Numerical study on underground structures subjected to shock loading
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ABSTRACT

This paper presents a numerical investigation on shock response of underground structures made of fibre reinforced composite material. A finite element model is developed for determining the shock response of the structure including soil-structure interactions. Commercially available software ABAQUS is used for the finite element model and computations. Circular, horseshoe and square cross-sections of the underground structures are considered for the analysis and design against the air blast of 500 kT TNT at a distance of 1 mile. Equivalent of TNT yield value the pressures and time histories are generated and accordingly blast parameters are obtained. The mechanical properties of steel fibre composite material used in the study are determined based on laboratory tests. The soil-structure interaction effects are implemented through spring and dashpot systems at the boundary regions. Various responses like displacements, von Mises stresses and strain energies obtained from the numerical analysis are reported.

Keywords: Numerical study; Underground structures; Shock response; Soil spring and dashpots; Fibre reinforced composites; TNT.

Notation

\begin{itemize}
\item $\varepsilon_c$ Inelastic compressive strain
\item $\varepsilon_0c$ Total compressive strain
\item $\varepsilon_0\text{el}$ Elastic strain corresponding to undamaged material
\item $E_0$ Initial Young’s modulus of the material
\item $\sigma_c$ Compressive stress corresponding to undamaged state
\item $\varepsilon_p$ Compressive equivalent plastic strain
\item $\varepsilon_k$ Cracking strain
\item $\varepsilon_t$ Total tensile strain
\item $\varepsilon_0\text{el}$ Cracking strain corresponding to undamaged material
\item $\sigma_t$ Tensile stress corresponding to undamaged state
\item $\varepsilon_p$ Tensile equivalent plastic strain
\end{itemize}

1. Introduction

The response of underground structures subjected to subsurface blast is an important topic in protective engineering. Due to various constraints, pertinent experimental data are extremely difficult to get. Therefore adequately detailed numerical simulation becomes a desirable alternative. However, the physical processes involved in the explosion and blast wave propagation are very complex, hence a realistic and detailed reproduction of the phenomena
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would require sophisticated numerical models for the loading and material responses. The soil-structure interaction plays an important role in the behaviour of underground structures subjected to shock load (Kumar. M et al.,2010). Hence it is necessary to consider soil structure interaction effects in the design of underground structures for better performance. An experimental study (Hailong Chen et al., 2014) has been reported on buried scaled down reinforced concrete arch structure is subjected to underground close-in explosions. Varying charge weights ranging from 0.6 kg, 3 kg, 6 kg and 17.4 kg TNT have been used. The results have shown that buried arch deforms at a dominant flexural mode (Hailong Chen et al., 2014). Since blast loads are high intensity transient loads they result in steep stress gradient in structures, so the material considered for structure should be able to withstand suddenly applied loads. Structural response to blast loads should cover a time domain much longer than shock duration, because response computations performed within the shock duration reported to produce underestimate the maximum structural response (Dhakal R.P, T.C. Pan. 2003). Hence, in the FE analysis time duration should be considered more than that the blast pressure duration to capture complete response of the structure.

This numerical study focuses on predicting the shock response of underground structures of different cross sections namely, circular, horseshoe, and square (Figure.1). The shock loading generated due to an air blast of 500kT (equivalent of TNT) at a distance of 1 mile from the underground structure is considered. The depth of the top level of structures is 1.5 m from the ground level. The internal dimensions of the structures are kept constant i.e. 2.7 m x 2.7 m. Initially a thickness of 180 mm is assumed for structures (Figure 1). The type of soil considered for study is c-Ø soil. The soil surrounding the structure is treated as disturbed soil of dimensions 6.5m x 6.06m and soil away from the structure is considered as undisturbed soil of dimensions 6.5m x 7.06 m on either side of disturbed soil region.

