Research Paper

Materials for Sacrificial Blast Wall as Protective Structure

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Investigation on the performance of sacrificial blast walls subjected to blast loading is presented here. Three dimensional nonlinear dynamic analyses of sacrificial blast walls under blast loading have been performed using finite element software Abaqus 6.11. Sacrificial blast walls of different thicknesses made up of steel, plain, reinforced and steel fibre reinforced concrete have been examined under blast loading. Strain rate dependent properties of all the materials have been used in the analyses. Blast load is applied in the form of pressure-time history curve calculated using the TM5-1300 manual and the modified Friedlander’s equation. Stress and peak displacement response of the sacrificial blast walls have been studied. It is observed that the sacrificial blast walls made up of steel fibre reinforced concrete show lesser stress and peak displacement, and thus higher blast response reduction capability.

Key Words: Blast; Sacrificial Blast Wall; Steel Fibre Reinforced Concrete; Strain Rate

1. Introduction

The necessity of constructing blast resistant and protective structures to safeguard the civil infrastructure has increased with rising terrorist activities all over the world. Unstiffened and stiffened steel and reinforced concrete sacrificial blast walls and sandwich structures which dissipate large amount of energy by plastic deformation under blast loading are the types of protective structures used against blast loading [1-4]. Several experimental and numerical studies have been reported on steel plates, and concrete structures under blast loading [5-8]. However, comparative performance of sacrificial blast walls made up of different materials under blast loading is not available in the literature.

Herein, the objective is to compare the performance of different materials used for sacrificial blast walls as protective structure against blast induced impulsive loading. The stress and peak displacement responses of the sacrificial blast walls are studied through three dimensional (3D) nonlinear finite element (FE) analysis using Abaqus software [9]. Numerical investigations are performed for (a) steel plates, (b) plain concrete (PC), (c) reinforced concrete (RC) and (d) steel fibre reinforced concrete (SFRC) slabs. Fig. 1 shows all the analyses conducted in the present study. Parametric studies have been performed using three different thicknesses ($t_p$) of 100, 150 and 200 mm for PC, RC, SFRC slabs. Table 1 summarises the different cases of parametric investigations conducted in the present study.

Three Dimensional Finite Element Modelling

Herein, steel plates of thickness ($t_p$) 20 mm have been modelled. The concrete slabs are modelled using M25 grade of concrete (with considered compressive strength of 25 MPa) in the PC, RC and SFRC slabs. The thicknesses of these slabs are given in Table 1.

The RC panels have been modelled with 0.25% steel reinforcement and 10 mm diameter steel reinforcing bars. The blast loading is applied on the side opposite

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Constitutive models have been used for all the materials in the present investigations. Table 2 summarises the physical properties of all materials, the material constitutive models used in the present study and the constitutive model parameters. The steel plates and the stiffeners are made up of steel with Young’s modulus, $E = 210$ GPa; Poisson’s ratio, $\nu = 0.3$; and density, $\rho = 7800$ kg/m$^3$. The rate dependent elastic-plastic behaviour of steel is defined by the Johnson-Cook plasticity constitutive model [11]. The strain rate dependent stress-strain response of the model is given by

$$\sigma = (A + B\varepsilon^m)(1 + C \log \dot{\varepsilon})(1 - T^m)$$

where $\varepsilon^* = \dot{\varepsilon}/\dot{\varepsilon}_0$ is the dimensionless plastic strain rate at reference strain rate $\dot{\varepsilon}_0 = 1$/sec, $\dot{\varepsilon}$ is the equivalent plastic strain rate, and $T^*$ is the homologous temperature. Parameters $A$, $B$, $C$, $m$ and $n$ are the material constants as described in Table 2, which are taken from [3]. The effects of temperature are ignored in the present analyses.

The PC slabs are modelled using the strain rate dependent concrete damaged plasticity material model. For the RC slabs, the material behaviour of concrete is characterised by the strain rate dependent concrete damaged plasticity model, and the material behaviour of the steel reinforcement bars has been characterised using the strain rate dependent Johnson-Cook plasticity model. The SFRC is modelled as an equivalent continuum in the present study using the strain rate dependent concrete damaged plasticity model. The yield surface equation of the concrete damaged plasticity model is given by

$$F = \left(\sqrt{3/2}\sqrt{s_1} : S_1\right) - 3\alpha \tilde{\sigma} + \beta \left(\tilde{\sigma}_{\text{max}}\right)^3 - \gamma \left(-\tilde{\sigma}_{\text{max}}\right) - (1 - \alpha)\tilde{\sigma} = 0$$

where $\tilde{\sigma}$ is the equivalent stress, $\tilde{\sigma}_{\text{max}}$ is the maximum stress, $\alpha$, $\beta$ and $\gamma$ are material constants, and $s_1$ is the deviatoric stress.
Table 2: Mechanical properties for different materials

