Modelling of a Reinforced Concrete Panel Subjected to Blast Load by Explicit Non-linear FE Code

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Abstract

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A 5000 kg TNT blast test was conducted at Woomera, South Australia on a reinforced concrete panel. This paper reports on the numerical analysis of this 1.19m×2.19m×0.14m RC panel under the blast load. The peak reflected overpressure, peak reflected impulse, arrival time and blast duration are predicted by computer programs AIR3D and CONWEP. These blast pressure time histories are compared with the pressure measured by a pressure transducer attached to the test module. The dynamic structural behavior of the RC panel is modelled by the explicit numerical analysis code, LS-DYNA. The concrete model in LS-DYNA which considers three failure surfaces is adopted to model the concrete material for the panel. An elastic-perfectly plastic model represents the steel reinforcement in the panel. The maximum deflection of the panel from the explosive test is compared with the numerical analysis results. The comparison of the results obtained from numerical analysis and experiment demonstrates the capabilities of these software packages to simulate the structural behavior of a RC panel under explosion.

Keywords: Blast loading, RC panel, non-linear FE code

1. INTRODUCTION

A performance assessment of structural components for blast loading is now an important issue due to increasing terrorism attacks and accidental explosions. To understand the behavior of structures under this kind of loading, full scale blast tests are required. However, these tests are limited due to security restrictions and also the considerable resources required. Numerical analysis and computer modelling are now proven to be a valuable tool to simulate the behavior of such structures under blast loading.

This paper presents details of an experimental program performed at Woomera, South Australia. Observations from the test modules in a full scale blast trial conducted by the APTES group from the University of Melbourne are reported. The blast pressure time histories are investigated by using the computer programs CONWEP (1993) and AIR3D (Rose, 2006). An explicit non-linear finite element program, LS-DYNA (v.971, 2006) is utilised to simulate the behavior of a reinforced concrete panel. The concrete model used in this analysis is the latest version of *MAT_CONCRETE_DAMAGE (RELEASE III) (Crawford *et al.*, 2006). Comparisons of maximum and rebound deflections analysed by this concrete model are presented in this paper as well as the predicted compressive and tensile stresses in concrete and steel reinforcement.

2. WOOMERA BLAST TRIAL

A full scale blast test took place at the Woomera Prohibited Area, South Australia on 15^{th} March 2007. This blast test was conducted in accordance with regulations stipulated by the UK Ministry of Defence and operated by the Australian Department of Defence. A number of participants from the UK, USA, Singapore and Australian researchers/consultants and the University of Melbourne took part in this blast trial. Moreover, some commercial participants joined in this blast test to verify the performance of their products to resist blast pressure.

The Advanced Protective Technologies for Engineering Structures (APTES) group from The University of Melbourne tested two trial modules in this blast test event to better understand the behavior of precast reinforced concrete panels, glass panels and blast doors under high blast loading. Figure 1 shows the 5000 kg TNT charge used in this trial. The 1^{st} P trial module, UM1, standing 40m away from the explosion detonation point comprises of one conventional steel reinforced concrete panel, one wire-mesh

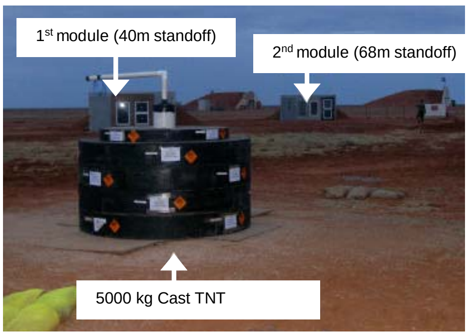

Figure 1. 5000 kg TNT explosive and two tested modules in site

reinforced concrete panel, one glass-polycarbonate composite and one blast resistant laminated glass panel as shown in Figure 2. The 2nd trial module, UM2, with a 68m standoff distance, as shown in Figure 3, was fitted with two types of blast doors and two types of glass panels. This paper focuses on the analysis and behavior of the conventional steel reinforced concrete panel in the 1st trial module.

