**Numerical and physical modeling of geofoam barriers as protection against
effects of surface blast on underground tunnels**

Anirban De ^{*1}, Alberto N. Morgante², and Thomas F. Zimmie³

¹Civil & Environmental Engineering Department, Manhattan College, Bronx, NY
10471, E-mail: anirban.de@manhattan.edu

²Formerly, Civil & Environmental Engineering Department, Manhattan College,
Bronx, NY 10471, E-mail: anmorgante@gmail.com

³Civil & Environmental Engineering Department, Rensselaer Polytechnic Institute,
Troy, NY 12180, E-mail: zimmit@rpi.edu

* - Corresponding author

ABSTRACT

An explosion on the ground surface may cause significant damage to an underground structure, such as a tunnel or a pipeline. The extent of damage would depend on the intensity of blast, the material and configuration of the structure, as well as the nature and geometry of the intervening material.

1 An underground structure may be protected by means of a protective barrier, installed
2 directly above the structure. The effectiveness of using a compressible barrier, made
3 of polyurethane geofoam, to mitigate the effects of surface explosion was
4 investigated.

5
6 The effects of a surface explosion were studied through a combination of physical
7 model tests and numerical modeling. Reduced-scale (1:70 scale) physical model tests
8 were conducted using a geotechnical centrifuge, where the scaling law for explosions
9 was utilized to model the effects of a large explosion using a relatively small mass of
10 explosives under a high gravitational field (70 g, in this case). The results of the
11 physical model tests were used to calibrate a three-dimensional numerical model in
12 which a fully-coupled Euler-Lagrange solver was utilized to model the explosions.

13
14 Tunnel configurations with and without protective barriers were studied to assess the
15 mitigation provided by protective barriers. Material properties for polyurethane
16 geofoam barriers were evaluated from laboratory tests. The influence of barrier
17 thickness in reducing the strains, stresses, and pressure on the tunnel induced by an
18 explosion was studied. The beneficial effects of a protective geofoam barrier were
19 found to increase with increasing barrier thickness only up to a certain thickness,
20 beyond which, further increase in thickness did not result in additional reductions.
21 The results will help in design optimization, while planning protection systems for
22 new tunnels, as well as for retrofitting existing tunnels.

23

- 1 **Keywords:** geosynthetics; centrifuge model; explosion; numerical model; geofoam
- 2 barrier; tunnel

1. INTRODUCTION

Anthropogenic explosions pose a threat to many components of the transportation infrastructure. Underground structures, such as tunnels and pipelines are vulnerable to explosions that may occur both above and below the ground. The extent of damage depends on the characteristics of the structure and those of the intervening subsurface deposit, as well as the nature of the explosion.

While the threat of underground explosions can generally be mitigated through carefully controlled access into tunnels, it is more difficult to prevent explosions that are set off on the ground surface. Therefore, it is important to evaluate the damage potential of various sizes of explosive devices set off on the ground surface with the intention to cause harm to underground structures, such as transit tunnels and pipelines.

The effects of explosions on geotechnical media and structures may be studied through physical model testing as well as through numerical modeling. Full-scale physical tests provide the most direct assessment of the effects, but are also the most difficult to conduct due to logistical and safety concerns. For these reasons, full-scale tests using explosives are typically not conducted in the civilian sector. In contrast, numerical modeling of explosions must rely on careful calibration with physical tests before the results are considered acceptable in design.

In this paper, the effects of surface blasts are studied through a combination of numerical modeling and reduced-scale physical model tests conducted on a geotechnical centrifuge. First, the concept of centrifuge scaling related to the effects of an explosion is explained, followed by a description of the physical models and instrumentation. Next, the basic components of the numerical model are presented and material properties selected for use in the model are discussed. Lastly, results from the physical model tests and numerical models are analyzed and compared. An essential component of the model, which is of key interest to this paper, is the use of a protective geofoam barrier to mitigate the effects of a surface explosion.

2. PRINCIPLE OF CENTRIFUGE MODELING OF EXPLOSIONS

The effects of an explosion under high gravity, such as those experienced on board a geotechnical centrifuge, follow the Hopkinson (or cube-root) scaling law (Baker et al., 1973). Thus, the shock waves created at two different scaled distances by two explosive charges, having the same geometry and type of explosive but different quantities of charge, would scale as the cube-root of the weight of explosives. Conversely, if two explosions produce similar shock waves, their weights will vary by the cube of the distance.

This relationship makes geotechnical centrifuge modeling a viable method for conducting experiments to study the effects of explosions on geosystems. A

1 relatively small amount of charge may be used in a reduced-scale (1:N scale)
2 centrifuge model, which is tested at a high gravity: N times g, where g is the normal
3 gravitational acceleration (9.8 m/s^2). The effects of the explosion, in terms of
4 cratering and damage caused by shock wave, will be proportional to those caused by
5 a much larger quantity of explosive under earth's normal 1 g gravity. In fact,
6 according to the cube-root relation, the effects of the explosion on the centrifuge
7 model will be proportional to the full-scale effects due to a quantity of explosives that
8 is N^3 (N raised to the third power) times the weight of explosives used in the
9 experiment (Goodings et al., 1988 and Kutter et al., 1988).