Figure 1: Schematic plan of underground structures
Due to higher energy absorbing capability, ductility, tensile strength and toughness of steel fibre reinforced composites perform better as compared to conventional plain and reinforced cement concrete (Balaguru P. N 1992). Fibre reinforced concrete is well suited for structures which are intended to resist impact, shock loads. The free field pressure time variations are clearly explained in (Theodor, Krauthammer, 1993) which consists of positive and negative phases and the pressure decays in an exponential manner with respect to time. The peak values of the negative pressures are generally small compared to peak positive overpressures so the values in negative phase can be neglected.

The structure is modeled using non linear material properties based on concrete damage plasticity concepts besides compression damage and tension damage. Soil structure interaction effects, contact and interaction properties are included in the finite element model. Based on numerical analysis the responses of the structures are compared and favorable shape for underground structure under blast loading is reported.

2. Finite Element Model

Geometric modeling of structure, undisturbed soil, and disturbed soil is developed using ABAQUS V 6.13-1 and they are assembled. Due to symmetry of the model in Z direction, and assuming that over pressure induced by the blast wave is same at every point in the Z direction only 1m strip is modeled. Boundary conditions are applied by restraining UR and U in X, Y and Z directions for the outer peripheries of undisturbed soil which are parallel to Y direction. A finite strip of soil is modeled below the actual model to facilitate the addition of spring and dashpots. The outer periphery of undisturbed soil i.e. parallel to X direction and finite soil strip below the model are connected with soil spring and dashpots. For bottom nodes of finite soil strip UR and U are restrained in X, Y and Z directions.

Finite element used for soil and structure is Eight-node linear brick element (C3D8R) with reduced integration and hourglass control which is chosen from Abaqus/Explicit solid element library. Partitioning cell methodology is employed to divide the part instances in

![Figure 2: Finite element model of underground structure](image-url)
assembly into regions. These regions can be used to gain finer control over the meshing process. Meshing is done on the assembled model by assigning seeds on each and every edge.

**Undisturbed soil region**

![Figure 3: Zoomed view of spring and dashpots](image)

Surface to surface contact explicit is chosen for this model by creating three different surfaces on structure, disturbed soil and undisturbed soil. Circumferential surface of disturbed soil region is modeled as master and outer surface of structure is modeled as slave. Mechanical interaction property is modeled between these surfaces using friction formulation in tangential direction and hard contact in normal direction to avoid penetration of surfaces into each other. Separation is allowed after contact between disturbed soil region and structure.

### 2.1 Properties of fibre reinforced composite material

In this study the structures are made of fibre reinforced composite material. The values presented below are based on laboratory tests.

| Table 1: Details of mechanical properties of the material |
|---|---|---|
| **Material** | **Description** | **Value** |
| Concrete | Ultimate compressive stress, MPa | 82 |
| | Permissible tensile stress, MPa | 8 |
| | Permissible flexural tensile stress, MPa | 24 |
| Fibres | Young’s modulus of fibre, MPa | $2.1 \times 10^5$ |
| | Young’s modulus of matrix, MPa | 20,000 |
| | Poisson’s ratio of fibre | 0.3 |
| | Poisson’s ratio of matrix | 0.2 |
| | Volume of fibres (% volume of composite material) | 7% |
| FRC | Young’s modulus, MPa | 26844 |
| | Poisson’s ratio | 0.21 |
| | Density, kg/m$^3$ | 2500 |

The stress-strain behavior of FRC in uniaxial compression and tension used in the study are given in Figure.5 and 6.

### 2.2 Properties of soil
The type of soil being c-Ø, the undisturbed and disturbed soil parameters vary in density, Young’s modulus and Shear modulus. The Poisson’s ratio and density are taken as standard. For c-Ø soil shear wave velocity which is related to depth below the ground surface is taken as 135 m/sec (Das, B.M 1983). The Dynamic young’s modulus and shear modulus which are based on shear wave velocity are evaluated by using equations mentioned in (Das, B.M 1983). The soil spring and dashpot coefficients which are frequency independent are evaluated using expressions mentioned in (Hammam AH 2000).