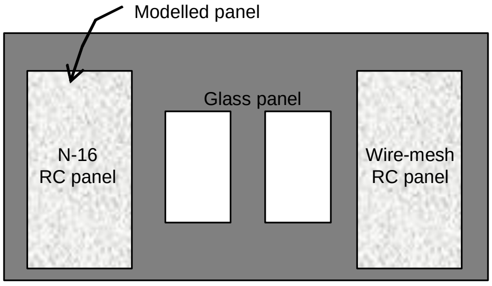

Figure 2. 1st P trial module (UM1)

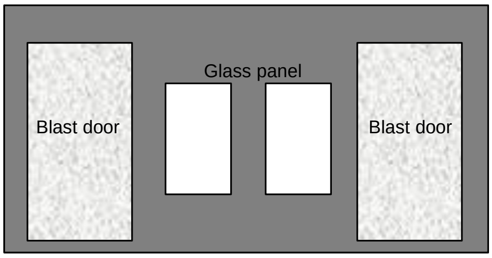

Figure 3. 2nd P trial module (UM2)

3. EXPERIMENTAL PROGRAM

The dimensions of the reinforced concrete panel and the steel arrangement are shown in Figure 4. A 40 MPa compressive strength of concrete was chosen to represent normal strength concrete used in general RC structures. N16 reinforcing bars@120 mm spacing with minimum yield strength of 500 MPa were distributed in two directions inside both the front and rear faces of the RC panel. Reinforcing steel inside both faces of the panel is necessary due to dynamic action effects in which these structures might rebound due to loading reversal. The tested panel was bolted with M16 bolts (100mm in length) and supported by two L-125×75×10 mm angle sections which were welded together with steel plates in the tested module (see Figure 4). So as to measure the blast pressure acting on the tested modules, one pressure transducer on the front face and another on the side face of the tested modules were installed. The maximum and rebound displacements of the tested panel were recorded by means of a mechanical device. This device consists of a main vertical steel hollow section connected to the base of the tested module and a measurement rod attached to this steel hollow section. To measure the rebound displacement, an aluminium plate was connected to the main steel hollow section with two steel wires embedded in the concrete panel as shown in Figure 5. These two steel wires pull an aluminium plate when the panel rebounds and this causes the aluminium plate to be deformed. The rebound deflection of the tested panel is assumed to be equal to the deflection of the aluminium plate.

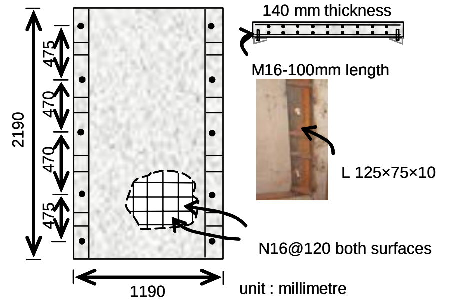

Figure 4. Details of tested RC panel and bolt connection

Figure 5. Deflection measurement device

4. EXPERIMENTAL RESULTS

As mentioned earlier, this paper aims to study the dynamic behavior of a reinforced concrete panel. Experimental results for the glass panel and blast door are not presented. The displacement measurement device recorded a maximum deflection for the RC panel of 36mm. By measuring deflection of the aluminium plate attached to the main measurement device, a 5 mm rebound of the tested panel was reported. Figures 6 and 7 show the observed minor cracks at the rear face and some spalling at the front face of the tested RC panel. It was believed that the localised concrete spalling at the front face of the tested panel resulted from higher pressure which takes place at the right edge rather than the left edge of the tested module due to clearing effects. This means that instead of concrete spalling at the centre of the panel where the pressure is lower than the pressure at the right edge of the tested panel, concrete spalling occurred at the right edge of the panel as shown in Figure 7. This local damage also might result from the fragment effects.

