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RESPONSES OF UNDERGROUND STRUCTURES SUBJECTED TO BLAST LOADING

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CUNY City College

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RESPONSES OF UNDERGROUND STRUCTURES
SUBJECTED TO BLAST LOADING

A THESIS IN
Civil Engineering

Submitted in partial fulfillment of
The requirements for the degree of
MASTERS OF CIVIL ENGINEERING

at
The City College of New York of the
City University of New York

by

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Professor Huabei Liu, Thesis Advisor

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Professor Julio Davalos, Chairman
Department of Civil Engineering

May 2014
ABSTRACT

Terrorism against American citizens and assets is real and growing. The number and intensity of domestic and international terrorist events, along with the September 11, 2001, attacks, change the way Americans think and live.

According to the Blue Ribbon Panel (BRP) on Bridge and Tunnel Security assigned by AASHTO, the US transportation system consists of 337 highway tunnels and 211 transit tunnels in 2003. The number is expected to grow in the near future. These tunnels are subjected to the threats of internal explosion, either accidental or maliciously intentional. Explosions inside transportation tunnels would result in direct casualties; and the subsequent damages of tunnel structures could further lead to large socioeconomic losses. Specifically the century-old cast-iron subway tunnels in cities such as New York and London are very vulnerable to this type of attack.

This study aims to reveal the fundamental knowledge on the interaction between transportation tunnels and saturated soils subject to internal explosions using medium amounts of explosives (< 100Kg TNT). Centrifuge modeling made it possible to create small scale models using a relatively small quantity of explosives under a high g-level. Two tests conducted at 50g, one under dry sand and the other under saturated one, using 1.2 g of TNT equivalent of explosives, resulted in explosions equivalent to 150 Kg or 1.47 KN (0.15 tons) of explosives under normal gravity (1 g). Strains induced at different location of the model as a result of the explosions were captured using TML strain gauges. Results showed that the stresses in the lining depend on its thickness and the nature of the debris that project due to the explosion which most likely caused the rupture of the tunnel lining.
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CHAPTER 1: INTRODUCTION

The following explores the current state of knowledge related to this study, including responses of underground structures subjected to blast loading, and blast-induced pore pressure and soil liquefaction, with the intention to identify knowledge gap and research need.

1.1 Responses of Underground Structures Subjected to Blast Loading

Owing to its military importance, responses of underground structures subjected to explosive loading have been extensively studied. For explosions outside underground structures, most of the studies focused on cratering, earth pressure on underground structures, and corresponding structure damage; some also investigated protective measures and found that flexible barriers reduced more earth pressure than rigid ones [1]. Only few of these studies considered the coupling of pore fluid and soil particles, not to mention the change of effective stress and its effect on underground structures. For explosions inside underground structures, air-blast, ground blast wave, blast pressure, collapse and debris of underground structures have been investigated. These studies are mostly related to large-scale explosions inside underground ammunition storages in rock mass, the findings of which cannot be directly applied to transportation tunnels in saturated soils subjected to medium blast loading [2].

In contrast, few studies on the responses of civilian underground structures subjected to internal blast loading can be found in the literature. The subjects of Chille et al and Choi et al. were both underground structures in rock masses [3]. Choi et al. [4], through analysis of coupled air-solid interaction, found that the blast pressure on tunnel lining was not the same as the CONWEP normally reflected pressure. Preece et al. [5] investigated the response of a 13-ft diameter aluminum tunnel in moist soil subjected to internal blast loading from 6600 pounds of TNT using centrifuge test, which is not realistic for the hazard facing general transportation
tunnels. Port Authority of New York and New Jersey and several other transportation agencies investigated the blast vulnerability of specific tunnels after 9/11 but unfortunately their results are not released. Very recently, through numerical analysis the PI found that single track subway tunnels in saturated silty soil with cast-iron lining, which are used extensively in New York City, would damage under modest internal explosion (50 – 75 kg of TNT-equivalent). Unfortunately, due to the restriction of numerical procedure, the changes of compressive strain, pore pressure and effective stress could not be considered and the full picture of saturated soil-tunnel interaction was not captured. This research also found that under single blast loading, the tunnel vibrated drastically and applied multiple shocks to the ground media, as shown in Fig. 1, which coincided with the finding of Feldgun et al. [6].

