Performance of One-way Composite Reinforced Concrete Slabs under Explosive-induced Blast Loading

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Abstract. A finite element model is developed in ABAQUS/CAE to simulate the behavior of one-way reinforced concrete slabs under air-blast loading. Several published empirical equations have been utilized to model air-blast pressure, P(t), for different combinations of the explosive charges and standoff distances. The values of the peak overpressure have been taken from the field explosion tests conducted on four normal reinforced concrete (NRC) slabs of size 2000 mm x 1000 mm x 100 mm in the School of Civil, Environmental, and Mining Engineering, The University of Adelaide, Australia (2009). The slabs were doubly reinforced with HYSD steel re-bars of diameter 12 mm spaced at 100 mm c/c in the major bending plane (ρ = 1.34%) and 200 mm c/c in the other plane (ρ = 0.74%), where “c/c” is the center-to-center distance between two adjacent re-bars. A plasticity-based damage model for concrete and an explicit solver in ABAQUS/CAE are adopted in the finite element (FE) simulation. The available test results and finite element analysis predictions, including maximum mid-span displacement and damage to the RC slabs, are presented and weighed. The computed results are found in good agreement with the experimental ones. The validated model is used to examine the performance of RC slabs with 25% replacement of the conventional steel re-bars by the FRP re-bars of equivalent strength on the compression face only, remote face only, and both faces of the slab. The replacement has been done for higher strength and ductility. Substituted FRP re-bars are of (1) Aramid fiber-reinforced polymer (AFRP), (2) Basalt fiber-reinforced polymer (BFRP), (3) Carbon fiber-reinforced polymer (CFRP), and (4) Glass fiber-reinforced polymer (GFRP). The results are also compared with those obtained with a 50% replacement of the steel re-bars by the FRP re-bars.