Top part of panel Bottom part of panel

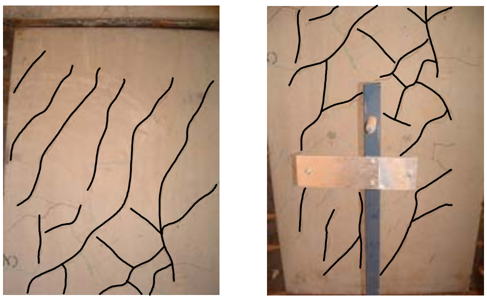

Figure 6. Observed crack pattern at rear face of tested panel

Figure 7. Observed spalling at front face of tested panel

5. BLAST PRESSURE PREDICTION AND MEASUREMENT

The blast pressure time histories predicted by an analytical model based on equations and curves of TM5-855-1, CONWEP (1993) and the Computational Fluid Dynamics (CFD) code, AIR3D (Rose, 2006) are presented and compared to the measured blast pressure in this section. Two pressure transducers were fixed into nylon mounts to the front and side faces of the 2nd module and connected to the data acquisition system. Unfortunately the data for the reflected pressure time history recorded at the front face was corrupted. The only incident pressure time history measured in this 2007 trial was at the side face of the 2nd module and is reported here. However, the APTES group possesses the data acquisition records of the reflected pressure obtained from an earlier blast trial in May 2006 (Gupta *et al*., 2006) with the same charge weight and at a 40m standoff distance which matches with the 1st trial module.

The AIR3D (Rose, 2006) program predicts the peak reflected overpressure and peak incident overpressure close to the corresponding values obtained from the experiment as shown in Tables 1 and 2, i.e. approximately 1% difference for peak reflected overpressure of the 1st trial module and 26% difference for the peak incident overpressure for the 2nd trial module. Due to limitations in capability of CONWEP (1993), the negative pressure time history is not reported. It should be noted that the impulse obtained from CONWEP (1993) is higher than the experimental corresponding value by about 60% for the incident impulse in the 2nd trial module and by approximately 100% of the reflected impulse for the 1st trial module. This significant difference in measured and CONWEP (1993) predicted impulses resulted from the infinite reflecting surface assumed by CONWEP (1993). In the experiment, blast pressure dissipated rapidly through the roof face of the tested module (see Figure 8) while blast pressure would spend more time to dissipate through the infinite reflecting surface in CONWEP (1993).

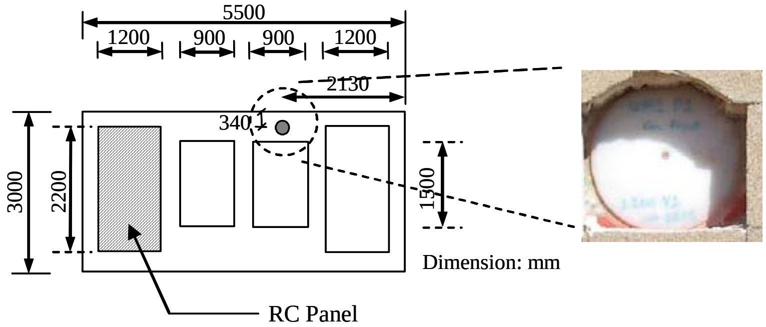

Figure 8. Dimension of the tested module and location of the pressure gauge

Table 1. Summary of wave-front parameters for 5000 kg TNT at standoff distance 40m (1st trial module)

	Peak reflected over pressure (kPa)	**Reflected impulse (kPa.msec)**	**Arrival time (msec)**
Experiment	694 May 2006 trial (Gupta *et al*., 2006)	2349 May 2006 trial (Gupta *et al*., 2006)	38 May 2006 trial (Gupta *et al*., 2006)
AIR3D (Rose, 2006)	699	3742	35
CONWEP (1993)	663	5147	39

Table 2. Summary of wave-front parameters for 5000 kg TNT at standoff distance 68m (2nd module)

	Peak incident over pressure (kPa)	**Incident impulse (kPa.msec)**	**Arrival time (msec)**
Experiment	91	776	95
AIR3D (Rose, 2006)	67	876	94
CONWEP (1993)	66	1238	98

Generally the predicted reflected and incident overpressure obtained from AIR3D (Rose, 2006) and CONWEP (1993) agree well with the corresponding pressure values obtained from the experiment as shown in Figures 9 and 10.