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Material description</th>
<th>Mass density kg/m³</th>
<th>Young’s modulus GPa</th>
<th>Poisson’s ratio</th>
<th>Material model (Strain rate dependent)</th>
<th>Material model parameters</th>
<th>References</th>
</tr>
</thead>
</table>
| 1     | Steel                                                                                | 7800                | 210                 | 0.3            | Johnson-Cook                           | \( A = 360 \text{ MPa}, \)  
\( B = 635 \text{ MPa}, \)  
\( n = 0.114, C = 0.075 \)                                                                             | [3]                     |
| 2     | Plain Concrete (M25)                                                                 | 2643                | 31                  | 0.2            | Concrete damaged plasticity             | 36°,  
\( \sigma_{y,\text{yield}} = 0.55 \text{ MPa}, \)  
\( \sigma_{y,\text{yield}} = 0.52 \text{ MPa} \)                                                             | [12]                    |
| 3     | Reinforced concrete (M25, with 0.25% reinforcement, 10 mm diameter steel reinforcement bar) | 2643                | 31                  | 0.2            | Concrete damaged plasticity             | 36°,  
\( \sigma_{y,\text{yield}} = 0.55 \text{ MPa}, \)  
\( \sigma_{y,\text{yield}} = 0.52 \text{ MPa} \)                                                             | [3], [12]               |
| 4     | Steel fibre reinforced concrete (3% steel fibre by volume)                           | 4583                | 42                  | 0.2            | Concrete damaged plasticity             | 36°,  
\( \sigma_{y,\text{yield}} = 1.0 \text{ MPa}, \)  
\( \sigma_{y,\text{yield}} = 1.0 \text{ MPa} \)                                                             | [9], [13], [14]         |

where

\[
\alpha = \frac{(\sigma_{60}/\sigma_{o})}{2(\sigma_{60}/\sigma_{o})} - 1
\]

\[
\beta = \frac{\overline{\sigma}}{\overline{\sigma}_{1}}(1 - \alpha) - (1 + \alpha)
\]

\[
\gamma = \frac{3(1 - K_{c})}{2K_{c} - 1}
\]

\[
\overline{\sigma}_{c} = \frac{\sigma_{c}}{(1 - d_{1})}
\]

\[
\overline{\sigma}_{1} = \frac{\sigma_{1}}{(1 - d_{1})}
\]

In equations (2) to (7), all quantities under bar ("\(^\wedge\)"") represent the magnitude of the same quantities considering damage. Here, \( \overline{\sigma}_{y} \) is the deviatomic stress tensor; \( \overline{\sigma}_{\text{max}} \) is the maximum principal effective stress; \( \sigma_{60} \) is the initial equibiaxial compressive yield stress; \( \sigma_{o} \) is the initial uniaxial compressive yield stress; \( d_{1} \) is the damage variable; and \( K_{c} \) is the ratio of the second deviatoric stress invariant on the tensile meridian to that on the compressive meridian at initial crushable state for any given value of effective mean stress, \( \overline{\sigma} = (\overline{\sigma}_{1} + \overline{\sigma}_{2} + \overline{\sigma}_{3})/3 \), where \( \overline{\sigma}_{1} \), \( \overline{\sigma}_{2} \) and \( \overline{\sigma}_{3} \) are the effective principal stresses. The Macaulay bracket \( \langle \cdot \rangle \) in equation (2) signifies that the quantities within the bracket will take either positive or zero value.

The model follows a non-associated flow rule with the plastic potential surface \( (G_{p}) \), different from the yield surface \( (F) \), given by

\[
G_{p} = \sqrt{\left(\varepsilon \sigma_{o} \tan \psi \right)^{2} + \left[ \frac{3}{2} \frac{\overline{\sigma}}{\overline{\sigma}_{y}} : \overline{\sigma}_{y} \right] - \overline{\sigma} \tan \psi
\]

where \( \psi \) is the dilation angle measured in the mean stress-deviatoric stress plane; \( \sigma_{0} \) is the uniaxial tensile stress at failure; and \( \varepsilon \) is the eccentricity parameter. The compressive and tensile stress-strain curves of concrete at different strain rates and the evolution of damage with strain are obtained from the literature and added in the model as input.