10
11 For example, the tests described in this paper were conducted on a 1:70 scale model,
12 subjected to a centrifuge acceleration of 70 g. Thus, for the purpose of these tests, N
13 = 70. Therefore, the effects of explosion, such as cratering and shock waves, were
14 the same as those created at normal gravity by an amount of explosives that is $(70)^3$
15 times, i.e., 343,000 times greater. Taking advantage of this scaling law makes
16 handling and testing of centrifuge explosion tests logistically less challenging than
17 full-scale tests.

18
19 Several researchers have demonstrated the advantage of utilizing a geotechnical
20 centrifuge to model explosions and have shown that centrifuge scaling of
21 characteristics, such as soil grain size, etc. do not affect the results related to
22 explosions [Goodings et al. (1988) and Kutter et al. (1988)]. Based on results of
23 over 100 tests, conducted at g levels between 1 g and 100 g, Goodings et al. (1988)

concluded that centrifuge tests provided a valid method of modeling the effects of explosion on soil. Kutter et al. (1988) and Davies (1991) studied the effects of explosions on underground structures through centrifuge model tests. Liu and Nezli (2015) reported results of centrifuge tests where a tunnel buried in saturated soil was subjected to internal explosion. Jayasinghe et al. (2013) reported on comparison between centrifuge test results and numerical simulation to study the effects of an underground blast on piles in saturated soil.

3. DESCRIPTION OF CENTRIFUGE MODEL

The centrifuge tests were conducted on a 150 g-ton geotechnical centrifuge at Rensselaer Polytechnic Institute, located in Troy, New York. The model consisted of a cylindrical tunnel located inside a dry sand subsurface. The sand used in the tests was Nevada #120 sand, characterized extensively during prior geotechnical research programs and published by Arulmoli et al. (1992). The sand was placed dry at a uniform unit weight of 15.7 kN/m^3 , which corresponds to a relative density of approximately 60%.

The model underground structure consisted of a 760-mm long copper pipe with an outer diameter of 76 mm and a wall thickness of 2.5 mm. Linear dimensions, such as diameter and thickness, scale directly in proportion with acceleration level. At 70 g acceleration, the model represented a prototype with an outside diameter of 5.5 m,

1 wall thickness of 0.133 m, and flexural stiffness (EI) of 13×10^6 kN m². The length of
2 the prototype structure was 53 m, based on a 760-mm long model tested at 70 g. In
3 the numerical model, a 54-m long structure was simulated.

4
5 No specific prototype condition was modeled in the analyses reported here; rather,
6 the characteristics of a wide range of prototype structures were studied and a model
7 was selected such that it represents the behavior of a generic prototype structure.
8 Further discussions on this are provided in De and Zimmie (2007). Figure 1 shows
9 the typical model configuration used in these tests.

10
11 Each model structure was instrumented with up to 19 strain gages to measure axial
12 and circumferential strains at different locations. Strain gage measurements were
13 acquired in real time during the tests and saved at a rate of 15,000 data points per
14 second (15 kHz) for each strain gage. The high rate of data acquisition ensured that
15 the relatively short peak, reached instantaneously after the explosion, was captured
16 with sufficient resolution.

17
18 In each explosion, 2.6 grams of TNT equivalent of explosive was used. According to
19 the centrifuge scaling relations, this corresponded to approximately 900 kg of TNT
20 equivalent under a normal 1 g environment. The charge was located on the ground
21 surface along the mid-section of the model structure, as shown in Figure 1. The inner
22 surfaces of the aluminum sample container were lined with corrugated cardboard in
23 order to dampen the shock waves and minimize reflected energy returning to the

structure. Strain gage measurements (presented later in the paper) indicate that there was no measurable indication of reflection from the container.

Further details of the centrifuge model are described by De and Zimmie (2007).

4. PROTECTIVE GEOFOAM BARRIER

The object of the study described in this paper was to assess the effectiveness of a protective geofoam barrier in mitigating the effects of an explosion. A compressible barrier consisting of a layer of polyurethane foam was utilized. The geofoam barrier was in the form of a semi-circular, cylindrical shield, placed directly in contact with the top half of the tunnel, as shown in the transverse section in Figure 1.

Figure 2 shows a photograph of the model tunnel with a compressible protective geofoam barrier. In this particular model, the geofoam barrier has a thickness of 19 mm (0.75 inch) in model scale, which corresponds to a thickness of 1.33 m in prototype scale at 70g.

Results from tests with a compressible geofoam protective barrier configurations are compared later in this paper with tests where no barrier was used, to evaluate the relative effectiveness of this kind of geofoam barrier. Comparisons between a compressible and rigid barrier were presented by De et al. (2013).