![Acceleration of cast-iron lining (6.5-cm thick, inner diameter=5 m) subject to internal explosion of 75 kg TNT](image)

**Figure 1** Acceleration of cast-iron lining (6.5-cm thick, inner diameter=5 m) subject to internal explosion of 75 kg TNT

### 1.2 Blast induced Pore Pressure and Soil Liquefaction

Under blast loading, the large compressive pressure can induce large compressive strain owing to the compression of soil particles and pore water, and upon unloading, the inelastic
compressibility of soil skeleton would induce residual excess pore pressure, the magnitude of which can be adequately large to liquefy saturated soils, as illustrated in Fig. 2 below.

This issue has been investigated extensively since 1960’s and blast loading has been used to densify loose soil deposits and to initiate soil liquefaction for research purpose. Studies have found that more crushable soil particles result in higher compressive strain and residual excess pore pressure. While most studies have focused on sandy soils, some showed that silty soil and clayey sand can also liquefy under blast loading [7]. Equations that relate peak compressive strain, peak pore pressure and residual pore pressure to explosive, distance, initial confining stress and relative density have been proposed, but the influence of relative density on residual excess pore pressure was found to be small, and blast-induced liquefaction can occur in dense sand. Magnitudes of particle velocity around 0.4 cm/s were reported to have initiate blast-induced soil liquefaction, and past studies also showed that multiple shocks significantly increase residual excess pore pressure, which implies the large possibility of high residual excess pore pressure due to blast loading inside transportation tunnels.
1.3 Research Objective

This research is intended to study the damage mode of tunnels in both dry and saturated soil. It is also aimed at determining the extent of residual excess pore pressure as well as its effect on tunnel response. The research will also provide well-instrumented test results for calibration of numerical procedures.
CHAPTER 2: CENTRIFUGE MODELING

Centrifuge modeling is a proved approach for investigating geotechnical problems, including development of excess pore pressure, soil liquefaction and their effects on structure responses. For simulating explosion-related soil response and soil-structure interaction, it has been extensively employed since 1980’s, the accuracy of which was demonstrated by the comparison of full-scale and centrifuge tests.

2.1 Scaling of Explosion

The centrifuge tests reported in this paper were both conducted at 50g. In Each test two exploding bridgewire (EBW RP-81) charges, Fig. 3, were set in series to explode simultaneously. Each charge carried 0.45 g of cyclotrimethylene trinitramine (RDX), with a TNT equivalency of 0.6 g [8]. Since two such charges were used in each test, the TNT equivalent used was 1.2 g under normal gravity.

Knowing that the centrifuge modeling of explosion is based on the scaling law of $W_{\text{prototype}} = W_{\text{model}} \cdot N^3$, in which $W_{\text{prototype}}$ is the weight of explosive in prototype scale, $W_{\text{model}}$ is the one in model scale, and $N$ is the centrifugal acceleration in g, 1.2 g of explosives detonated in each test under a 50 g acceleration produced the same output as would 150 Kg of TNT or 1.47 KN (0.15 tons) of explosives under normal gravity (1 g). Table 1 below gives the scaling factors that have been established for basic soil parameters in centrifuge tests [5].
<table>
<thead>
<tr>
<th>Parameter</th>
<th>Scale Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>1/N</td>
</tr>
<tr>
<td>Displacement</td>
<td>1/N</td>
</tr>
<tr>
<td>Stress</td>
<td>1</td>
</tr>
<tr>
<td>Strain</td>
<td>1</td>
</tr>
<tr>
<td>Acceleration</td>
<td>N</td>
</tr>
<tr>
<td>Time</td>
<td>1/N</td>
</tr>
<tr>
<td>Energy</td>
<td>1/N^3</td>
</tr>
<tr>
<td>Frequency</td>
<td>N</td>
</tr>
</tbody>
</table>

### 2.2 Modeling of Tunnel Lining

Single-track transit tunnels in New York City are generally lined with reinforced concrete (RC), grey cast iron or steel. These linings’ stiffness, strength and ductility are very different, which significantly affect the ground-tunnel interaction under internal blast loading. In this study, cast iron lining will be investigated and the following discusses its modeling in centrifuge tests.

The model lining will be 0.06” thick and 3.94” in diameter, simulating a prototype of 7 cm (2.75”) in thickness and 5 m (16.4’) in diameter, using scale factors shown on Table 1, under a centrifugal acceleration of 50g.

Because it is very difficult to find thin cast iron to meet the analogy requirement, this study will not attempt to model the failure of grey cast-iron lining in centrifuge modeling. Rather, it will focus on the responses of ground-lining system under smaller explosion, which is mainly governed by lining stiffness. According to ASTM standards, the modulus of elasticity of grey cast-iron Class 20 is about 70~80 GPa, which is very similar to that of aluminum alloy. As a result, 6061 T6 aluminum round tubes with 4 in outside diameter and 0.065 in wall thickness
were selected for this project. 6061-T6 Aluminum has a modulus of elasticity of $7 \times 10^6$ psi and a yielding strength of 35 ksi, according to the tension test performed on the specimen in Fig. 4.