1. Introduction
Numerous studies have been undertaken experimentally as well as analytically to investigate the performance of conventionally reinforced concrete (RC) slabs under close-in or near field detonations (Zhou Q. X. et al., 2008 [20]; Wu C. et al., 2009 [17]; Wang W. et al., 2012 [15]; Castedo R. et al., 2015 [7]; Li J. et al., 2015 [13]; Yao S. et al., 2016 [18]; Yang F. et al., 2019 [19]; Zhao C. et al., 2019 [21]; Kumar V. et al., 2020 [12]; Anas M. S. et al., 2020 [3]). Zhou Q. X. et al. (2008) [20] numerically investigated the behavior of 1300 mm x 1000 mm x 100 mm one-way high strength steel fiber reinforced concrete (170 MPa) and ordinary reinforced concrete (50 MPa) slabs under close-in detonations using ANSYS FEA program. The damage in the concrete was modeled using Mazar’s
plasticity-based damage model. Two types of cylindrical shaped charges were considered, namely; CompB (60:40 RDX: TNT) and Orica Powergel Magnum at a fixed detonation distance of 0.10 m in free air. It was reported that the blast performance of the SFRC slab was superior to the ORC slab when subjected to similar peak reflected pressures. Wu C. et al. (2009) [17] conducted field explosion tests to investigate the effect of concrete strength on the damage resistance of 2000 mm x 1000 mm x 100 mm one-way RC slabs under close-in air-blast loading. A total of 8 slabs were tested. Six out of eight slabs were constructed with normal strength concrete (39.50 MPa), one with plain ultra-high performance fiber concrete (151.60 MPa), and one with reinforced ultra-high performance fiber concrete. Two out of six NSC slabs were strengthened with externally bonded CFRP sheets on the blast face only. The slabs were subjected to different explosive charge weights ranging from 1 to 20 kg TNT detonated at different standoff distances in free air. The slabs were doubly reinforced with Fe600 HYSD steel re-bars of diameter 12 mm spaced at 100 mm c/c in the major bending plane ($\rho = 1.34\%$), and 200 mm c/c in the other plane ($\rho = 0.74\%$), where “$\rho$” is the reinforcement ratio. All the slabs were supported on two short edges. The average compressive strength, tensile strength, and Poisson’s ratio were 8.20 MPa, 0.20, and 28.30 GPa, respectively [17]. The thickness of the clear cover was 10 mm. The HYSD steel had a yield strength of 600 MPa, the ultimate tensile strength of 660 MPa, and Young’s modulus of 210 GPa [17]. They concluded that the concrete slabs constructed with ultra-high-strength concrete experienced lesser damage than normal reinforced concrete (NRC) slabs when subjected to similar air-blast loading. However, the RUHPFC slab performed better than all other slabs. Wang W. et al. (2012) [15] conducted small scale field tests to investigate the damage modes of one-way square RC slabs under close-in detonations. Tests were conducted on six slabs. Two out of six slabs were 750 mm x 750 mm x 30 mm, two with 1000 mm x 1000 mm x 40 mm, and two with 1250 mm x 1250 mm x 50 mm in dimensions. The RC slabs were subjected to different TNT charge weights ranging from 0.13 to 0.94 kg at standoff distances of 0.3, 0.4, and 0.5 m in free air. The average 28-day compressive strength of the concrete was 39.50 MPa. The slabs were singly reinforced in both planar directions with 6 mm diameter HYSD steel bar mesh spaced at 75 mm c/c ($\rho = 1.43\%$). They identified two damage modes, namely; spall damage with few cracks, and moderate spall damage. It was reported that the increase of charge weight could lead to a change of damage mode in the slabs from flexural failure to perforation failure mode. Yang F. et al. (2019) [19] experimentally and numerically investigated the performance of 2000 mm x 2000 mm x 100 mm one-way reinforced concrete slabs under close-in air-blast loading. A total of 11 slabs were tested. Three out of 11 slabs were constructed with normal strength concrete (34.91 MPa), two concrete slabs were constructed with a 10% replacement of fine aggregate by recycled rubber powder (32.58 MPa), four slabs with 30% replacement of fine aggregate (27.93 MPa), and two with 50% replacement of fine aggregate by recycled rubber powder (24.63 MPa). All the slabs were doubly reinforced in both directions with 8 mm diameter mild steel re-bars spaced at 200 mm c/c ($\rho = 0.251\%$). The slabs were subjected to different blasts of charge weights varying from 2.30 kg to 5.60 kg ANFO at a fixed detonation distance of 0.5 m in free air. It was found that the slabs with concrete having fine aggregate replaced by recycled rubber powder underwent lesser mid-span deflection and damage than the normal concrete slabs (without replacement of fine aggregate by recycled rubber). RC slabs with concrete having a 10% replacement of fine aggregate by recycled rubber powder were reported better than all other slabs with concrete having other replacement levels of fine aggregate tested in the experiment. The results were also compared and found to be in reasonable agreement with the predictions of LS-DYNA 971R9 software. Zhao C. et al. (2019) [21] numerically investigated the performance of 1000 mm x 1000 mm x 40 mm one-way normal reinforced concrete (NRC) and ordinary reinforced concrete (ORC) slabs under air-blast loading using LS-DYNA software. The slabs were isotopically reinforced on tension face only with HYSD steel re-bars of 6 mm diameter spaced at 75 mm c/c ($\rho = 1.43\%$) and were supported on two opposite edges. The NRC slab was modeled with a 60-degree steel reinforcement layout. A cylindrical shaped TNT explosive charge of weight 0.46 kg was considered at a fixed standoff distance of 0.40 m in free air. Results indicated that the NRC slab experienced lesser damage and mid-span deflection than the ORC slab. Kumar V. et al. (2020) [12] conducted field blast
tests to investigate the effect of charge mass (W) and standoff distance (R) on the behavior of 1000 mm x 1000 mm x 100 mm one-way RC slabs under air-blast loading. A total of six slabs were tested. The average 28-day compressive strength of the concrete was 40 MPa. All the slabs were singly reinforced in both directions with 10 mm diameter Fe500 HYSD steel re-bars spaced at 100 mm c/c. The slabs were subjected to three different quantities of gelatin explosive at standoff distances of 0.10 and 0.50 m in free air. They found that the decrease of the scaled distance (R/W 1/3) could lead to a change of failure mode of slabs from localized failure to the bending-punching failure. The experimental results were also compared and found to be in good agreement with the predictions of the LS-DYNA program.