1

2 *4.1. Compressible Barriers*

3

4 Compressible porous inclusions, such as geofoams, are widely utilized in
5 geotechnical engineering in applications where a reduction of pressure under static
6 condition is required. Such inclusions, when placed behind earth retaining structures,
7 beneath foundations, and above pipelines and tunnels, compress readily and thereby
8 reduce earth pressure on the structure (Horvath, 1996). The application of controlled
9 deformations in reducing pressures on earth retaining structures has been documented
10 by researchers and practitioners [McGown et al. (1987), Karpurapu and Bathurst
11 (1992), Partos and Kazaniwsky (1987), and Bartlett et al. (2015)]. Bathurst et al.
12 (2007a and 2007b) and Zarnani and Bathurst (2008) have presented results of
13 numerical and physical model tests with expanded polystyrene (EPS) geofoam used
14 as seismic buffers. Murillo et al. (2009) reported results of centrifuge experiments in
15 which they studied the effectiveness of EPS geofoam as vibration isolators.

16

17 A protective barrier, situated between the source of explosion and a structure, can
18 mitigate the adverse effects of the blast on the structure. Such application using
19 metal cladding sheets were reported by Guruprasad and Mukherjee (2000a and
20 2000b) and Hanssen et al. (2002). A few recent publications have reported the use of
21 compressible material to absorb energy due to high strain-rate dynamic loading, such
22 as direct impact and blast loading.

23

1 The use of compressible porous barriers composed of a polyurethane layer to reduce
2 the impact of surface explosions on an underground tunnel was reported by De and
3 Zimmie (2007). Results of numerical modeling to study the ability of polyurea, in
4 the form of helmet suspension-pads, to mitigate the effects of blast waves and
5 reducing traumatic brain injury as a result of impact was presented by Grujicic et al.
6 (2010). A similar application on helmet liner was also studied by Christou (2010).
7 The material properties utilized in this analysis for polyurea were based on work
8 reported by Amirkhizi et al. (2006), which consisted of an elastic hydrostatic
9 response and a non-linear viscoelastic deviatoric response. Wang et al. (2006)
10 conducted numerical analyses to understand influence of geofoam in attenuating the
11 effects of blast-induced stress waves incident on underground structure.

12
13 Skews et al. (1993) reported experimental observations of shock wave passing
14 through a compressible foam slab. Subramaniam and Andreopoulos (2013) reported
15 results of experiments where blast pressure loading was applied on cement foam with
16 cellular structure. They found that there was a minimum length of foam necessary to
17 completely attenuate the applied blast wave. Similarly, Nian et al. (2012) reported
18 there is a critical thickness of foam necessary to completely absorb the impulse of the
19 overpressure created by a blast.

20
21 Ackland et al. (2013) evaluated the effectiveness of polyurea coating over steel plates
22 in possibly reducing the deformation of the plates when subjected to blast loading. In
23 that analysis, a hyperelastic material model was utilized for polyurea, with a bonding

strength against a steel plate, which was calibrated to experimental results. Mostafa et al. (2014) reported on the development of a rigid polyurethane foam separation system intended to attenuate blast wave energy. Results reported by Bahei-El-Din et al. (2006) indicated that the presence of a polyurea interlayer in a sandwich plate was effective in mitigating the effects of a shock wave.

Ossa and Romo (2011) reported results of cyclic compression tests and resonant column tests on geofoam to evaluate the effects of cyclic loading on shear and elastic moduli. The cyclic compression tests were conducted at a frequency of 1 Hz for up to 4,000 cycles.

4.2. Laboratory Testing of Polyurethane geofoam

A series of laboratory tests were conducted to characterize the geofoam material under static loading. A cylindrical specimen, with a 63.5 mm diameter and 21 mm height, with a mass density of 40 kg/m^3 , was tested under monotonic and cyclic loading. The range of strain rates employed in these tests was between 5% and 200% per minute (between $8.3\text{e-}4$ to $3.3\text{e-}2 \text{ sec}^{-1}$).

Figure 3 shows the axial stress vs. strain plots for monotonic load-unload tests on unconfined geofoam specimens at three different strain rates. The plots show the typical compressive stress-strain response of polymeric foams (Tedesco et al., 1993), with an initial linear elastic zone, followed by a plateau, and finally, a densification, where the stress rises steeply with little increase in strain. The highest strain rate of

200% per minute ($3.3\text{e-}2$ per sec) was determined by the limitation of the loading device (Load Trac II equipment by GeoComp Corporation) and is significantly smaller than what would be expected in a blast.

4.3. Material Properties of Polyurethane geof foam

Stress-strain response at high strain rates were modeled using a method proposed by Nagy et al. (1974), where stress is expressed as a function of strain and strain rate, as shown in equation (1) below:

$$\sigma(\varepsilon) = \sigma_0(\varepsilon) \left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_0} \right)^{n(\varepsilon)} \quad (1)$$

where n is given in equation (2)

$$n(\varepsilon) = a + b\varepsilon \quad (2)$$

Equation (1) is valid for $10^{-3} \leq \dot{\varepsilon} \leq 10^2$, with strain rate, $\dot{\varepsilon}$ expressed in sec^{-1} .