![Stress Strain Curve for 6061 T6 Aluminum](image)

**Figure 4 Stress Strain Curve for 6061 T6 Aluminum**

### 2.3 Test Soil

A Homogeneous, medium-dense sand (Nevada #120) was used as the soil material supporting the test models, as well as the backfill and cover material. The sand was placed dry and compacted in place to achieve a relative density of approximately 52.4% for the first test, and about 60% for the second test in the case of saturated soil.

### 2.4 Boundary Conditions

Centrifuge modeling of internal explosion inside underground structures should resolve the issues of boundary condition and instrumentation. Boundaries of the test box may reflect blast wave and impair result accuracy. Consequently, in the centrifuge tests, 0.5 in DOW TUFF-R
(25 psi compressive strength) insulation foam was used at the specimen’s opening to absorb blast wave at the ends of the model and represent the boundary conditions of the prototype. This will simulate the effect of a long tunnel under internal blast loads while minimizing the interference of reflective pressures from the edge walls. The foam was selected based on an assumed pressure of approximately 20 psi at the edge wall of the tube [8].
CHAPTER 3: EXPERIMENTAL SETUP

3.1 Model structure and soil

Two centrifuge tests were conducted using a large Aluminum test box with dimensions of (40 in x 24 in x 15 in) and under a centrifugal acceleration of 50g. The first test demonstrated the effect of internal explosions in underground tunnels under dry soil, while the second was under saturated soil. The test models were 21 in long aluminum tubes with an outer diameter of 4 in and a wall thickness of 0.065 in as discussed in section 2.2. Fig. 5 shows the tests experimental setup including the location of explosives, accelerometers, and pressure transducers, while Fig. 6 shows the configuration of the strain gauges.
Section B-B

Section A-A

Figure 5 Experimental Setup

Figure 6 Strain Rosette Configuration
In order to simulate the authentic condition of transportation tunnel in the ground and to prevent soil and fluid from entering the model pipe, two aluminum plates will be attached to the pipe at both ends using latex membranes glued to the pipe and the plates, as shown in Fig. 5. The pipe will then be able to deform with minimal influence from the plates.

The EBW will be placed at the center of the model tube, tied to a thin steel threaded rod fixed to hangers suspended from top of tube to its center at both ends, as shown in Fig. 5.

3.2 Instrumentation

Each tube was instrumented with 12 TML strain gauges (FLA-5-23 type). Axial, circumferential, and shear strains at half and quarter span were measured during the two tests. Fig. 6 shows a schematic diagram of the model including the locations of the strain gauges.

Four strain rosettes, Fig. 7, each containing three strain gauges whose axes are 45° apart, were placed at the top and the side of the tube at the mid-span and at the quarter span. Once the strain values \( \varepsilon_a, \varepsilon_b, \) \& \( \varepsilon_c \) of the three gauges were measured, the values of \( \varepsilon_x, \varepsilon_y, \gamma_{xy} \) can then be obtained by simultaneous solution of the following equations:

\[
\begin{align*}
\varepsilon_a &= \varepsilon_x \cos^2 \theta_a + \varepsilon_y \sin^2 \theta_a + \gamma_{xy} \sin \theta_a \cos \theta_a \\
\varepsilon_b &= \varepsilon_x \cos^2 \theta_b + \varepsilon_y \sin^2 \theta_b + \gamma_{xy} \sin \theta_b \cos \theta_b \\
\varepsilon_c &= \varepsilon_x \cos^2 \theta_c + \varepsilon_y \sin^2 \theta_c + \gamma_{xy} \sin \theta_c \cos \theta_c
\end{align*}
\]

Where:

\( \theta_a = 0^\circ \); \( \theta_b = 45^\circ \) \& \( \theta_c = 90^\circ \)
In both tests, Kyowa AS-200A accelerometers and GE PDCR 81 pressure transducers were placed around the tubes as shown in Fig. 4 to capture inertia forces and soil pressures.

### 3.3 Data Acquisition

Strain gauge values were obtained and recorded at a rate of 10,000 points per second for each strain gauge for a total duration of 6.5 seconds. This ensured the capture of the conditions before, during and after the explosion be it occurring at the first 2 seconds.
CHAPTER 4: RESULTS

Axial, hoop, and shear strains were continuously captured during each test along with soil acceleration and pressure at various locations. The following discusses the results for both tests and the effect of pore pressure by comparing the measured strains at similar locations of the test specimens.