A finite element model is developed in this study to simulate the dynamic response of one-way reinforced concrete slabs under close-in air-blast loading. The commercial finite element analysis (FEA) program, ABAQUS/CAE is employed for this purpose. The model is validated using available values from blast tests of four NRC slabs conducted by Wu C. et al. (2009) [17]. The validated model is used to examine the performance of concrete slabs with 0.25 reinforcement replacement ratio of the conventional reinforcement by the equivalent FRP reinforcement on the compression face (D2), on the remote face (D3), and on both the faces (D4). The replacement has been done for higher strength and ductility. The results are also compared with those obtained with 0.50 reinforcement replacement ratio of the conventional reinforcement by the equivalent FRP reinforcement on the compression face (C2), on the remote face only (C3), and on both the faces (C4). Table 1 lists the mechanical properties of the replaced FRP re-bars. The effect of replacement of the conventional reinforcement by the FRP reinforcement of equivalent strength on the behavior of the slabs under blast loading had not been investigated. Equivalent diameters of replaced re-bars are given in Table 2. This study shall open a new vista for engineers and researchers working in the field of blast dynamics.

Table 1: Mechanical properties of replaced polymer re-bars [4]

| Properties                  | AFRP | BFRP | CFRP | GFRP |
|-----------------------------|------|------|------|------|
| Mass density (g/cm³)        | 1.33 | 2.13 | 1.60 | 1.95 |
| Ultimate tensile strength (GPa) | 2.54 | 1.45 | 3.10 | 1.25 |
| Young’s modulus (GPa)       | 83   | 78   | 148  | 55   |
| Ultimate percentage elongation (%) | 3.15 | 2.40 | 1.70 | 2.50 |

Table 2: Equivalent diameter of substituted FRP re-bars

| Reinforcing/substitute material | Tensile strength (MPa) | Diameter of re-bar (mm) | Cross-sectional area of re-bar (mm²) |
|---------------------------------|------------------------|-------------------------|-------------------------------------|
| HYSD (Fe600)                    | 660                    | 12                      | 113                                 |
| AFRP                            | 2540 (628%)            | 6                       | 28                                  |
| BFRP                            | 1450 (6120%)           | 8                       | 50                                  |
| CFRP                            | 3100 (6370%)           | 6                       | 28                                  |
| GFRP                            | 1250 (689%)            | 10                      | 79                                  |

* * Equivalent diameter to HYSD steel re-bar  
* * Percentage increase in tensile strength (%)  
* c Equivalent cross-sectional area of re-bar

2. Numerical Modelling

2.1. Air-blast loading

The idealized time history of air-blast wave pressure proposed by Wu C. and Hao H. (2005) [16] is shown in Figure 1(a) consists of positive and negative pressure phases. Current blast design guidelines available in IS 4991(1968) [5], IS 6922 (1973) [6], TM 5-1300(1990) [11], UFC 3-340-02 (2008) [14], and ASCE/SEI 59-11(2011) [2] suggests to consider positive pressure phase only in analysis and design of concrete structures by assuming that the suction pressure phase is much weaker and does not affect the structural response/damage [8]. Thus, the effect of the negative pressure phase on the
behavior of RC slabs under blast loading has been neglected in the present work. The values of air-blast wave parameters such as arrival time of blast wave, rising time, and duration of positive phase have been calculated using empirical formulae proposed by Wu C. and Hao H. (2005) [16] for different scaled distances. Wu C. and Hao H. (2005) [16] compared the results of proposed empirical equations and found to be in good agreement with the predictions of TM 5-1300(1990) [11] and Henrych J. (1979) [9] experimental results.

\[ t_A = 0.34 S^{1.4} W^{-0.2} / C_a \]  \hspace{1cm} (1)

\[ t_1 = 0.0019 \left( \frac{S}{W} \right)^{1.30} \]  \hspace{1cm} (2)

\[ t_2 = 0.0005 S^{0.72} W^{0.16} \]  \hspace{1cm} (3)

\[ t_d = t_1 + t_2 \]  \hspace{1cm} (4)

\[ P(t) = \begin{cases} P_o, & (0 \leq t < t_A) \\ P_o + P_{OP} \left( \frac{t}{t_1} \right), & (t_A \leq t \leq t_1) \\ P_o + P_{OP} \left( 1 - \frac{t - t_1}{t_2} \right) \exp \left( -\frac{\zeta(t-t_1)}{t_2} \right), & (t_1 \leq t) \end{cases} \]  \hspace{1cm} (MPa) \hspace{1cm} (5)