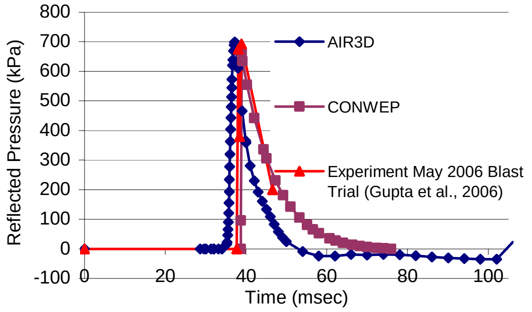

Figure 9. Reflected pressure time history of 5000 kg TNT at a 40m standoff distance

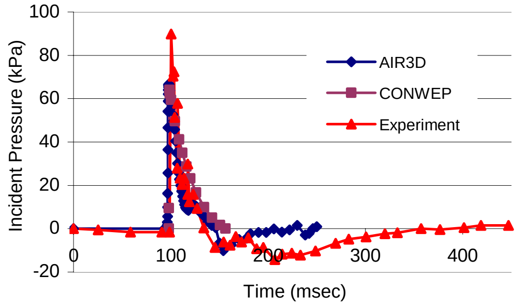

Figure 10. Incident pressure time history of 5000 kg TNT at a 68m standoff distance

6. NON-LINEAR FINITE ELEMENT ANALYSIS

The numerical analysis of the reinforced concrete panel subjected to blast loads is presented in this section. A simulation of the dynamic behavior of the 1.19m×2.19m×.14m panel was performed by using commercial software, LS-DYNA (v.971, 2006). The concrete model (Crawford *et al.*, 2006) (MAT_CONCRETE_DAMAGE_REL3) which considers three failure surfaces, i.e. initial yield failure surface, maximum failure surface and residual failure surface, is used to model concrete. A total of eight parameters in Equations (1), (2) and (3) define the three failure surfaces. It can be seen that stress difference at each failure surface depends on pressure in a particular element. In this analysis, automatic input data generation was employed which yields the values of surface parameters as listed in Table 3.

$$\Delta\sigma_m = a_0 + \frac{p}{a_1 + a_2 p} \qquad \text{(maximum failure surface)} \qquad (1)$$

$$\Delta\sigma_r = \frac{p}{a_{1f} + a_{2f} p} \qquad \text{(residual failure surface)} \qquad (2)$$

$$\Delta\sigma_y = a_{0y} + \frac{p}{a_{1y} + a_{2y} p} \qquad \text{(yield failure surface)} \qquad (3)$$

where $\Delta\sigma$ defines the difference in the principal stresses and p is the pressure in an element

Table 3. Three failure surfaces parameters obtained from automatic input data generation (pressure unit : N/m^2)

Maximum failure surface	Residual failure surface	Yield failure surface
$a_0 = 1.182\times10^7$ $a_1 = 0.4463$ $a_2 = 2.02\times10^{-9}$	$a_{1f} = 0.4417$ $a_{2f} = 2.958\times10^{-9}$	$a_{0y} = 8.928\times10^6$ $a_{1y} = 0.625$ $a_{2y} = 6.438\times10^{-9}$

The steel reinforcement is modeled by using MAT_PLASTIC_KINEMATIC. In this study, the authors adopted an elastic-perfectly plastic material model to represent the steel reinforcement. A total of 2880 eight-node solid elements and 1328 two-node truss elements were created to construct the concrete and reinforcement meshes in this FE model respectively. In this analysis, the full bond behavior of reinforcement and concrete is assumed by sharing nodes between concrete and reinforcement elements. This FE model consists of 3887 nodes. The boundary conditions of supports were achieved by restraining the translation in the x- and y-directions at nodes located at the positions of the centre line of L-125×75×10 mm in the tested RC panel. By performing sensitivity analyses, four layers of solid elements through the panel thickness direction were found to be efficient to capture the bending behavior of the 140 mm RC panel under blast loading.