4.1 Dry Soil test

Plots of axial, circumferential, and shear strains were developed using MatLab and classified according to prescribed locations discussed in section 3.2 as shown on Fig. 6.

Plots of strains measured at location 1 of the test tube, Fig. 8, register a peak axial compressive strain of -3000 µ, a maximum hoop strain of 3400 µ and a peak shear strain of -1900 µ.
Figure 8 Strain Values at Location 1 of the Test Tube
(Dry Test)
Apparently theses strain values are below the yielding strain of 5000 µ based on the material test results shown in section 2.2, however, this is possibly due to the fact that the peak blast pressures did not occur directly below the location of the top strain gauges as demonstrated in Fig. 9 below.

![Figure 9 After Shock Effect at Location 1 of the Test Tube](image)

Fig. 9 clearly shows a permanent deformation at the top surface of the tube which suggests that strains at that location are beyond the yielding point. However, stresses and strains at the bottom surface of the tube reached ultimate values, hence rupture of the material as shown in Fig. 10.

![Figure 10 After Shock Effect at the Bottom Centerline of the Test Tube](image)
Figure 11 Strain Values at Location 2 of the Test Tube
(Dry Test)
Axial strain values measured at location 2 the test tube, Fig. 11 show a peak value of 402 µ which corresponds to the time of detonation and suggests an expansion due to the blast load. Hoop strains measured at the same location show a peak tensile value of 218 µ that is followed by a shift to compressive strains at a value of -150 µ. Shear strains also show a peak value of -245 µ which means that the right angle at the surface of the tube between the axial and circumferential directions at the location of the strain rosette is increasing, suggesting an expansion of the material which agrees with axial and hoop strain values previously discussed.

Plots of axial strains measured at location 3 of the test tube, Fig. 12, register a peak axial tensile strain of 83 µ at the time of detonation and reach a maximum value of about 125 µ. Hoop strain values reach 417 µ at the moment of detonation and drop to a considerably constant value of about 80 µ. Shear strains however suggests shrinkage of the material at this location according to a positive strain value of 185 µ (value obtained using strain rosette equations discussed in section 3.2 since shearing strain graph was inconclusive due to excessive instrumentation noise).

Fig.13 shows pre-detonation values of axial strain at location 4 of the test tube to be around 60 µ, hoop strain values at about 40 µ, and shear strains at around -300 µ. This is possibly caused by compression loads originating from soil pressure above the tube which explains tensile hoop strains, as well as lateral loads originating from the tangential acceleration of the centrifuge which explains tensile axial strains. During the explosion, values of axial strains peaks to 120 µ and drops to almost null during the aftershock phase. Similarly, hoop strains jumps to 230 µ before dropping to -15 µ and settling at their initial value of 40 µ during the aftershock phase. Shear strains values also peak at -250 µ before dropping to initial values of -300 µ.
Figure 12 Strain Values at Location 3 of the Test Tube
(Dry Test)
Figure 13 Strain Values at Location 4 of the Test Tube (Dry Test)
Fig. 14 shows a peak acceleration value of 5g at about 1 in away from the side centerline of the tube as shown in the experimental setup. No pressure transducers were used in this test. A summary of all peak strain values at the different locations discussed previously for the dry soil test are shown on Table. 2 below.
### Table 2 Summary of Peak Strain Values

<table>
<thead>
<tr>
<th></th>
<th>Location 1</th>
<th>Location 2</th>
<th>Location 3</th>
<th>Location 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Strain</td>
<td>-3000</td>
<td>426</td>
<td>83</td>
<td>100</td>
</tr>
<tr>
<td>Hoop Strain</td>
<td>3400</td>
<td>240</td>
<td>417</td>
<td>195</td>
</tr>
<tr>
<td>Shear strain</td>
<td>-1900</td>
<td>-245</td>
<td>185</td>
<td>-250</td>
</tr>
</tbody>
</table>

#### 4.2 Saturated Soil test

Plots of strains measured at location 1 of the test tube, Fig. 18, register a peak axial compressive strain of about -500 µ followed by a peak strain value of about 500 µ, a maximum hoop strain of around 5800 µ and a peak shear strain of -5700 µ. Apparently these strain values except for axial ones are above the yielding strain of 5000 µ unlike the results obtained during the dry soil test. However, just like in the dry test, the top section of the tube showed signs of permanent deformation while the bottom portion experienced stresses that are beyond ultimate values that eventually caused rupture as shown in Fig. 15 and Fig. 16 below.