Here, \( W \) = Explosive charge (kg); \( S \) = Detonation distance (m); \( C_a \) = Speed of sound in air (≈340 m/sec); \( t_A \) = Blast wave arrival time (sec); \( t_1 \) = Rising time (sec); \( t_2 \) = Decreasing time (sec); \( t_d \) = Duration of positive phase (sec); \( P_o \) = atmospheric pressure (≈0.1 MPa); \( P_{OP} \) or \( P_i \) = peak overpressure (MPa); and \( \zeta \) = decay coefficient. The decay coefficient \( (\zeta) \) can be calculated using the following equations [16]:

\[ \zeta = \begin{cases} 3.02 P_{OP}^{0.38} + 6.85 P_{OP}^{0.79} \exp \left( -4.55 \frac{t-t_1}{t_2} \right), & (t_1 \leq t \leq t_d) \\ 1.96 P_{OP}^{0.25} + 0.176 P_{OP} \exp \left( -0.73 P_{OP}^{-0.49} \left( \frac{t-t_1}{t_2} \right) \right), & (t_d < t) \end{cases} \]  \hspace{1cm} \text{for } P_{OP} \leq 1.0; \hspace{1cm} (6)

and,

\[ \zeta = \begin{cases} 1.62 P_{OP}^{0.30} + 5.13 P_{OP}^{0.28} \exp \left( -1.05 P_{OP}^{0.37} \left( \frac{t-t_1}{t_2} \right) \right), & (t_1 \leq t \leq t_d) \\ 0.74 P_{OP}^{0.17} + 2.71 P_{OP}^{0.28} \exp \left( -0.26 P_{OP}^{0.33} \left( \frac{t-t_1}{t_2} \right) \right), & (t_d < t) \end{cases} \]  \hspace{1cm} (7)

for \( 1 \leq P_{OP} \) (MPa) \leq 100. The test results of peak overpressure \( (P_{OP}) \) published by Wu C. et al. (2009) [17] have been used in the current study summarized in Table 3. An explicit solver in ABAQUS/CAE has been adopted to simulate the blast response of RC slabs. The top surface of the slabs has been acted upon by time-dependent blast pressure, \( P(t) \) from above as shown in Figure 2(a). Simple supports are assigned to the short edges of the slabs. The material model used to simulate the damage in the RC slabs under air-blast loading is described in the following sub-section.
Table 3: Estimated values of air-blast wave parameters

| Slab ID | Scaled distance, Z (m/kg$^{1/3}$) | Wu C. (2009) experimental Peak overpressure, $P_{OP}$ (MPa) | Arrival time of air-blast wave, $t_A$ (ms) | Rising time, $t_1$ (ms) | Positive phase duration, $t_d$ (ms) | $t_A + t_d$ (ms) |
|---------|----------------------------------|--------------------------------------------------|----------------------------------|-----------------|----------------------------------|-----------------|
| NRC-1   | 3.0                              | 0.13                                             | 4.65                             | 3.28            | 4.38                             | 9.03            |
| NRC-2   | 1.50                             | 0.54                                             | 3.07                             | 0.15            | 1.69                             | 4.76            |
| NRC-3   | 0.93                             | 1.43                                             | 1.25                             | 0.48            | 1.73                             | 2.98            |
| NRC-4   | 0.75                             | 1.72                                             | 1.16                             | 0.15            | 1.22                             | 2.38            |

Figure 1. (a) Typical air-blast pressure – time history proposed by Wu C. and Hao H. (2005) [16]; and (b) Estimated air-blast pressure - time histories

Figure 2. (a) Load application of blast pressure, $P(t)$; and (b) Rendered view of reinforcement

