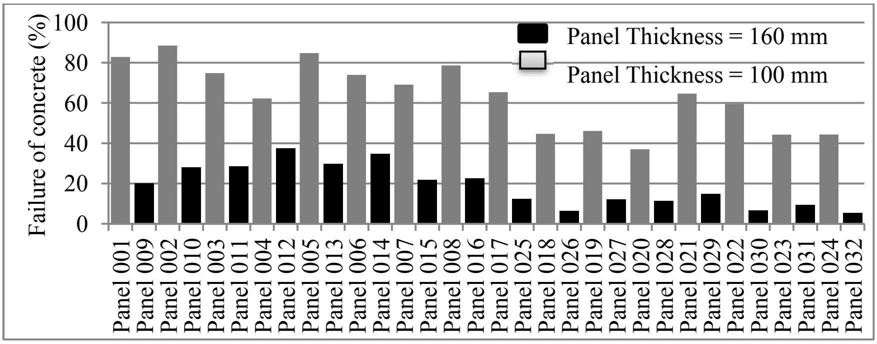

Figure 7: Effect of panel thickness on failure percentages of concrete panels.

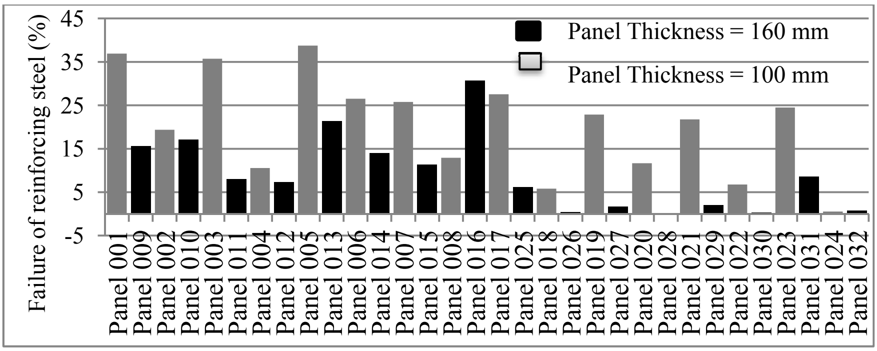

Figure 8: Effect of concrete panel thickness on failure percentages of steel.

4.3 Effect of reinforcement steel ratio

Panels 001 and 005 are similar except for the reinforcing bars diameter which is 12 mm and 10 mm for panels 001 and 005, respectively, both spaced at 150 mm. Thus, panel 001 has higher reinforcement ratio (0.016) than panel 005 (0.011) by 45.4%. Panel 001 experienced maximum central deformation equal to 68.1 mm which is slightly higher by 10.1% than panel 005 which had 61.87 mm. Panel 001 experienced lower percentage of concrete failure (82.9%) than panel 005 (84.8%) by 2.24%. Reinforcement steel of panel 001 had lower percentage of failure than panel 005 by 4.77%. The concrete body of panel 001 exhibited higher internal energy (4.2×10^7 MJ) than panel 005 (3.4×10^7 MJ) by 23.5%. The upper reinforcement steel in panel 001 exhibited higher internal energy (7.1×10^6 MJ) than panel 005 (2.7×10^6 MJ) by 163%. Likewise the lower reinforcement of panel 001 exhibited higher internal energy (5.9×10^6 MJ) than panel 005 (2.4×10^6 MJ) by 145.8%. The effect of reinforcement ratio on the performance of all considered panels showed similar effect as shown in Figures 9–11.

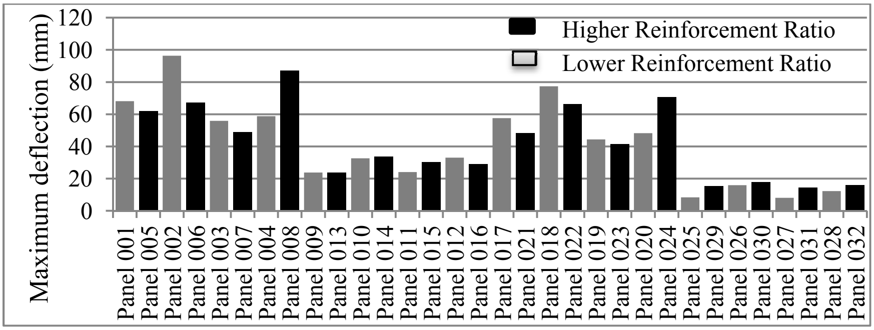

Figure 9: Effect of reinforcement ratio on maximum central deflection.

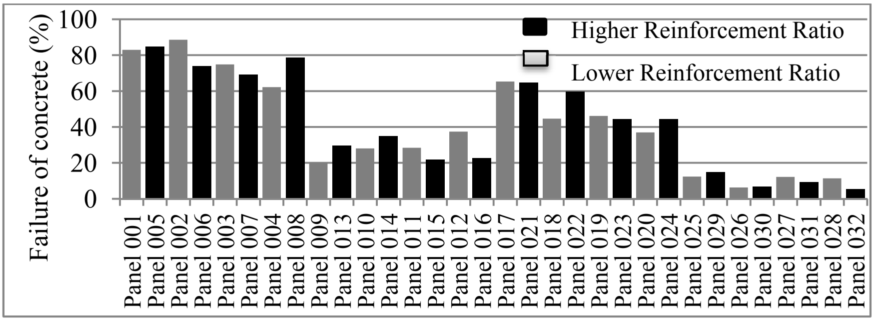

Figure 10: Effect of reinforcement ratio on failure percentages of concrete.