Fig. 3(a) shows the concrete stress-strain curves under rate independent and strain rate dependent loading which are considered in the present study. For PC, Young’s modulus, \( E = 31 \text{ GPa} \); Poisson’s ratio,
\[ \nu = 0.2; \ \rho = 2643 \text{ kg/m}^3; \ \text{dilation angle, } \psi = 36^\circ; \ \text{compression yield strength, } \sigma_{c, \text{yield}} = 0.55 \text{ MPa}; \ \text{and tensile yield strength, } \sigma_{t, \text{yield}} = 0.52 \text{ MPa are assumed. The rate dependent stress-strain curves for concrete are obtained from [12]. The plain concrete material behaviour is characterised by the strain rate dependent concrete damaged plasticity model with Young’s modulus, } E = 31 \text{ GPa; Poisson’s ratio, } \nu = 0.2; \ \text{and density, } \rho = 2643 \text{ kg/m}^3. \ \] 

The material parameters of concrete, e.g., dilation angle, \( \psi = 36^\circ; \) compression yield strength, \( \sigma_{c, \text{yield}} = 0.55 \text{ MPa}; \) tensile yield strength, \( \sigma_{t, \text{yield}} = 0.52 \text{ MPa; and the rate dependent stress-strain curves for concrete are obtained from [12] and are added in the model as input. The stress-strain curves of the SFRC under compression and tension loading are obtained from [13] and [14] with Young’s modulus, } E = 42 \text{ GPa; Poisson’s ratio, } \nu = 0.2; \ \text{and density, } \rho = 4583 \text{ kg/m}^3. \]

Fig. 3(a) and 3(b) show the strain rate dependent stress-strain curves for PC and SFRC.

**Calculation of Blast Load**

Blast load is applied in the present study as pressure time history curves on the plates, slabs and panels. All the analyses have been performed for blast scaled distance, \( Z = R/\sqrt[3]{W} = 0.5 \text{ m/kg}^{1/3} \), where \( R \) is the distance of explosive from the target and \( W \) is the mass of explosive. The blast pressure time history curve for the scaled distance, \( Z = 0.5 \text{ m/kg}^{1/3} \) is shown in Fig. 4. The peak positive blast pressure is obtained from TM5-1300 US Army manual [15] as 1.16 MPa. The exponential decay in the blast pressure and the negative blast pressure are calculated using the modified Friedlander’s equation [3]. In the modified Friedlander’s equation, shown in Fig. 4, \( P_{w}^{1/2} \) is peak overpressure in MPa; \( t_0 \) is arrival time in millisecond; \( t_0 \) is positive pressure wave duration in millisecond; and \( b \) is dimensionless wave decay coefficient expressed as \( \log_e \left( b \left| P_{w1} / P_{w2} \right| \right) + b + 1 = 0. \)

Here, \( P_{w1} \) denotes negative pressure expressed in MPa. The finite element analyses are performed using Abaqus 6.11 with explicit central difference numerical integration algorithm. An automatic time increment estimator with global stable increment without any time scaling factor is used in the analyses.

**Fig. 4: Blast load profile**
2. Simulation Results and Discussion

The in-plane stress ($\sigma = \sigma_2$) response in the plane of the sacrificial blast walls and the peak displacement ($\Delta_r$) of the walls in the direction of blast loading have been studied in the present investigation. Tables 3 and 4 summarise the peak stresses and the peak displacements for all the cases. The peak stress magnitude in steel plate is 788.324 MPa whereas the peak deformation magnitude in steel plate is 56.3 mm. The stress time history has been studied for the steel plate as shown in Fig. 5. It is observed that the steel plate exhibits high amount of oscillation in the stress time history curve.

Table 3: Peak stress at the central element of the plate and slabs

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Analysis cases</th>
<th>Thickness $t_p$ (mm)</th>
<th>Peak displacement at the central node of the plate and slabs, $\sigma_r$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Steel</td>
<td>20</td>
<td>788.324</td>
</tr>
<tr>
<td>2</td>
<td>PC</td>
<td>100</td>
<td>83.283</td>
</tr>
<tr>
<td></td>
<td></td>
<td>150</td>
<td>65.796</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200</td>
<td>38.889</td>
</tr>
<tr>
<td>3</td>
<td>RC</td>
<td>100</td>
<td>73.283</td>
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<tr>
<td></td>
<td></td>
<td>150</td>
<td>65.130</td>
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<tr>
<td></td>
<td></td>
<td>200</td>
<td>38.030</td>
</tr>
<tr>
<td>4</td>
<td>SFRC</td>
<td>100</td>
<td>90.190</td>
</tr>
<tr>
<td></td>
<td></td>
<td>150</td>
<td>35.844</td>
</tr>
<tr>
<td></td>
<td></td>
<td>200</td>
<td>27.818</td>
</tr>
</tbody>
</table>