2.2. Concrete material model

In the present study, the concrete damaged plasticity (CDP) model is employed to model the damage in the concrete. The CDP model is a damage-plasticity model that assumes tensile cracking and compressive crushing as two possible failure mechanisms of the concrete [1, 10]. The pattern and characteristics of cracks can be identified using this plasticity model. The evolution of the yield or failure surface is controlled by two hardening variables, namely; tensile equivalent plastic strain ($\varepsilon_{pl}^{t}$), and compressive equivalent plastic strain ($\varepsilon_{pl}^{c}$) [10]. These variables are connected to failure mechanisms of the concrete under tension and compression loadings. It is assumed that the uniaxial stress-strain curves can be converted into stress versus inelastic-strain curves [1]. This conversion is performed automatically by ABAQUS/CAE from the user-provided stress versus inelastic-strain data. Figure 3 shows that the response of concrete under uniaxial compression and tension is assumed to be influenced by damaged plasticity and this assumption formed the basis of the
CDP model [10]. The uniaxial compressive and tensile responses of concrete with respect to the CDP model under compression and tension loadings are given by:

\[ \sigma_c = (1 - d_c)E_0(\varepsilon_c - \varepsilon_c^{pl, h}) \]  
\[ \sigma_t = (1 - d_t)E_0(\varepsilon_t - \varepsilon_t^{pl, h}) \]

Here, \( \sigma_c \) = nominal compressive stress, \( \sigma_t \) = nominal tensile stress, \( \sigma_{cu} \) = ultimate compressive stress, \( \sigma_{t0} \) = failure stress, \( \varepsilon_c \) = Compressive strain \( (\varepsilon_c^{pl, h} + \varepsilon_c^{el}) \), \( \varepsilon_t \) = Tensile strain \( (\varepsilon_t^{pl, h} + \varepsilon_t^{el}) \), \( \varepsilon_c^{pl, h} \) = plastic hardening compressive strain, \( \varepsilon_t^{pl, h} \) = plastic hardening tensile strain, \( \varepsilon_c^{el} \) = elastic compressive strain, \( \varepsilon_t^{el} \) = elastic tensile strain, \( E_0 \) = initial elasticity modulus of concrete (undamaged), \( E_u \) = reduced modulus of elasticity, \( d_c \) and \( d_t \) are the two damage variables ranging from zero (undamaged material) to one (total loss of strength). The model assumes that the reduction of the modulus of elasticity of the material can be expressed in terms of a scalar degradation variable, \( d \), as

\[ E_u = (1 - d_{i=c,t})E_0 \]

The input parameters for the CDP model of M40 grade concrete including plasticity parameters, Young’s modulus, and stress versus inelastic-strain data are derived from the study conducted by Hafezolghorani M. et al. (2017) [10].

![Figure 3](image-url)  
**Figure 3.** Concrete response to a uniaxial loading condition: (a) Compression, and (b) Tension [10]

2.3. Convergence test

In the current study, the concrete slabs and re-bars are discretized with explicit C3D8R 8-node 3D-Stress solid element and B32 3-node quadratic beam element. Seed sizes of 8 mm, 10 mm, 12.50 mm, 25 mm, and 50 mm have been considered for the mesh convergence test conducted at two different scaled distances of 0.75 and 3.0 m/kg\(^{1/3}\). The discretization of the model has been done with a 10 mm size of the mesh (1/10 of slab thickness) following the convergence test. The FE model with a 10 mm seed size has 225,315 nodes and 200,440 brick elements.

3. Results and Discussions

In this paper, experimental data published by Wu C. et al. (2009) [17] on four NRC slabs have been assumed as a benchmark to study the influence of the replaced FRP reinforcement on the blast response of the slabs. A summary of predicted mid-span deflections and available test values is presented in Table 4. There is a good agreement between the analytical and Wu C. (2009) [17]
experimental results. The predicted values of mid-span deflection in the NRC slabs are within 3% of the measured values (Table 4). Figure 4 shows that the ABAQUS/CAE model with a 10 mm seed size successfully captured the pattern of cracks developed along the span of the NRC-3 and NRC-4 slabs. The mid-span deflection – time histories for different slabs and reinforcement combinations are presented in Figure 5. It is apparent from Figure 5 that mid-span deflection variation in the concrete slabs with the conventional reinforcement on the compression face and 25% substituted FRP re-bars and 75% Fe600 HYSD steel re-bars on the remote face (D3) is more as compared with slabs having conventional reinforcement on the bottom face and 25% replaced FRP reinforcement and 75% conventional reinforcement on blast face (D2). Table 5 summarizes the predicted values of maximum deflection in the slabs for reinforcement combinations taken in this study.

