CHAPTER 4

EVALUATION OF STEEL-CONCRETE COMPOSITE (SCC)
CONSTRUCTION UNDER BLAST LOADING

4.1 INTRODUCTION

Laced Reinforced Concrete (LRC) has proven performance against blast loading. Steel-Concrete Composite (SCC) can be considered as an alternative material in view of complex detailing requirements in LRC storage structures particularly at the joints and also concreting poses problems. In this chapter, the performance of SCC under blast loading is studied using a simplified model to see the suitability of SCC as an alternate material to LRC.

The performance of a SCC composite structure is dependent upon efficient interaction and transfer of stresses between the steel plates and the concrete core. This can be achieved using mechanical shear connectors which prevent vertical separation in addition to longitudinal slip. Types of shear connectors used for sandwich composite construction include headed shear studs, angle shear connectors and mechanically fixed through connectors such as through-through connectors, J-hook connectors.

Conventional headed stud construction shown in Figure 4.1, is the one in which the studs are welded to the steel plates before concrete is
cast. The resistance of the two face plates against tensile separation depends on the pull out strength of the headed studs. The conventional headed shear studs are installed on the steel plate and thus there is no restriction on the core thickness and thus making the casting of concrete easier.

SCC with through-through connectors as shown in Figure 4.2 are the one in which steel round bar is rotated at high speed and opposite external force is applied to the face plates generating frictional heat that fuse the bar and the plates together. The bar connectors provide direct connection to the two face plates allowing effective shear transfer even without the presence of a concrete core. This system can only be fabricated in a factory environment, which reduces site work and improves the quality of the construction. The disadvantage of such method is that the core thickness must not be too thin (≤200mm) to restrict the placement of the steel rod connectors.

![Figure 4.1 Conventional headed shear connector](image)

![Figure 4.2 SCC system with through-through connector](image)
The restriction in core thickness in bi-steel connector has lead to development of a slim and lightweight SCS system J-hook connectors which is proposed as a means to connect the two face plates as shown in Figure 4.3. The minimum core thickness can be as thin as 50 mm. This connection technology together with the use of lightweight concrete core would reduce the overall weight of SCS system making it a competitive choice for marine and offshore structures.

![Figure 4.3 SCC system with J-hook connectors](image)

**Figure 4.3 SCC system with J-hook connectors**

### 4.2 FINITE ELEMENT MODELING APPROACH

Finite element (FE) method is a widely used numerical technique for blast analysis. Conventional approach of modeling SCC panel is to employ solid elements to discretise all the components, namely steel, concrete and shear connector. This modeling approach results in large number of degrees of freedom (DOF) due to the complex nature of geometry. This poses more demand on modeling requirements.

#### 4.2.1 Simplified approach

A simplified approach is proposed for modeling SCC panels. Based on the characteristics of the components, appropriate elements are identified to represent them. Concrete core, cover plates and shear
connectors are idealized using solid, plate and link elements, respectively. Solid elements are eight-noded hexahedral (3D) elements with 3 translational DOF per node. Plate elements are four-noded quadrilateral (2D) elements with 6 DOF per node. Link element is uniaxial (1D) element with 3 DOF per node. This reduces the number of DOF. Interface between concrete and steel plates is modelled using contact pair. Contact and target surfaces constitute a “Contact Pair”. This contact pair takes care of the compatibility between solid and shell elements at their interfaces. Contact element is located on the surface of plate elements called underlying element. It has the same geometric characteristics as the underlying elements. Target surface is concrete surface facing the plate as shown in Figure 4.4. In this study, augmented Lagrangian method of contact algorithm is adopted. The augmented Lagrangian method is an iterative series of penalty updates to find the Lagrange multipliers, i.e. contact tractions. Contact detection points are the integration point and are located at Gauss points. Friction model adopted in this study is Coulomb’s friction model. In this model, two contacting surfaces can carry shear stresses up to a certain magnitude, $\tau_{\text{lim}}$ across their interfaces before they start sliding relative to each other. Coulomb friction model is defined as

$$\tau_{\text{lim}} = \mu P \quad (4.1)$$

$$|\tau| \leq \tau_{\text{lim}}$$

where

$\tau_{\text{lim}}$ = limit shear stress,

$\tau$ = equivalent shear stress,

$\mu$ = coefficient of friction,

and $P$ = contact normal pressure
4.2.2 Numerical Validation

Validation of the proposed simplified approach is carried out by analysing two SCC structural components. In the first example, a SCC beam is subjected to monotonic loading. Load-deflection response of the beam obtained from the FE analysis and experiments are compared. In order to ascertain that the proposed approach can be used for dynamic response as well, a SCC panel subjected to air-blast loading is analysed. Numerical results are verified with that of an analytical model.