Table 4: Peak displacement at the central node of the plate and slabs

<table>
<thead>
<tr>
<th>S.No.</th>
<th>Analysis cases</th>
<th>Thickness $t_p$ (mm)</th>
<th>Peak displacement at the central node of the plate and slabs, $\Delta_r$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Steel</td>
<td>20</td>
<td>56.3</td>
</tr>
<tr>
<td>2</td>
<td>PC</td>
<td>100</td>
<td>48.0</td>
</tr>
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<td></td>
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<td>150</td>
<td>10.6</td>
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<tr>
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<td>RC</td>
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<td></td>
<td>200</td>
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<tr>
<td>4</td>
<td>SFRC</td>
<td>100</td>
<td>16.5</td>
</tr>
<tr>
<td></td>
<td></td>
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</tr>
<tr>
<td></td>
<td></td>
<td>200</td>
<td>2.1</td>
</tr>
</tbody>
</table>

Fig. 5: Stress time history for steel plates

Fig. 6(a) shows the peak in-plane stress and peak deformation plots for 150 and 200 mm thick PC, RC and SFRC slabs. The trend of peak stress and peak displacement follows each other for both the cases. However, from Tables 4 and 5, it may be noted that the trend is not same for 100 mm thick slab; in which, the highest stress and the lowest peak displacement is observed in the SFRC slab. The stress time history is studied for all three slabs with three thicknesses as shown in Fig. 6(b). The stress time history curves show that the 100 mm thick PC and
RC slabs fail to take the blast load after reaching 83.283 and 73.283 MPa stresses, respectively whereas the 100 mm thick SFRC slab shows oscillatory stress response and can carry more stress as compared to the PC and RC slabs. The peak stress observed in the 100 mm thick SFRC slab is 90.91 MPa. For 150 and 200 mm thick slabs, PC and RC slabs do not fail which is observed from continual oscillations that are not seen in case of the 100 mm thick slab. However, the 150 and 200 mm thick PC and RC slabs exhibit higher stress and higher peak displacement as compared to the SFRC slab of the same thicknesses.

Evidently, the higher stresses in the PC slab as compared to the RC and SFRC slabs are due to the absence of reinforcement in the PC slabs. In the RC slab, reinforcement is located opposite to the loading face of the slab. On the other hand, in the SFRC slabs, steel fibre reinforcement is distributed in the slab. As a result, the SFRC slab shows higher strength and stiffness as compared to the PC and RC slabs, as also reported in [10] and [13]. The lower stress in the SFRC slabs may be attributed to higher strength and stiffness and higher mass density of these slabs. The absence of oscillation in stress in 100 mm thick RC and PC slabs is due to the damage of these slabs under blast loading. Such damage of slabs is not observed in the case of SFRC slabs under the blast loading.

It may be summarised from all the figures that under the same blast load, the SFRC slabs have the lowest peak stress induced followed by the RC and PC slabs. Steel plates show higher stress and displacement than that in PC, RC and SFRC slabs. For better understanding, a bar chart summarising the peak stresses has been presented in Fig. 7. The 200 mm thick SFRC slab shows the lowest peak stress of 27.818 MPa. From these plots, it may be concluded that among the slabs and plates, the SFRC slabs provide higher blast response reduction as compared to the conventional PC and RC slabs and steel plates.

3. Conclusions

The performance of sacrificial blast walls made up of steel plates and reinforced and unreinforced concrete slabs under blast loading is investigated in the present study. The effects of thicknesses of slabs have also been investigated. The following conclusions are drawn from the investigation:

1. The steel plate exhibits high amount of oscillation in the stress time history curve. The stress generated in steel plate is higher than that in PC, RC and SFRC slabs.

2. The stress time history curves show that the 100 mm thick PC and RC slabs fail to take the blast load. The absence of oscillation in stress in the 100 mm thick PC and RC slabs is due to the damage of these slabs under the blast loading. Such damage of slabs is not observed in the case of SFRC slabs under the blast loading. Hence, the SFRC slabs may be more effectively used in blast response reduction.

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