**Table 4: Comparison of mid-span deflection in RC slabs**

| Slab ID | Explosive charge, \( W_{\text{TNT}} \) (kg) | Scaled distance, \( Z \) (m/kg\(^{1/3}\)) | Applied peak overpressure, \( P_{\text{OP}} \) (MPa) | Maximum mid-span deflection in concrete slab with conventional reinforcement on both faces (mm) | Error (%) |
|---------|---------------------------------|-----------------------------------|---------------------------------|---------------------------------|-----------|
|     |                                 |                                   |                                | Wu C. et al. (2009) experimental result | FEA result |
| NRC-1  | 1.00                            | 3.00                              | 0.13                            | 1.50                            | 1.55      | 3.40 |
| NRC-2  | 8.00                            | 1.54                              | 0.54                            | 10.50                           | 10.45     | 0.48 |
| NRC-3  | 3.44                            | 0.93                              | 1.43                            | 13.90                           | 13.05     | 6.09 |
| NRC-4  | 8.00                            | 0.75                              | 1.72                            | 38.90                           | 38.58     | 0.82 |

**Figure 4.** Distribution of cracks along the length of the slabs with conventional reinforcement on both faces: (a) NRC-3 (t=2.98 ms) and (b) NRC-4 (t=2.38 ms)

Finite element (FE) simulation indicates that the concrete slabs with 25% replacement of the conventional steel re-bars by the FRP re-bars of equivalent strength (D2-D4) suffered less damage and mid-span deflection than the slabs with reinforcement on both faces with Fe600 HYSD steel re-bars (C1) when subjected to similar air-blast loads which shows that the higher tensile strength of the FRP re-bars with the conventional steel re-bars provides a good combination of reinforcing material for the design of concrete structures exposed to explosive-induced blast loading (Table 5). Although, reduction of mid-span deflection and damage dissipation energy is maximum for 0.25 reinforcement
replacement ratio on both the faces of the slabs, yet 0.25 replacement ratio of the conventional reinforcement by equivalent CFRP reinforcement on the tension face only (D3-CF) is found to be more effective combination under the applied air-blast loading (Table-5).

Table 5: Summary of maximum mid-span deflections in RC slabs with 25% replacement of conventional reinforcement by equivalent FRP reinforcement (D2-D4)