4.2.2.1 Static response

The proposed simplified approach is validated by analysing a SCC beam with through-through connectors subjected to static load. Overall length of the beam is 2.2 m, while width and depth of the beam are 400 mm and 200 mm, respectively. The compression and tension plate thicknesses are 11.93 mm and 6.2 mm, respectively. Connectors of 25 mm diameter are equally spaced at 300 mm c/c between the simply supported span of 1.8 m. The beam contains two rows of connectors spaced at 200
mm c/c. Characteristic compressive strength of concrete used is 40 MPa. Properties of steel used for plate and connector are summarized in Table 4.1. Multi-linear material model is used to represent the behaviour of concrete, while steel behaviour is represented by using bilinear stress-strain curve. Beam is subjected to central concentrated load. Details of the beam are shown in Figure 4.5.

<table>
<thead>
<tr>
<th>Component</th>
<th>Yield Strength, MPa</th>
<th>Ultimate Strength, MPa</th>
</tr>
</thead>
<tbody>
<tr>
<td>Plate</td>
<td>384</td>
<td>507</td>
</tr>
<tr>
<td>Connector</td>
<td>541</td>
<td>566</td>
</tr>
</tbody>
</table>

Figure 4.5 Details of Steel-Concrete Composite Beam

Two approaches are used to model the beam. In the first approach, solid elements are used to represent the entire composite beam, while in the second approach, proposed simplified model is used. Surface to surface contact is applied on interfaces between steel and concrete. Nonlinear static analysis is carried out to obtain the load-deflection response.
Responses from both approaches are compared with that of the experimental results available in literature as shown in Figure 4.6. Results of the solid model are found to be in close agreement with that of experiments up to yield load. Stiffness of the load-deflection curve obtained by using the simplified approach is found to be less than that of experiment. Yield load, ultimate load, yield and ultimate deflections predicted by the solid model, simplified model from experiments are shown in Table 4.2. Difference between prediction of ultimate load and deflection and experimental value is less than 10%, while yield load and deflection are predicted with about 16% difference. From the results, it can be seen that simplified approach is found to be computationally efficient, without losing the accuracy in prediction of the responses of steel-concrete composite panel.

\[\text{Figure 4.6 Load-deflection response of SCC beam}\]
### Table 4.2 Comparison of response of SCC beam

<table>
<thead>
<tr>
<th></th>
<th>Experiment</th>
<th>Solid model</th>
<th>Simplified model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield load, kN</td>
<td>333</td>
<td>360</td>
<td>280</td>
</tr>
<tr>
<td>Ultimate load, kN</td>
<td>545</td>
<td>616</td>
<td>530</td>
</tr>
<tr>
<td>Yield deflection, mm</td>
<td>7.1</td>
<td>8.44</td>
<td>9.02</td>
</tr>
<tr>
<td>Ultimate deflection, mm</td>
<td>46.25</td>
<td>52.59</td>
<td>50.24</td>
</tr>
</tbody>
</table>

### 4.2.2.2 Dynamic response

To ensure the applicability of the simplified approach for blast response analysis, a SCC panel subjected to air blast loading is solved by conducting finite element analysis. Results from the analysis are compared with that obtained using an analytical model proposed by Coyle and Cormie (2009).

Dimensions of steel-concrete composite panel are 2 m x 2 m. The panel is simply supported on all four sides. Through-through connectors of diameter 16 mm are provided at a spacing of 200 mm c/c in both directions. Concrete core thickness is 200 mm and thickness of the plates on either side is 6 mm. Simplified approach is used to model the panel. Load transfer from concrete to steel plates is realised through the shear connector. Interface between steel and concrete surfaces is modeled using surface to surface contact. Figure 4.7 shows the finite element model of the panel.
(a) Panel with concrete core   (b) Steel plates and shear connector

Figure 4.7 Finite element model of the panel

Properties of the concrete and steel used in the study are given in Table 4.3. Steel behaviour is idealized using bilinear stress-strain model and a parabolic curve is used to characterise the behaviour of concrete as shown in Figures 4.8 (a) and 4.8 (b) respectively.

Table 4.3 Properties of concrete and steel

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Material</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Steel</td>
</tr>
<tr>
<td>Yield stress (MPa)</td>
<td>350</td>
</tr>
<tr>
<td>Young’s modulus (GPa)</td>
<td>200</td>
</tr>
<tr>
<td>Cube strength (MPa)</td>
<td>-</td>
</tr>
</tbody>
</table>

(a) Steel   (b) Concrete

Figure 4.8 Material model used in the study
Blast pressure due to explosion of 200 kg TNT at 5 m distance from the panel is computed (Kinney and Graham, 1985) and is shown in Figure 4.9. Nonlinear transient dynamic analysis is carried out with time step of 0.00001 sec. Responses in terms of transverse displacement at centre of the panel are obtained. Figure 4.10 shows the displacement time history at centre point of the panel, and peak displacement is found to be about 17.6 mm.