**Figure 4:** Stress-strain behaviour of fibre reinforced composites in uniaxial compression

![Stress-strain behaviour of fibre reinforced composites in uniaxial compression](image)

**Figure 5:** Stress-strain behaviour of fibre reinforced composites in split tension

![Stress-strain behaviour of fibre reinforced composites in split tension](image)

**Table 2:** Dynamic soil properties for undisturbed and disturbed soil

<table>
<thead>
<tr>
<th>Property</th>
<th>Undisturbed soil</th>
<th>Disturbed soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shear Modulus (G)</td>
<td>33440367 N/m²</td>
<td>27866972.5 N/m²</td>
</tr>
<tr>
<td>Young’s Modulus (E)</td>
<td>93633027.6 N/m²</td>
<td>78027523 N/m²</td>
</tr>
</tbody>
</table>

**Table 3:** Density and Poisson’s ratio of the soil

<table>
<thead>
<tr>
<th>Property</th>
<th>Undisturbed soil</th>
<th>Disturbed soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density</td>
<td>1800 kg/m³</td>
<td>1500 kg/m³</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.4</td>
<td>0.4</td>
</tr>
</tbody>
</table>
2.3 Blast pressure data

For an air blast of 500kT TNT at a distance of 1 mile from the structure and height of burst being 500 feet, the blast parameters are calculated using (IS 4991-1968). Based on (ABAQUS 2011) only general overpressure $P_{so}$ is considered and the reflected and dynamic pressures are neglected. Blast pressure time history is idealized as triangular type pressure distribution Figure 6.

$$t = 1.136 \text{ s}$$

$$P_{so} = 175822.5$$

Figure 6: Idealized pressure-time history for 1 mile air blast

<table>
<thead>
<tr>
<th>Time(s)</th>
<th>Pressure $(N/m^2)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.01</td>
<td>87911.25</td>
</tr>
<tr>
<td>0.568</td>
<td>175822.5</td>
</tr>
<tr>
<td>1.136</td>
<td>0</td>
</tr>
<tr>
<td>10</td>
<td>0</td>
</tr>
</tbody>
</table>

Table 4: Blast pressure-time history

3. Constitutive material model

3.1 Concrete damage plasticity model

A constitutive model called concrete damage plasticity model based on models proposed by Lubliner et al. (1989) and Lee (1998) in (ABAQUS III) is considered to account for the inelastic mechanical properties and to model the behavior of concrete under shock loading. This model is based on the assumption of scalar damage in which the degradation of elastic stiffness induced by plastic straining both in tension and compression are considered. Degradation of the stiffness is significantly different between compression and tension. The response of degraded concrete is characterized by two independent uniaxial damage variables $d_c$ (compression) and $d_t$ (tension) which are assumed to be functions of the plastic strains and field variables. The damage variables $d_c$ and $d_t$ have values between 0 and 1.

3.1.1 Defining compressive behavior

In ABAQUS compressive stress data is provided as a tabular function of inelastic strain (or crushing strain) $\varepsilon^{\text{cr, in}}$. The stress-strain curve is defined beyond the ultimate stress, in to strain
softening regime and positive values are given for the compressive stress and strain. First, compressive inelastic strain is calculated as illustrated in Figure.8 and by using the following equations.

\[ \varepsilon_{c}^{\text{in}} = \varepsilon_{c} - \varepsilon_{0} \]  
\[ \varepsilon_{0}^{\text{pl}} = \frac{\varepsilon_{c} - \varepsilon_{0}}{1 - d_{c}} \]  

\[ \varepsilon_{c} = \varepsilon_{c} - \varepsilon_{0} \]  
\[ \varepsilon_{0}^{\text{pl}} = \frac{\varepsilon_{c} - \varepsilon_{0}}{1 - d_{c}} \]  

Figure 7: Defining inelastic strain in compression

These compressive inelastic strain values are converted into plastic strain values \( \varepsilon_{c}^{\text{pl}} \) using the following relationship

\[ \varepsilon_{c}^{\text{pl}} = \varepsilon_{c}^{\text{in}} - \frac{d_{c}}{1 - d_{c}} \varepsilon_{0} \]  