| Combination No. | Maximum mid-span deflection in RC slab (mm) |
|-----------------|-------------------------------------------|
|                 | NRC-1 (3.0 m/kg\(^1\)) | NRC-2 (1.54 m/kg\(^1\)) | NRC-3 (0.93 m/kg\(^1\)) | NRC-4 (0.75 m/kg\(^1\)) |
| C1 (C & T: Fe600 HYSD steel re-bars) | C1-S | 1.55 | 10.45 | 13.05 (7703.35) | 38.58 (64070.65) |
| D2 (C: 75% Fe600 HYSD steel + 25% substituted FRP re-bars) | T: Fe600 HYSD re-bars | D2-AF | 1.28 (17%; 77.55) | 8.77 (16%; 521.17) | 11.42 (12%; 5465.83) | 35.79 (7%; 54897.36) |
| | | D2-BF | 1.39 (10%; 81.88) | 9.25 (11%; 743.13) | 12.12 (7%; 6105.40) | 36.93 (4%; 58041.84) |
| | | D2-CF | 1.15 (26%; 72.50) | 8.35 (20%; 639.04) | 11.09 (15%; 5226.59) | 33.40 (13%; 49169.84) |
| | | D2-GF | 1.42 (8%; 82.98) | 9.49 (9%; 866.42) | 12.54 (4%; 7097.91) | 37.80 (2%; 60979.25) |
| D3 (C: Fe600 HYSD steel re-bars) | T: 75% Fe600 HYSD steel + 25% substituted FRP re-bars | D3-AF | 1.17 (25%; 73.40) | 8.15 (22%; 365.72) | 10.85 (17%; 5356.06) | 34.56 (10%; 51849.03) |
| | | D3-BF | 1.30 (16%; 78.50) | 9.13 (13%; 690.09) | 11.79 (10%; 6162.89) | 35.76 (7%; 56097.02) |
| | | D3-CF | 1.06 (32%; 67.92) | 7.72 (26%; 292.88) | 10.41 (20%; 5146.06) | 31.86 (17%; 44765.41) |
| | | D3-GF | 1.39 (10%; 81.90) | 9.37 (10%; 805.57) | 12.20 (7%; 7252.83) | 36.96 (4%; 60272.28) |
| D4 (C & T: 75% Fe600 HYSD steel + 25% substituted FRP re-bars) | D4-AF | 1.08 (30%; 68.74) | 7.65 (27%; 275.77) | 10.56 (19%; 5059.54) | 33.98 (12%; 48197.77) |
| | | D4-BF | 1.23 (21%; 75.62) | 9.01 (14%; 556.68) | 11.62 (11%; 5908.92) | 35.77 (7%; 53403.93) |
| | | D4-CF | 0.94 (39%; 61.87) | 7.20 (31%; 239.00) | 9.98 (24%; 5526.77) | 31.42 (19%; 41318.59) |
| | | D4-GF | 1.39 (10%; 81.42) | 9.25 (11%; 658.29) | 11.98 (8%; 6371.33) | 36.80 (5%; 58311.98) |

**a** Percentage decrease in mid-span deflection (%)  **C**: Compression face  
**C**: Compression face

**b** Cumulative damage dissipation energy (J)

3.1. Crack propagation

In the current study, the damage in the concrete slabs has been observed in the form of concrete crushing, cumulative damage dissipation energy, and development of transverse flexural cracks, flexural-shear cracks, and longitudinal cracks. The finite element simulation showed that the flexural cracks first start appearing on the remote face in the flexural zone and propagate to the periphery of the slab along with the application of the air-blast load. No noticeable cracks have been observed in slab NRC-1 which is subjected to incident blast pressure of 0.13 MPa. Flexural cracks in the transverse direction have developed in slab NRC-2 as shown in Figure 6. Longitudinal and transverse flexural cracks have appeared in slab NRC-3 which is subjected to blast pressure of 1.43 MPa. Flexural-shear near the support of the slab NRC-4 causes the failure of concrete. Table-6 summarizes the observed damage levels of RC slabs. It can be noted from Table-6 that the decrease of the proximity factor (Z) could lead to a change of damage mode of the slab from localized failure (i.e. formation of cracks on the remote face) to globalized failure (i.e. crushing of concrete at supports). Referring to Table 7, longitudinal and transverse cracks of significant depth and number have formed
on both blast and remote faces of the slab NRC-4. The pattern of cracks on the remote face of the slab NRC-4 obtained using the CDP model with a 10 mm mesh size is shown in Figure 7. The use of GFRP re-bars as a 25% replacement of the conventional reinforcement on the tension face only increased the average depth of transverse cracks developed at mid-span on the remote face of the slab NRC-4; thus the concrete slab with reinforcement combination D3-GF gives the inferior performance against cracking (Table 7).

![Figure 5](image)

**Figure 5.** Maximum deflection – time histories: (a) NRC-1 (t=9.03 ms), (b) NRC-2 (t=4.76 ms), (c) NRC-3 (t=2.98 ms), and (d) NRC-4 (t=2.38 ms)

**Table 6:** Damage levels of RC slab under air-blast loading

| Slab ID | Scaled distance, Z (m/kg\(^{1/3}\)) | Applied peak overpressure, \(P_{OP}\) (MPa) | Damage level | Remark |
|---------|--------------------------------------|---------------------------------------------|--------------|--------|
| NRC-1   | 3.00                                 | 0.13                                        | Low          | No cracking |
| NRC-2   | 1.54                                 | 0.54                                        | Moderate     | Transverse flexural cracks on remote face only; No concrete crushing |
| NRC-3   | 0.93                                 | 1.43                                        | Moderate     | Formation of longitudinal and transverse flexural cracks on remote face only |
| NRC-4   | 0.75                                 | 1.72                                        | High         | Development of longitudinal and flexural cracks on both faces; Bending; Concrete crushing at supports |
3.2. **Comparison of results**