Cross-section of steel-concrete composite panel and the stress variation along its depth are illustrated in Figure 4.11. When the panel is in elastic state, bottom plate experience tension and top plate and concrete above neutral axes are in compression as shown in Figure 4.11(a). When the section further deflects the concrete assumes a stress variation as shown in Figure 4.11(b). In both the cases, concrete above neutral axis contributes to the moment capacity of the section. At ultimate stage, capacity of the section is only due to the outer steel plates and is as shown in Figure 4.11(c).

![Figure 4.9 Pressure time history](image1)
![Figure 4.10 Displacement time history](image2)
Depth to neutral axes is calculated by applying equilibrium of forces. Second moment of area $I_b$ is calculated based on the transformed section. A factor ‘c’ is applied to $I_b$ to obtain the effective moment of inertia, $I_{eff}$. Effective stiffness, $k_e$ which is a function of $I$, $E$ and $L$ is obtained from standard charts available in TM5-1300. Peak deflection of the panel is calculated based on the procedure described in Coyle and Cormie (2009) and is as given below:

Maximum resistance of the section, $R_m$ is calculated for simply supported condition and subjected to uniformly distributed load using the following equation:

$$R_m = \frac{12}{a} \left( M_{Pa} + M_{Pb} \right)$$  \hspace{1cm} (4.2)

where

- $M_{Pa} = \text{total positive ultimate moment capacity along midspan section parallel to short edge / m}$,
- $M_{Pb} = \text{total positive ultimate moment capacity}$
along midspan section parallel to long edge
/ m,

\( a = \) short span,

\( b = \) long span

\( M_{Pa} = M_{Pb} = M_R \), which is given by

\[
M_R = t_f y (h_c + t) \tag{4.3}
\]

where

\( M_R = \) moment of resistance,

\( t = t_c = t_t \),

\( t_c = \) thickness of compression steel plate,

\( t_t = \) thickness of tension steel plate,

\( h_c = \) thickness of concrete core,

and \( f_y = \) yield stress of the steel plate

Depth to neutral axis, ‘x’ is given by

\[
x = B + \sqrt{B^2 - 2C} \tag{4.4}
\]

where

\( B = \alpha_c t_c + \alpha_e t_t - t_t \) \tag{4.5}

\( C = -D \alpha_c t_c + \frac{t_f^2 \alpha_e}{2} + \frac{t_c^2 \alpha_e}{2} + \frac{t_t^2}{2} \) \tag{4.6}

\( D = \) overall thickness,

\[
\alpha_e = \frac{E_s}{E_c} \tag{4.7}
\]

Second moment of area of unit width of the panel, \( I_b \), is calculated as

\[
I_b = b t_c \left( x - \frac{t_c}{2} \right)^2 + \frac{b}{\alpha_e} \left( x - t_c \right)^3 + b t_t \left( D - x - \frac{t_t}{2} \right)^2 \tag{4.8}
\]
Due to reduced shear stiffness that results from slip between the face plates and concrete, effective second moment of area, $I_{\text{eff}}$ is obtained by multiplying second moment of area with a factor $c$, which is calculated from

$$c = \frac{\delta_b}{\delta_b + \delta_s} \quad (4.9)$$

where $\delta_b$ and $\delta_s$ are deflections due to unit bending and shear loads respectively.

Effective shear stiffness, $G'$, which is required in calculation of $\delta_s$ is determined from empirical equation.

$$G' = 4.53 \times 10^5 \left( \frac{t_t + t_c}{s_x \times 0.7} \right) - 310 \quad (4.10)$$

where $s_x$ and $s_y$ are the shear connector spacing in the primary and secondary span directions respectively and $t_c$, $t_t$, $s_x$ and $s_y$ are in mm and the calculated $G'$ is in N/mm$^2$.

Effective stiffness of unit width of the panel, $k_e$ is obtained from

$$k_e = \frac{252EI_{\text{eff}}}{a^2} \text{N/m} \quad (4.11)$$

Elastic deflection of the panel, $X_e$ is obtained from

$$X_e = \frac{R_m}{k_e} \quad (4.12)$$

Basic impulse equation is obtained by equating areas under resistance deflection curve from Figure 4.12 and pressure time curve from Figure 4.13.
\[
\frac{i^2A_2}{2K_{LM}M} = \frac{R_mX_e}{2} + R_m(X_m - X_e)
\]  

(4.13)

where

\begin{align*}
K_{LM} & = \text{load-mass factor given in IS: 4991-1968}, \\
X_m & = \text{maximum deflection attainable by the section}, \\
i & = \text{unit impulse}, \\
A & = \text{area on which blast pressure ‘p’ is acting}, \\
M & = \text{mass of panel}
\end{align*}