Parameters a and b may be determined from experimental results, as explained below.

Reference values of stress-strain (σ_0, ε_0) are taken at static or quasi-static strain rates.

The original model was validated for the range given above.

The relation of stress with strain and strain rate, proposed by Nagy et al. (1974) has been compared with experimental data by Zhang et al. (1998) and Ouellet et al. (2006). It has been utilized by Christou (2010) and others to develop constitutive models that have been used in numerical modeling.

The terms in equation (1) are manipulated and logarithm of both sides is taken to obtain equation (3):

$$\frac{\log\left(\frac{\sigma(\varepsilon)}{\sigma_0(\varepsilon)}\right)}{\log\left(\frac{\dot{\varepsilon}}{\dot{\varepsilon}_0}\right)} = n(\varepsilon) = a + b\varepsilon \quad (3)$$

Experimental results (Figure 3) from the compression load test at a reference strain rate, $\dot{\varepsilon}_0 = 0.03 \text{ sec}^{-1}$ were used as reference stress-strain data (σ_0, ε_0) with equations (1) and (2). The strain rate, $\dot{\varepsilon}$ during application of load from an explosion was estimated through an iterative process. A preliminary numerical analysis was conducted on the underground tunnel, using an initial material model of the polyurethane geofoam and the strain rates for some of the critical strain components were measured. The maximum strain rate was found to be about 50 sec^{-1} . This value was verified in subsequent numerical analyses, where the final form of the polyurethane geofoam model was utilized.

Equation (3) was used to estimate a stress-strain relation for a strain rate, $\dot{\epsilon} = 50 \text{ sec}^{-1}$, following a method proposed by Christou (2010). Figure 4 shows plots of stress versus strain rate using data from stress-strain plots (Figure 3) for three different strain rates. Plots were developed for 15 different strain levels between 2% and 65%. However, only five are shown in Figure 4 for sake of clarity. It can be seen that the log-log plots of stress vs. strain rate show an almost linear relation, varying with strain level. Values of a and b for 15 different strain levels were identified from the plots. Next, these values were used in equations (1) and (2) to calculate the stresses corresponding to those 15 strain levels. The stress-strain plot for a strain rate of 50 sec^{-1} , developed by this method, is shown in Figure 5.

A curve-fitting method was applied to this stress-strain plot to develop a suitable material model for use in numerical analyses. A 5-parameter Mooney-Rivlin hyperelastic material model was developed. The strain energy function for the 5-parameter Mooney-Rivlin model is shown in equation (4), from ANSYS (2011a). The parameters C represent material constants, which characterize the deviatoric deformation of the material. The terms $\bar{I}$ are stress invariants.

$$\psi = C_{10}(\bar{I}_1 - 3) + C_{01}(\bar{I}_2 - 3) + C_{11}(\bar{I}_1 - 3)(\bar{I}_2 - 3) + C_{20}(\bar{I}_1 - 3)^2 + C_{02}(\bar{I}_2 - 3)^2 + \frac{1}{d}(J - 1)^2 \quad (4)$$

The initial shear modulus (G) is defined as in equation (5):

1 $G = 2(C_{10} + C_{01})$ (5)

2

3 The initial bulk modulus (K) is defined as in equation (6):

4

5 $K = \frac{2}{d}$ (6)

6

7 The values of parameters were obtained from curve-fitting on ANSYS Workbench
8 (2011a). A plot of this model, shown in Figure 5, can be seen to compare favorably
9 with the estimated stress-strain results for for a strain rate of 50 sec^{-1} . Table 1
10 presents the values of parameters of the Mooney-Rivlin hyperelastic model used to
11 represent the polyurethane geofoam in the numerical analyses.

12

13

14 **5. NUMERICAL MODELING**

15

16 *5.1. Solvers*

17

18 The numerical analyses were conducted utilizing ANSYS Autodyn (2011b), which is
19 a hydrocode, executing a non-linear explicit dynamic computation. Three-
20 dimensional analyses were conducted, utilizing a fully-coupled Euler-Lagrange
21 solver. This approach has been previously reported by Choi et al. (2006) in modeling
22 the effects of explosions on tunnels. An Euler solver is used to model the fluid

1 phases (air and TNT), while a Lagrange solver is used to model the solid phases
2 (geofoam barrier, soil, and tunnel). The coupling of the two solvers allows large
3 deformations to occur in the fluid phase, while utilizing the solids to provide
4 boundaries which interact with the fluids. Thus, realistic fluid-structure interaction is
5 modeled even as large deformations occur, which is critically important while
6 studying the effects of blast loading on structures.