![Figure 15 After Shock Effect at Location 1 of the test tube](image-url)
Additionally, different from results during the dry soil test, it was observed that two craters have formed above the centerline of the test tubes following the detonation of the EBW. This is due to the excess pore water pressure generated during the explosion that caused the soil to liquefy. Fig. 17 shows the details and the cross section of the craters.
Figure 18 Strain Values at Location 1 of the Test Tube (Saturated Test)
Figure 19 Strain Values at Location 2 of the Test Tube (Saturated Test)
Plots of strains measured at location 2 of the test tube, Fig. 19, register a peak axial compressive strain of about -1700 µ, a maximum hoop strain of around -1300 µ and a peak shear strain of 3400µ. These values are consistent with those measured at the top centerline of the tube as the tube expands due to blast pressures. As the top surface material expands, the side material contracts, which is clearly depicted by the strain values shown in Fig. 18 and Fig. 19. Fig. 20 below demonstrates the effect of blast pressures on the test tubes.

![Figure 20 Blast Pressure Effect on the Test Tube](image)

Plots of axial strains measured at location 3 of the test tube, Fig. 21, register a peak axial compressive strain of -1300 µ at the time of detonation, hoop strain values reach -370 µ at the moment of detonation and rise to 370 µ before dropping back to initial values of almost null during the aftershock phase. Shear strains however suggests shrinkage of the material at this location according to a positive strain value of 3090 µ (value obtained using strain rosette equations discussed in section 3.2 since shearing strain graph was inconclusive due to excessive instrumentation noise).

Fig. 22 shows values of axial strain at location 4 of the test tube to be around -540 µ, hoop strain values at around -880 µ, and shear strains at approximately 320 µ.
Strains values at the quarter length top and side of the test tube suggest that the tube’s cross section contracts possibly due to the effect of the blast’s first shock waves that caused the middle section to expand and the quarter length section to shrink.

Figure 21 Strain Values at Location 3 of the Test Tube (Saturated Test)
Figure 22 Strain Values at Location 4 of the Test Tube (Saturated Test)
Figure 23 Saturated Test Acceleration & Pressure Values at 1" to the Side of the Tube
Fig. 23 shows a peak acceleration value of -810 g away from the tube at about 1 in to the side of location 2 of the test tube as shown in the experimental setup. This high negative acceleration value suggests suction and it is due to the incompressibility effect of water as it helps propagate most of the blast waves rather than absorb them as it was the case during the dry soil test.

Side pressure transducers P1 and P2 both show peak values of about 45 kpa, except P1 registers a negative pressure (suction) of almost -150 kpa possibly generated by the first blast shock wave that caused the tube sides to shrink inward as discussed earlier and demonstrated by Fig. 20.

Top pressure transducer P3 confirms the strain reading found at the top centerline of tube. As the tube’s cross section expands outward due to the blast waves, a maximum positive pressure of about 5 kpa is generated which in turn increased the pore water pressure above the tube causing the soil to liquefy and hence the formation of the craters shown earlier in Fig. 17. The soil liquefaction causing the soil to fail, also generates suction effects, the fact that was captured by the transducer as negative pressures that reached a maximum value of -13 kpa.

A summary of all peak strain values at the different locations discussed previously for the saturated soil test are shown on Table. 3 below.

**Table 3 Summary of Peak Strain Values**

<table>
<thead>
<tr>
<th></th>
<th>Location 1</th>
<th>Location 2</th>
<th>Location 3</th>
<th>Location 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Axial Strain</td>
<td>500</td>
<td>-1700</td>
<td>-1300</td>
<td>-540</td>
</tr>
<tr>
<td>Hoop Strain</td>
<td>5800</td>
<td>-1300</td>
<td>-370</td>
<td>-880</td>
</tr>
<tr>
<td>Shear strain</td>
<td>-5700</td>
<td>3400</td>
<td>3090</td>
<td>320</td>
</tr>
</tbody>
</table>
CHAPTER 5: CONCLUSION

Centrifuge modeling can be very useful in predicting blast loading effects on underground structures. As shown by the results discussed previously, underground tunnels with cast iron lining like those built during the 1920s in NY City can be very vulnerable to blast loading generated by medium size explosions. Additionally, tunnels in saturated soils can experience far higher values of stresses due to the effect of pore water pressure compared to tunnels in dry soil. More importantly, most of the damages incurred to the tube’s structure were mainly due to impact of shredded particles from the EBW against the tunnel lining that acted like projectiles as a result of the explosion.

Results of centrifuge model tests can also provide a mean for calibration of numerical procedures that are often used to investigate the effects of blast loadings.
REFERENCES