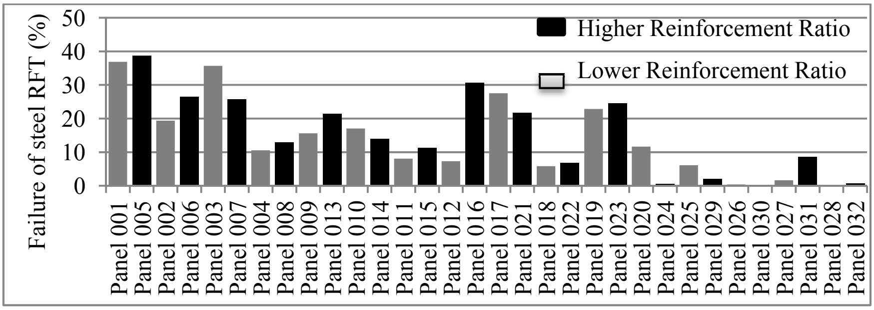

Figure 11: Effect of reinforcement ratio on failure percentages of steel.

4.4 Effect of reinforcement steel arrangement

Panels 001 and 007 are similar except for bar spacing which is 100 mm in panel 001 and 150 mm in panel 007. Maximum central deflection of panel 001 (68.1 mm) is slightly higher than panel 007 (48.95 mm). Panel 001 experienced higher percentage of concrete failure (82.94%) than panel 007 (69.1%) and lower percentage of steel failure than panel 007 by 43%. The concrete of panel 001 exhibited higher internal energy (4.2×10^7 MJ) than panel 007 (3×10^7 MJ). The upper reinforcement in panel 001 exhibited higher internal energy (7.1×10^6 MJ) than panel 007 (4×10^6 MJ). Likewise the lower reinforcement of panel 001 exhibited higher internal energy (5.9×10^6 MJ) than panel 007 (3.4×10^6 MJ). The effects of bar spacing on the blast performance of the considered panels are shown in Figures 12–14.

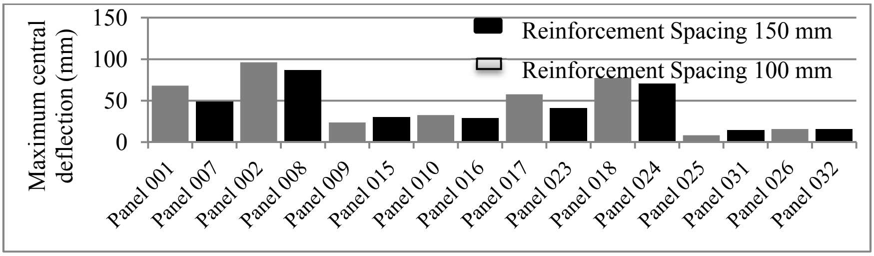

Figure 12: Effect of reinforcement steel arrangement on maximum deflection.

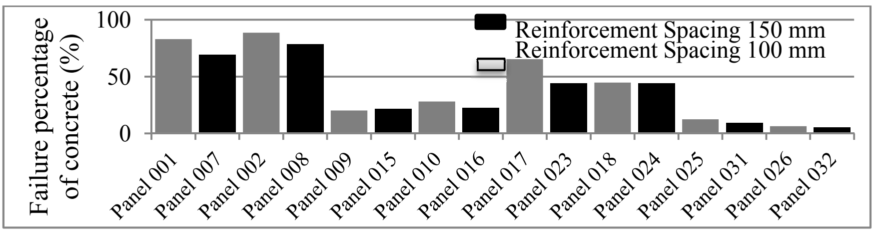

Figure 13: Effect of steel arrangement on failure percentages of concrete panels.

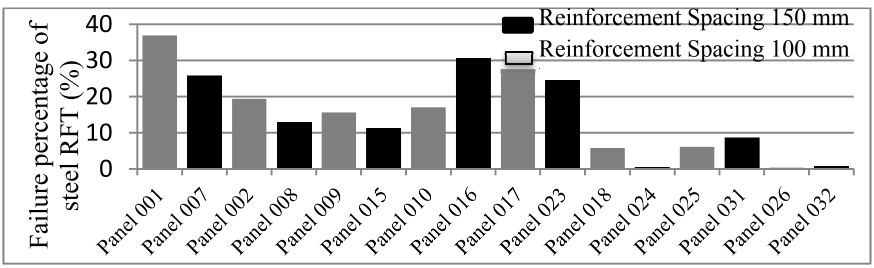

Figure 14: Effect of steel arrangement on failure percentages of reinforcing steel.