7
8 The three-dimensional finite element model is shown in Figure 6. Planes of
9 symmetry along the two vertical faces were utilized to allow modeling only one-
10 quarter of the full model, thus decreasing the run-time for each model. The axial and
11 transverse sections, shown in Figure 1, are drawn along these two planes of
12 symmetry. The one-quarter model consisted of 330,048 elements for soil, 19,872
13 elements for the barrier, 7,344 elements for the tunnel, and 1,397,250 elements for the
14 air.

16 *5.2. Material Model: TNT*

17
18 TNT was modeled as a high explosive, with an equation of state known as Jones-
19 Wilkins-Lee (JWL) and previously used in ANSYS Autodyn by Luccioni et al.
20 (2006). In this form, the rapid expansion of the explosive, following detonation, is
21 actually modeled as an equation of state with a rapidly expanding volume. This
22 creates pressure within the Euler part (air), which in turn, interacts with the Lagrange
23 parts (soil and tunnel), causing deformations. Thus, the effects of explosion are

1 directly created using a high explosives model and not indirectly applied by means of
2 an equivalent pressure.

3
4 The air part serves as the medium within which the rapid expansion of the explosive
5 takes place. A “flowout” boundary condition was assigned on the external
6 boundaries (i.e., all boundaries except the two planes of symmetry). The size of air
7 part was determined based on trials to determine the optimum height at which the
8 overall dimension does not have any influence on the effects of the explosion.

9
10 The explosive was introduced in the model by means of a “fill file”, described in
11 ANSYS (2011b) and used previously by Ackland et al. (2013), Choi et al. (2006), and
12 De (2012). A spherical body of charge was located immediately over the ground
13 surface at the geometric center of the model. Since a quarter of the full model was
14 analyzed (due to symmetry), a quarter of a sphere of charge was located at one edge
15 of the model, along the planes of symmetry, as seen in Figure 6.

16 17 *5.3. Material Model: Dry Sand*

18
19 The default sand model available in ANSYS Autodyn is based on one for dry sand
20 proposed by Laine and Sandvik (2001) In the tests reported here, dry Nevada #120
21 sand was compacted at approximately 60% relative density. This sand was
22 extensively characterized through static and dynamic laboratory testing as part of a
23 prior research initiative, reported by Arulmoli et al. (1992). The default Autodyn

1 model by Laine and Sandvik (2001) was modified to incorporate behavior
2 characterized in Arulmoli et al. (1992) to develop a new model for dry sand that was
3 first proposed by De (2012). This soil model was utilized in the numerical modeling
4 reported here, with minor modifications, as noted.

5
6 A brief description of this model is presented here. For more detailed information,
7 the reader is referred to De (2012). Consistent with the format of constitutive models
8 utilized in ANSYS Autodyn, the soil model consists of an equation of state, a strength
9 model, a failure model, and an erosion model.

10
11 The equation of state defines a relationship between density and pressure. A
12 compaction equation of state was utilized in these tests, with values for density,
13 pressure, and speed of sound based on laboratory testing. It is noted that the original
14 equation of state (shock) was modified to a compaction model, based on calibration
15 of test results.

16
17 The strength model utilized was MO Granular, which is the same as one originally
18 proposed by Laine and Sandvik (2001), with modifications to match the soil
19 properties reported by Arulmoli et al. (1992) through static and cyclic laboratory
20 tests. Table 2 presents a summary of the material properties utilized in the Autodyn
21 model to characterize the soil.

22

The failure and erosion models were set to default values, as specified in Laine and Sandvik (2001).

It is recognized that the results of the analyses are dependent on the properties of the soil model. However, the soil model has been kept constant in this study, in order to focus on the effects of the protective barrier. For more details on the soil model, the reader is referred to De (2012).

5.4. Material Model: Polyurethane Geofoam

A description of the properties of the polyurethane geofoam used in the analyses has been presented in a prior section. Table 1 presents a summary of the material properties utilized in the Autodyn model to characterize the geofoam. It is important to note that the material properties originally derived from 1-D laboratory testing have been extrapolated to high strain rates and are now being used in 3-D numerical analyses. This inherently introduces some approximation, which is not determined at this time.

5.5. Material Model: Copper

The material properties for Copper were based on Autodyn default properties (ANSYS, 2001b). They consist of a reference density of 8.9 g/cm^3 , a Shock Equation

of State (EOS) and a Piecewise Johnson-Cook strength model, with a shear modulus of 4.64×10^7 kPa.