By using the blast impulse obtained from CONWEP (1993), the ratio of time to reach maximum displacement and blast positive phase duration (t_m/t_d) is around 0.47. This value corresponds to the pressure loading regime instead of impulsive loading regime. Even though the impulse reported by CONWEP (1993) is much higher than the measured impulse, this does not significantly affect the behavior of the panel under the blast pressure loading regime. Therefore, the built-in CONWEP (1993) blast pressure in LS-DYNA (v.971, 2006) was applied to the FE model with reasonable confidence.

The finite element model adopted for the panel is illustrated in Figure 11. LS-DYNA (v.971, 2006) reports the maximum and rebound deflections of the tested panel as 30mm and 4mm respectively which agrees well with the corresponding values of 36 mm and 5 mm recorded from the experiment (see Figures 12 and 14). Figures 15 and 16 show contours of the damage parameter of concrete at the front and rear faces of the analysed panel. This damage parameter ranges from 0 to 2 based on the three failure surfaces as mentioned earlier. The damage values from 0 to 1 indicate the progression from the initial yield surface to the maximum failure surface. Likewise, damage values from 1 to 2 show the progression from the maximum failure surface to the residual failure surface.

The crack pattern at the rear face of the tested panel, as can be seen in Figure 6, shows that most of the cracks are vertical cracks which are consistent with the concrete damage contours obtained from the analysis (see Figure 16).

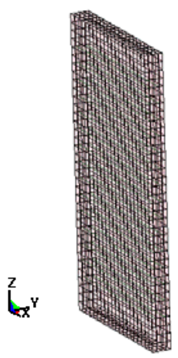

Figure 11. FE model (at t=0 msec)

Figure 12. Deformed shape when maximum deflection reaches (t=66 msec)

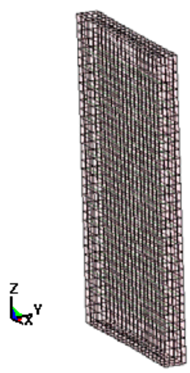

Figure 13. Deformed shape when rebound deflection reaches (t=130 msec)

The maximum concrete compressive stress of 23.8 MPa is reported which represents approximately 60% of the concrete compressive strength (see Figure 18). In addition, a maximum tensile stress in the concrete of 3.5 MPa (see Figure 19) is reached before the tested panel reaches maximum deflection at time 66 msec. The concrete model (Crawford *et al.*, 2006) used in this analysis adopts the tensile strength of concrete calculated from CEB-FIP (1990). The calculated concrete tensile strength, based on CEB-FIP (1990), is 3.55 MPa which is close to the maximum concrete tensile stress reported by LS-DYNA (v.971, 2006). Figure 20 shows maximum tensile and maximum compression stress in the steel reinforcement. It should be noted that only 300 MPa of tensile stress and 200 MPa of compression stress have been reached in the N-16 reinforcement. These clearly signify that only 40% - 60% of the yield stress is realised in the blast.

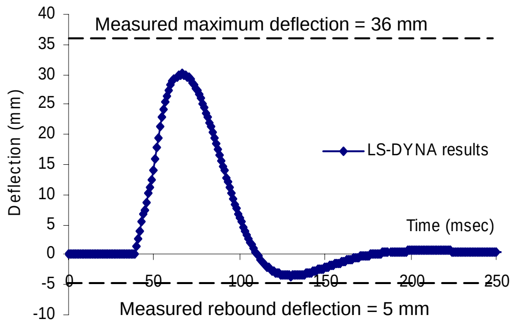

Figure 14. Maximum and rebound deflections obtained from numerical analysis compared to the observed values.