4.5 Effect of boundary conditions

Panels 001 and 002 are similar except for the boundary conditions which is fixed for panel 001 and hinged for panel 002. Panel 001 had maximum central deflection equal to 68.1 mm which is less by 29.4% than panel 002 of 96.4 mm. Panel 001 showed slightly lower percentage of concrete failure (82.9%) than panel 002 (88.6%) and higher percentage of steel failure than panel 002 by

90.1%. The concrete body of panel 001 exhibited lower internal energy (4.2×10^7 MJ) than panel 002 (4.6×10^7 MJ) by 8.7%. The upper reinforcement steel in panel 001 exhibited higher internal energy (7×10^6 MJ) than panel 002 (4.4×10^6 MJ) by 59.1%. Likewise the lower reinforcement of panel 001 exhibited higher internal energy (5.9×10^6 MJ) than panel 002 (4.6×10^6 MJ) by 28.3%. The results for all considered panels are shown in Figures 15-17.

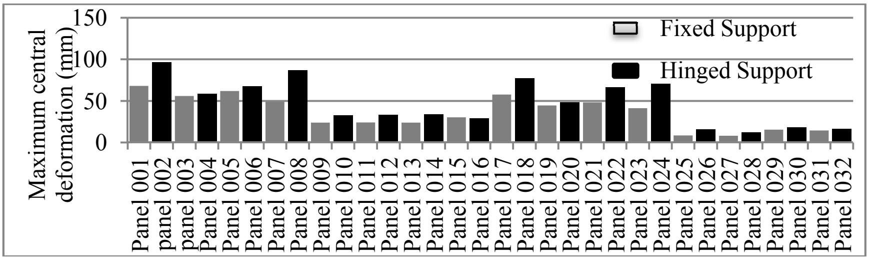

Figure 15: Effect of boundary conditions on maximum central deflection.

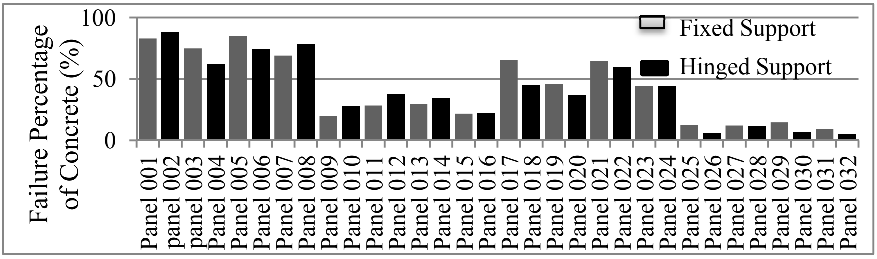

Figure 16: Effect of boundary condition on failure percentages of concrete.

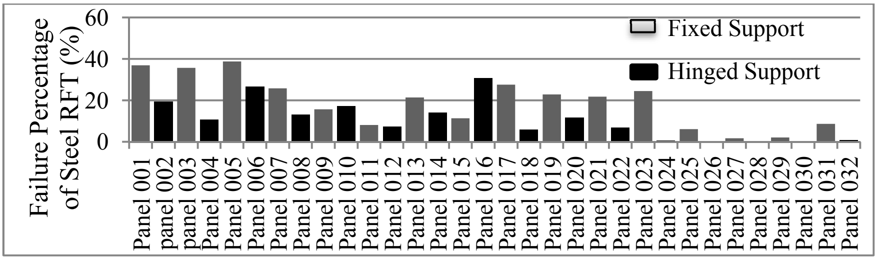

Figure 17: Effect of boundary condition on failure percentages of steel.

5 Conclusions

Nonlinear dynamic analysis of two-way concrete panel (1000×1000 mm) with different thicknesses under the effect of blast load is considered in this paper. The validity of the developed model was verified against experimental results. The developed model was further used for detailed parametric study. A total of

32 models were consider to examine the effect of panel thickness, concrete strength, reinforcement ratio, bar spacing, and boundary conditions on the behaviour of the concrete panel under the effect of simulated blast load.

Using high strength concrete (HSC) with small thickness (100mm) reduces the maximum central deflection by 16.5%, while using HSC with bigger thickness (160 mm) reduces the maximum central deflection by 53%. Using HSC with higher thicknesses reduces the failure percentage in the concrete panel by 63.5%, while in the case of small thicknesses it reduces the failure by 34%. The use of HSC with higher thicknesses also reduces the reinforcement failure up to 80%, while in the case of lower thicknesses it reduces the reinforcement failure up to 50%. Increasing the panel thickness is one of the most efficient methods of decreasing the maximum central deflection and the failure of the concrete panel as well. Based on the presented results, increasing the panel thickness by 60% decreases the maximum deflection by 66% and decreases the failure of concrete panel by 72%. Furthermore, increasing the panel thickness decease the damage of the reinforcement steel specially in the case of using HSC. Increasing the reinforcement ratio does not have a significant effect on the maximum deflection and failure percentage especially in the case of HSC. Panels with the same reinforcement ratio and less steel bars spacing have less maximum deformation and failure percentage. Panels with hinged supports experienced higher maximum central deformation than panels with fixed supports by 37%.

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