6. RESULTS

6.1. Configurations compared

This section contains comparisons to investigate the effect of thickness of the protective geofoam barrier. In this case, the effects of an explosion on the structure are evaluated based on the following parameters:

- Magnitude of peak axial strain at top of the tunnel, measured along the long axis of the tunnel
- Magnitude of peak circumferential strain at top of the tunnel, measured in the hoop direction of the tunnel
- Maximum effective plastic strain measured at the top of the tunnel
- Peak pressure experienced at the top of the tunnel
- Maximum von Mises stress at the top of the tunnel

Strains induced in the tunnel due to an explosion are suitable indicators of damage. Tunnel designers can compare induced strains against the threshold of acceptable strains to determine whether the explosion would cause significant damage (such as breach). Similarly, peak pressure and von Mises stress are indicative of the level of

load applied and may be compared with design limits. Choi and Munfakh (2006) and Liu (2012) have modeled conditions in underground tunnels subjected to explosions. They have reported levels of tunnel damage with reference to similar indicators, such as effective plastic strain, von Mises stress, and peak pressure.

With reference to the axial and circumferential strains, the results of three-dimensional numerical modeling using Autodyn have been compared with data from individual centrifuge tests, where available.

Figure 1 shows a longitudinal and transverse section through the model. From the ground surface down, there exist a soil cover, protective geofoam barrier, tunnel, and bedding soil. Figures 6 and 7 show a configurations where the soil cover was 0.9 m in prototype scale. This is a relatively shallow depth and represents a case where a tunnel is emerging from the underground to the ground surface. It should be noted that the dimensions were not chosen based on any specific field condition; rather, they were selected as reasonable values for which measurable tunnel deformations could be modeled, utilizing the size of explosives available for testing. Since the centrifuge tests were conducted at 70 g, 1.8 m in prototype dimension corresponded to a model dimension of 25.4 mm (1 inch).

6.2. Deformation due to explosion

Figures 7a, 7b, and 7c show the model deformation following an explosion. In this case, the deformations are shown 100 ms (milliseconds) after the explosion. Figure 7a shows the deformation on the soil surface, in the form of a crater. There is also noticeable deformation in the protective geofoam barrier and tunnel, mostly in the form of vertical displacement.

6.2.1. Strain gage measurements

Strain gages were utilized to measure strains experienced at different locations of the model tunnel during centrifuge tests. Plots of strain gage measurements versus time from experimental and numerical models are compared in Figures 8a and 8b. These strain results are from a test with the following configuration (dimensions given in prototype scale):

- 0.9-m thick soil cover
- 1.33-m thick polyurethane foam barrier

Strains shown here were measured at the top of the model tunnel midspan. The axial strain gauge measured strains along the longitudinal axis of the tunnel, which was denoted as CTX. The circumferential strain gage measured strains along the hoop direction of the tunnel, which was denoted as CTH. The horizontal axis indicates elapsed time on a prototype scale, which is centrifuge time multiplied by the scaling factor, (70 in this case).

Also show on the figures are the plots of corresponding strains measured at the same location on the numerical model of the same configuration. Because of the very large number of elements, the numerical model was computation-intensive and simulated only a short interval of time after the blast. Therefore, the plots for numerical model do not extend the same duration as the experimental results.

From Figure 8a, it can be seen that the magnitude of peak strain recorded at CTX is about 0.80%. The peak negative strain measured in the numerical model closely matches the one from centrifuge experiment. The numerical model also shows a relatively high positive strain, which is not present in the experimental result. The experimental result indicates that a permanent strain of about 0.20% remained in the tunnel even after the explosion.

Figure 8b shows that the peak strain recorded in CTH in the centrifuge experiment has a magnitude of 0.83%. The result from the numerical model closely matches the experimental measurement. The experimental result indicates that a permanent strain of about 0.11% remained after the explosion. The numerical model appears to indicate a higher permanent strain, but the result does not extend sufficiently to make a comparison. As indicated earlier, the goal of this study was to identify and compare the peak strains in the experimental and numerical models. Longer duration simulations, with optimized mesh, would be necessary to model residual permanent strains after the blast.

6.2.2. *Effect of thickness of compressible protective geof foam barrier*

Figure 9 shows a plot of peak axial strains recorded at CTX (in %), varying with thickness of protective barrier in prototype scale. Also shown on these graphs is a point representing data from a centrifuge model test. Figure 9 shows a plot for a constant thickness of soil cover equal to 0.9 m, in prototype scale. This corresponds to 0.5 inch (12.5 mm) in model scale, in a 1:70 scale model, tested at 70 g.

The value for protective geof foam barrier thickness equal to zero represents a model with only soil cover, i.e., the tunnel is located directly 0.9 m below the ground surface, with no protective barrier. The highest magnitude of peak axial strain is obtained for this configuration. The remaining points in the graph are for configurations with various thicknesses of protective barrier. As expected, the magnitude of peak axial strain decreases with increasing thickness of protective geof foam barrier layer.

From Figure 9 it can be seen that the peak strain recorded at CTX for a case without any protective barrier (“Soil only”) is 7%, based on results of numerical analyses. In the corresponding centrifuge experiment for the “Soil only” configuration, the recorded strain was beyond the measuring limits of the instrument. The data points with various thicknesses of polyurethane geof foam barrier indicate a relatively large decrease in peak axial strain with small increase in barrier thickness initially. Beyond a barrier thickness of 1 m, the peak axial strain becomes less than 1% and does not

1 appear to decrease further with increases in barrier thickness. From a design
2 standpoint, this transition point would indicate an optimum barrier thickness, beyond
3 which no incremental benefit can be obtained by further enhancing the barrier
4 thickness.