In Equation (4.13), all other values except \(X_m\) is known. Therefore,

\[
X_m = \frac{\left(\frac{i^2A_2}{2K_{LM}M}\right) + \frac{R_mX_e}{2}}{R_m}
\]

(4.14)

Value of peak displacement corresponding to the blast loading chosen in the example is calculated as 16.7 mm using the above procedure. This value is in close agreement with the peak displacement of 17.6 mm obtained from the finite element analysis. Thus, the simplified approach can be used for modelling the steel-concrete composite panels subjected to blast loading.
4.3 NUMERICAL STUDIES

A steel-concrete composite panel of 2 m x 2 m size with through-through connectors spaced at 200 mm c/c in both the directions is taken up for study. Size of the connector is kept as 16 mm. Concrete core thickness is 200 mm, while steel cover plates of 6 mm thickness. This panel is subjected to air-blast loading due to a charge at 5 m distance from the centre of the panel as shown in Figure 4.14.

![Figure 4.14 Charge weight location](image)

4.3.1 Charge variation

The charge at 5 m distance is varied from 100 kg TNT to 400 kg TNT. Pressure time histories due to explosion of these charges are generated. Panel is modelled using the simplified approach. Nonlinear transient dynamic analysis is carried out. Figure 4.15 shows the time history of the displacement at centre of the panel for different charge weights. It can be observed that the peak displacement increases with the charge weight. Analyses are repeated for different plate thicknesses of 8 mm, 10 mm and 12 mm. Variation of peak central displacement with charge weight is plotted in Figure 4.16. Peak displacement is found to vary in a nonlinear manner with charge and is proportional to the impulse. It can also be
noted that similar trend of variation is observed for all plate thicknesses. Figure 4.17 shows the variation of time period of response with charge weight. This variation is found to be in similar trend as observed in Figure 4.16.

![Figure 4.15 Time history of displacement](image)

![Figure 4.16 Variation of peak displacement with charge weight](image)
4.3.2 Thickness variation

Thickness of the cover plates is varied from 6 mm to 12 mm with increments of 2 mm. All other parameters are kept same as above. Panel is subjected to pressure loading due to explosion of 100 kg TNT at a distance of 5 m. Time history of displacement is shown in Figure 4.18. Peak displacement at centre of the panel is obtained. This set of analysis is repeated for 200, 300 and 400 kg TNT. Variation of peak displacement with thickness is plotted in Figure 4.19. It can be observed that the variation is nonlinear. Similar trend is observed for all charge weights. Time period of response is plotted for all plate thicknesses as shown in Figure 4.20. Similarity of variation of peak displacement and that of time period of response can be observed.
Figure 4.18 Time history of displacement for 200 kg TNT

Figure 4.19 Variation of displacement with thickness

Figure 4.20 Variation of time period of response with thickness of plate
4.3.3 Diameter variation

The diameter of connector is varied as 16 mm, 20 mm and 25 mm, and all other parameters are kept constant, namely, spacing of connector, charge weight and plate thickness. Spacing of connector is kept as 200 mm while charge weight is 100 kg. Thickness of plate is 6mm. Diameter of connector is found to have only negligible influence on the peak response as observed in Figure 4.21.

![Figure 4.21 Variation of peak displacement with connector diameter](image)

4.4 SUMMARY

In this chapter, a simplified approach of modelling the steel-concrete composite panel subjected to air-blast loading is proposed to simulate the behaviour. This approach is computationally efficient and requires less modelling effort. A SCC beam subjected to static load at centre is analysed using the proposed approach. Ultimate load and deflection are predicted with less than about 10% difference with that of experimental values, while yield load and deflection are predicted with about 16% difference. Dynamic response of the SCC panel modeled using
The proposed approach is verified using an analytical model. Peak displacement predicted by using the proposed approach is in good agreement with the analytical study. Parametric studies are conducted to find out their influence on peak response. Plate thickness is found to influence the response in a nonlinear manner, while diameter of connector has only negligible influence. Variation of displacement with charge is found to be nonlinear and is proportional to the impulse.

Steel-concrete composite panels can be constructed rapidly and can be the best alternative for laced reinforced concrete for blast-resistant construction. However, the welding of studs and placing concrete in between the parallel plates etc., will pose problems in construction. Only pre-fabrication is possible. However, storage structures of high energy chemicals / explosives are always kept away from populated areas. Thus, moving construction machinery and setting up of a pre-fabrication unit for limited number of structures is infeasible. A system that can combine the advantage of steel-concrete composite together with lacing appears to be the best choice. The development of such a scheme is discussed in the next chapter.