In the present investigation, the results obtained with 0.25 reinforcement replacement ratio of the conventional reinforcement by the equivalent FRP reinforcement have been compared with those computed by Anas M. S. et al. (2020) [3] with 0.50 replacement ratio of the conventional reinforcement by the equivalent FRP reinforcement on compression face, remote face, and both faces of the slab. The comparison of results is shown in Figure 8 to 10.

- The deflection performance of concrete slabs with 50% replacement of the conventional reinforcement by the equivalent FRP reinforcement is found better than the slabs with conventional reinforcement only, and with 25% replacement of the conventional reinforcement by the equivalent FRP reinforcement (see Figure 8). However, the slabs with 50% steel replacement level suffer from deeper flexural cracks developed at mid-span on the remote face than the slabs with a 25% steel replacement level (Figure 10).

- Although, reduction of mid-span deflection and damage dissipation energy is maximum for the 50% steel replacement level on both the faces of the slabs, yet 25% replacement level of the conventional reinforcement by the equivalent FRP reinforcement on the tension face only is found to be more effective reinforcement replacement combination (see Figure 8 & 9).
Figure 7. Pattern of cracks on the remote face of slab NRC-4 (t=2.38 ms) for different combinations

Figure 8. Comparison of mid-span deflection in RC slabs
Table 7: Geometric parameters of cracks in slab NRC-4 with 25% replacement of the conventional reinforcement by the equivalent FRP reinforcement

| Comb. No. | Average spacing of longitudinal cracks (mm) | Average spacing of transverse cracks (mm) |
|-----------|--------------------------------------------|-------------------------------------------|
|           | Blast face | Remote face | Blast face | Remote face |
|           | Quarter-span | Mid-span | Quarter-span | Mid-span | Quarter-span | Mid-span |
| C1-S      | 85          | -         | 80          | 85        | h           | -         | 35       | 87       |
| D2-AF     | 30          | -         | 90          | 85        | h           | -         | 25       | 57       |
| D2-GF     | 60          | 174       | 80          | 85        | h           | -         | 22       | 56       |
| D2-BF     | 45          | 10        | 80          | 85        | h           | -         | 10       | 71       |
| C3-GF     | 65          | 80        | 65          | 85        | h           | -         | 10       | 82       |
| C3-CF     | 80          | 10        | 64          | 74        | h           | -         | 17       | 74       |
| D4-BF     | 47          | 10        | 64          | 70        | h           | -         | 10       | 69       |
| D4-CF     | 80          | 10        | 64          | 70        | h           | -         | 10       | 83       |
| D4-GF     | 50          | 185       | 64          | 70        | h           | -         | 29       | 50       |

**Note:**
- Number of cracks developed
- Average depth of cracks (mm)
- Average depth of cracks (mm)

Figure 9. Comparison of damage dissipation energy of RC slabs
4. Conclusions
From the detailed numerical study conducted, the following conclusions are extracted.

a) The concrete slabs with 0.50 replacement ratio of the conventional reinforcement by the FRP reinforcement of equivalent strength have performed better than the slabs with conventional steel re-bars only, but not better than the slabs with 0.25 reinforcement replacement ratio with regards to the depth of transverse flexural cracks at mid-span on the remote face.

b) Although, reduction of mid-span deflection and damage dissipation energy is maximum for 0.50 reinforcement replacement ratio on both the faces of the slabs, yet 0.25 replacement ratio of the conventional reinforcement by equivalent FRP reinforcement on the tension face only is found to be more effective combination under the applied peak overpressures.

c) For the considered steel replacement levels, the concrete slabs with CFRP re-bars combined with Fe600 HYSD steel re-bars give outstanding performance, while the slabs with GFRP re-bars combined with Fe600 HYSD steel re-bars give the poorest performance under the air-blast loading considered.

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