3.1.2 Defining tension behavior

Tension stiffening in ABAQUS is provided as tabular form of post failure stress as a function of cracking strain, \( \varepsilon_{t}^{\text{cr}} \). Similar to compression, positive values are given for tensile stress and strain in tension also. Then the cracking strain is calculated as illustrated in Figure.9 and by using the following equations.

\[ \varepsilon_{t}^{\text{cr}} = \varepsilon_{t} - \varepsilon_{0} \]  
\[ \varepsilon_{0}^{\text{pl}} = \frac{\varepsilon_{t} - \varepsilon_{0}}{1 - d_{t}} \]  

\[ \varepsilon_{t} = \varepsilon_{t} - \varepsilon_{0} \]  
\[ \varepsilon_{0}^{\text{pl}} = \frac{\varepsilon_{t} - \varepsilon_{0}}{1 - d_{t}} \]  

Figure 8: Defining inelastic strain in tension

\[ \varepsilon_{t}^{\text{pl}} = \varepsilon_{t} - \varepsilon_{0} \]  
\[ \varepsilon_{0}^{\text{pl}} = \frac{\varepsilon_{t} - \varepsilon_{0}}{1 - d_{t}} \]
Figure 8: Defining cracking strain in tension

These cracking strain values are converted into plastic strain values ($\varepsilon_{c}^{pl}$) using the following relationship

$$\varepsilon_{c}^{pl} = \varepsilon_{c}^{ck} - \frac{\varepsilon_{c}^{ck}}{(1-\varepsilon_{c}^{ck})}$$

(6)

In addition to this, dilation angle for concrete is taken as $38^0$ which represents concrete has almost same dilation angle over a wide range of confining pressure, flow potential eccentricity G is considered as 0.1, and a factor K, which is the ratio of second stress invariant on the tensile meridian at initial yield for any given value of the pressure invariant is taken as 0.666 in the model. The values of these parameters are adopted as recommended (Guowei Ma et al., 2013).

4. Results and discussions

To ascertain the response of structures Explicit Dynamic analysis is executed using ABAQUS V 6.13-1. Displacement time history of structures, von Mises stresses and strain energies are presented for circular, horseshoe, square cross sections and compared. The total time duration of blast pressure including free vibration phase is 11.136 s. In the first step, analysis is carried out by considering 180 mm thickness for structures which resulted in stresses that are within the permissible limits for tension and compression. In the next step of analysis stress contours showed that circular and horseshoe cross sections are even safe at 150mm thickness where as square cross section requires 180mm.

Figure 9: Displacement time history

The maximum displacement (Figure 9) in the displacement time history is encountered by square structure i.e. 40 mm compared to circular and horseshoe structures whose maximum displacement is about 30 mm. Maximum von Mises stress in circular structure is observed at 0.5568s since maximum pressure in the pressure time history corresponds to 0.568s Figure 10. Similarly for horseshoe and square structures maximum von Mises stress is observed at 0.5568s.

Figure 10: von Mises stress contour in a circular structure at 0.5568s
The von Mises stress vs. time is plotted for region of maximum stress in circular structure as shown in Figure 11, similarly for horseshoe and square structures their respective region of maximum stress are plotted with respect to time. At a particular instant of time von Mises stress is around 8MPa for circular, square structures and for horseshoe structure it is close to 7MPa Figure 11.

![Figure 11: von Mises stress vs. time](image)

Strain energy vs. time is plotted for circular, horseshoe and square structures. Compared to horseshoe structure, strain energy in circular and square structures is relatively high Figure.12.

![Figure 12: Strain energy vs. time](image)

5. Conclusion

A three dimensional finite element model for underground structure is developed using the commercial software ABAQUS. The following conclusions may be drawn from the present study:

1. Square shaped structure has showed to have thicker walls as compared to other two shapes. Whereas circular and horseshoe shapes are nearly equal.
2. The vertical displacement of crown is lesser for circular and horseshoe structures as compared to square cross-section.
3. von Mises stress, are found lesser in horseshoe shaped underground structure hence found better than circular and square shaped structures.
4. Peak strain energy found minimum for circular shaped structure. However all the shapes have shown similar attenuation of strain energy?

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6. References