5
6 The data point obtained for a centrifuge test conducted with a 1.34 m thick geofoam
7 barrier indicates a peak strain recorded at CTX of about 0.81%, which is the peak
8 value in the plot on Figure 8a. The result of the numerical modeling for the same
9 thickness of geofoam barrier indicated a peak axial strain which compared favorably
10 with the centrifuge test result. Similar close matches were observed in measurements
11 on strain gages located at the bottom of the model and at a point half-way between
12 the center and end (quarter-span).

13 14 *6.2.3. Effect of thickness of soil cover above tunnel*

15
16 Figure 10 shows the variations in peak strain at CTX (axial direction) with thickness
17 of protective cover thickness, for different values of soil cover thickness above the
18 tunnel – varying from 0.9 m to 2.7 m. Where available, centrifuge results are also
19 plotted on the same graph. It can be seen that the peak axial strain generally reduces
20 with an increase in protective cover thickness and increase in soil cover thickness.

21
22 As the thickness of protective geofoam barrier increases, the magnitude of peak strain
23 at CTX first decreases rather sharply and then the rate of decrease becomes smaller.

1 There is no further reduction in strain when geofoam barrier thickness is increased
2 beyond a certain critical thickness. This critical thickness, at which any increase in
3 protective barrier thickness does not have any measurable beneficial effect, varies
4 depending on the soil cover thickness. For soil cover thickness of 0.9 m, the
5 thickness of protective geofoam barrier ceases to have any effect beyond a thickness
6 of 1 m.

7
8 For soil cover thickness of 2.7 m, the inclusion of a protective geofoam barrier (of
9 any thickness) does not appear to have any beneficial effect, i.e., the strain remains
10 approximately unchanged with increasing barrier thickness. This result was obtained
11 for a constant mass of explosives, which was 900 kg of TNT equivalent in prototype
12 scale. The critical thickness of protective geofoam barrier and soil cover would be
13 expected to change for other values of explosive mass.

14
15 The results of centrifuge tests (represented by points on the plot) appear to follow the
16 same trends and closely match the results of numerical analyses (shown as lines). In
17 addition to the point for a soil cover thickness of 0.9 m (which are discussed in
18 connection with Figure 9), two more data points from centrifuge tests are shown on
19 this figure. For a soil cover thickness of 2.7 m without any protective geofoam
20 barrier ("Soil only") the strain measured in the centrifuge test at CTX was 0.64%,
21 while the result of numerical analysis for the same case indicated a strain of 0.47%.
22 For a soil cover thickness of 1.8 m and a protective geofoam barrier thickness of 1.33

m, the strain measured in the centrifuge test at CTX was 0.15%, while the result of numerical analysis for the same case indicated a strain of 0.12%.

It is noted that in some cases (e.g., for soil cover of 1.8 m without any geofoam protection) the strain gage reading was out of measurable range and therefore, not shown on the plot.

6.2.4. Effect of barrier thickness on other parameters

Figure 11 shows the variation in peak strain at CTH (circumferential or hoop direction) with total thickness of protective barrier, for different values of soil cover thickness above the tunnel. A trend similar to CTX, discussed earlier, is seen here, with the thickness of barrier beyond 1.33 m having little effect on the strain.

The results of numerical analyses show excellent agreement with the result from the centrifuge test for 0.9 m soil cover (0.78% in numerical analysis, compared with 0.83% measured in centrifuge test, both for a 1.33 m protective barrier). However, the numerical analyses indicate smaller strains for 1.8 m soil cover as compared to the centrifuge tests. The reason for this deviation is not clear, however, it is noted that the centrifuge test results for 0.9m and 1.8 m are almost identical – which could perhaps be attributed to error in instrumentation.

Figure 12 shows peak value of effective plastic strain on top of the tunnel at midspan varying with thickness of protective barrier. A similar trend is observed in these plots, with the peak plastic strain decreasing with increasing barrier thickness. For a 0.9 m thick soil cover, the peak plastic strain decreases from 12% to 0.6% as geofoam barrier thickness increases from zero ("Soil only") to 2.22 m. The plots for other values of soil cover thickness show the same trend.

Figure 13 the peak pressure on the top of the tunnel at midspan (which is directly below the explosion), varying with thickness of the protective barrier. As in the previous cases, the peak pressure decreases with thickness of barrier and is not affected by the barrier thickness beyond a thickness of about 1 m. The maximum value of peak pressure of 105 MPa is recorded for a 0.9 m thick soil cover, without any protective geofoam barrier ("Soil only"). For the same thickness of soil barrier (0.9 m), the peak pressure reduces to 66 MPa when a 2.22 m thick protective barrier is installed.