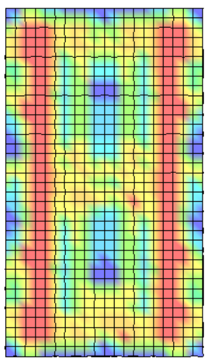

Figure 15. Contour of damage parameter at front face (t=300msec)

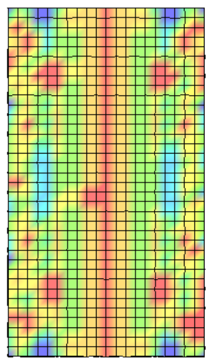

Figure 16. Contour of damage parameter at rear face (t=300msec)

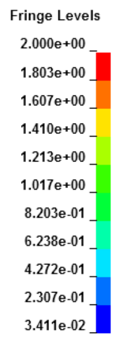

Figure 17. Fringe level of damage parameter

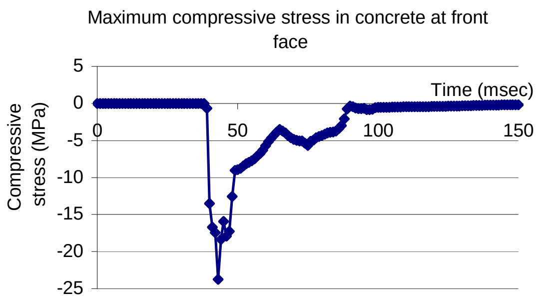

Figure 18. Maximum compressive stress in concrete at front face obtained from LS-DYNA (v.971, 2006)

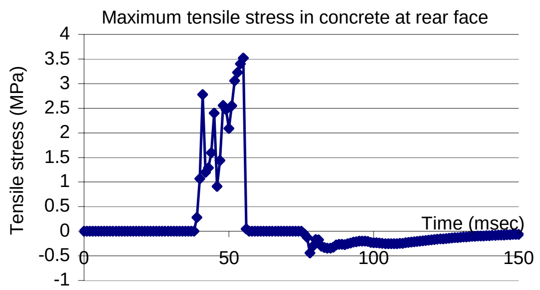

Figure 19. Maximum tensile stress in concrete at rear face obtained from LS-DYNA (v.971, 2006)

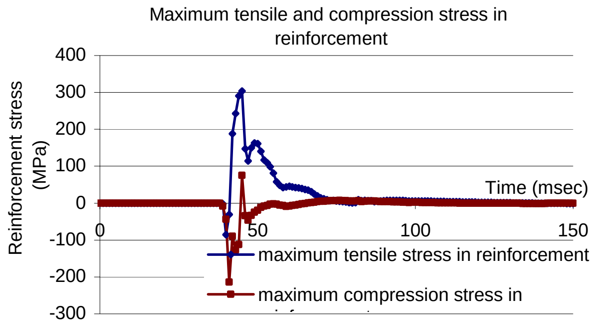

Figure 20. Maximum tension and compression stress in reinforcement obtained from LS-DYNA (v.971, 2006)

7. CONCLUDING REMARKS

This paper reports results from a recent blast test performed on a reinforced concrete panel. Both experimental and numerical analysis results are presented. Under a blast from a 5000 kg TNT explosive at a 40m standoff distance, a 140mm thickness RC panel was found to be lightly damaged with minor cracks. However there was some spalling at the edge on the front face of the concrete panel. Blast pressure time histories obtained from CONWEP (1993) and AIR3D (Rose, 2006) generally agree with the pressure measured in the experiment. This study shows that the behavior of a RC panel under blast loads can be simulated by the explicit non-linear finite element program, LS-DYNA (v.971, 2006). By using this FE analysis code with selected material models, *MAT_CONCRETE_DAMAGE_REL3 (Crawford *et al.*, 2006) and *MAT_PLASTIC_KINEMATIC, and inbuilt CONWEP (1993) blast pressure, the dynamic behavior of the tested RC panel could be predicted to a reasonably acceptable accuracy. The FE model shows only a 17% difference in maximum deflection in the tested panel compared to the observed value. Only 40% and 60% of the yield stress in the reinforcement in compression and tension respectively was found to be realised from this modelling.

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