Figure 14 shows the shows the peak value of von Mises stress on top of the tunnel at midspan varying with thickness of protective barrier. As in the other cases, the peak von Mises stress decreases as thickness of protective barrier increases. In case of a 0.9 m thick soil cover, it changes from a value of 250 MPa without any protective geofoam barrier ("Soil only") to a value of 127 MPa when a 2.22 m thick protective barrier is installed.

6.2.5. Effect of quantity of explosive

The results presented in the preceding sections (for both the numerical analyses and physical model tests) were all obtained for a constant mass of explosives equal to 900 kg of TNT equivalent. It is reasonable to expect that the damage to the structure will have a direct relation to the mass of explosives. Thus, less damage (in the form of lower strains and reduced pressure/stress) will result from an explosive with a smaller mass of TNT equivalent.

The dependence of damage on the mass of explosives for different barrier thicknesses was studied in a limited number of analyses. For this, the strains on a tunnel with a constant soil cover thickness of 0.9 m and different thicknesses of protective barrier thickness was analyzed for four masses of explosives: 300, 600, 900, and 1200 kg (respectively) of TNT equivalent.

Figures 15 and 16 show the plots of peak axial strain on top (CTX in %) and peak circumferential (hoop) strain on top (CTH in %) respectively, varying with thickness of protective barrier in prototype scale for different masses of explosives. All results shown on this figure are for a soil cover thickness equal to 0.9 m in prototype scale. Also shown on each graph is a point representing data from a centrifuge model test for a 1.34 m thick geofoam barrier. The data points for CTX and CTH from

centrifuge tests have previously been discussed in connection with Figures 9 and 11, respectively.

7. CONCLUSIONS

The beneficial effects of a compressible geofoam barrier in reducing the effects of a surface explosion on an underground tunnel were demonstrated through centrifuge model tests and numerical analyses. The protective geofoam barrier, made of polyurethane, was modeled in numerical analyses using a Mooney-Rivlin hyperelastic model, developed through an extrapolation of data obtained from uniaxial compression tests in the laboratory.

The effects of a surface explosion on the tunnel were evaluated based on various parameters, such as peak axial and circumferential strains, maximum plastic strain, peak pressure, and maximum von Mises stress. The values of axial and circumferential strains obtained through numerical analyses were compared with those measured in centrifuge tests. Favorable comparisons were obtained between numerical and experimental results for the limited number of cases tested.

The results indicate a decrease in the effects of an explosion (such as measured strains, stresses, and pressures) at the top of the tunnel as the thickness of the protective geofoam barrier was increased, while keeping the overall soil cover thickness unchanged. The results also include configurations where no protective cover was utilized. This study demonstrates that it is possible to significantly reduce the effects of explosion on an underground tunnel by installing a protective geofoam

1 barrier. It also demonstrated that there is an optimum barrier thickness, beyond
2 which any increase in thickness does not result in measurable incremental benefit
3 (i.e., there is no further reduction of strains, stresses and pressures).

4
5 Results of analyses with different masses of explosives show that generally, the level
6 of damage is directly related to the size of explosive, for the same thickness of
7 geofoam barrier.

8
9 Continued work in this topic may include results for different soil cover thicknesses,
10 such that design charts of measured strains versus protective cover thickness can be
11 developed for different values of soil cover. This will help identify optimum design
12 thicknesses for a given site condition and design explosive amount. Additional
13 centrifuge tests on protective geofoam cover thickness less than the critical thickness
14 would provide additional validation of the numerical model.

15
16 Material characterization of the polyurethane geofoam through high-strain rate
17 laboratory testing was beyond the scope of this study. The hyperelastic material
18 model used in the numerical analyses was developed based on results of laboratory
19 compression tests at different strain-rates, which were extrapolated utilizing
20 previously-established relations. The results of numerical analyses were found to
21 match closely those from centrifuge tests and thus, the material model was considered
22 adequate for this purpose. A further extension of this study would be to calculate

effects on the tunnel due to different material properties and higher strain rates, as well as to identify the optimum material properties required for enhanced protection.

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Reference density, ρ	1.265 g/cm ³
Equation of state (EOS)	Hyperelastic
Strength model	Hyperelastic 5-parameter Mooney-Rivlin
Material Constant: C01	5.91E+03 kPa
Material Constant: C02	5.11E+06 kPa
Material Constant: C10	-5.81E+03 kPa
Material Constant: C11	-8.46E+06 kPa
Material Constant: C20	3.54E+06 kPa

- 3 Table 2. Material Properties of Soil used in Numerical Analyses on ANSYS
- 4 Autodyn: Details provided in De (2012)

Reference density, ρ	2.641 g/cm ³
Equation of state (EOS)	Compaction
Strength model	MO Granular
Hydro tensile limit, $P_{\min}$	-1 kPa
Reheal	Yes
Erosion	Geometric strain = 2.00

Figures:

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(b)

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(b)

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