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ABSTRACT

Experimental tests of geo-materials interacting with rigid bodies under impact loading were conducted to collect critical information about the behavior of such systems. Full-scale vehicular tests were conducted to assess the performance and behavior of single boulders embedded in AASHTO uniformly-graded coarse aggregate soil as an anti-ram vehicle barrier, while drop weight tests were conducted on expanded polystyrene crushable concrete to develop a material model capable of incorporating strain rate effects for use in vehicle barrier designs. Numerical models were developed for each experimental test to calibrate material constitutive models.

Four full-scale vehicular tests were conducted to assess the performance and behavior of single boulders embedded in compacted AASHTO soil. LS-DYNA numerical models were developed and a soil constitutive model was calibrated based on a full-scale test with minimal soil movement. Each full-scale test progressed to larger boulder translation and rotation and soil deformation with the last test exhibiting boulder rotation out of the ground. Two modeling techniques were recommended based on the expected boulder and soil deformation. For small deformations, traditional finite element method (FEM) can be used for the boulder and soil domain. For large deformations, a hybrid approach combining Smoothed Particle Hydrodynamics (SPH) near areas of large deformation and FEM in areas of minimal deformation for soil was developed. Both numerical methods used the same calibrated constitutive model, Mohr-Coulomb failure criterion, for compacted AASHTO soil.
A crushable concrete mix design was tested where expanded polystyrene was used as the primary cell material to take advantage of its energy dissipating properties. Quasi-static and dynamic drop weight tests were conducted on fully confined specimens to help determine material constitutive properties for numerical modeling. A LS-DYNA model, using Material Type 16, “Pseudo Tensor”, was developed to simulate both the quasi-static and dynamic drop weight tests. The numerical model was able to capture the overall responses of the specimens during the quasi-static testing and consecutive impacts from the dynamic drop weight tests. Several compressed specimens from the dynamic drop weight test, representing different strain levels, were studied using non-destructive X-ray CT imaging to ascertain damage levels. From the CT scans, the volume of concrete with respect to voids along the height of each specimen could be determined. As the level of dynamic compression increased, the volume of concrete increased and the voids decreased, showing that the polystyrene beads were being crushed. Also, the bottom of the specimen exhibited higher amounts of crushing of the polystyrene beads. This was likely due to reflective waves from the bottom steel platen used to support and confine the concrete specimen.
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I would like to express my sincere gratitude to my advisors, Dr. Tong Qiu and Dr. Daniel Linzell for their unwavering support, guidance, patience and friendship. The positive atmosphere they have provided me during my career at Penn State has allowed me to grow tremendously as a student, as an engineer, and, most importantly, as a person. I would also like to thank my thesis committee members, Dr. Zoltan Rado, Dr. Sean Brennan, and Dr. Farshad Rajabipour, for additional support on this project. Next, I would like to give special thanks to Mr. Dan Fura, whose help in the lab was vital to the success of this project. I must also acknowledge the financial support provided by the U.S. Department of State.

I would also like to thank my research colleagues at Penn State. This research project was a collaborative effort and I would not have been able to complete it without the help of my fellow students, including Jiho Park, Yaneng Zhou, Eddie O’Hare, Sam Hoskins, Keith Doyle, Michael Bychkowski, Ryan Rudalavage and countless others.

Finally, I would like to thank my friends, family, and especially my parents, who I could never thank enough. Their unconditional love and support has helped me more than words can ever express.
Chapter 1

Introduction

1.1 Motivation

In recent years, terrorist attacks and security measures to stop or mitigate the damage they cause have been widely discussed and researched. With raised national security awareness due to these terroristic threats, more focus has been placed on safety systems that can prevent the loss of life and structural damage as a result of impact and blast loading. These systems rely on protective barriers to guard buildings and structures from potential vehicle attacks. The primary goal of these blockades is to stop a vehicle from penetrating critical assets.

The design of perimeter barriers has become critical for protecting buildings and facilities against vehicular impacts. Landscape Vehicular Anti-Ram (LVAR) systems are a group of protective barriers that effectively protect against threats using natural materials and, as a result, can be aesthetically pleasing in certain environments. To date, these systems have included single boulders embedded in soil, boulders placed on-grade, and groups of boulders connected below grade to act as an integrated anti-ram system.

The development process of these systems include: a conceptual idea implemented into a numerical model, performance assessment of the model, and then field-scale testing to validate the barrier design. The numerical model has several critical components that need to be precisely modeled to accurately predict the barrier response in field-scale tests.
These parameters include material constitutive models of the boulder and soil, overall global response of the truck including material crushing, interaction between the truck and boulder, and interaction between the boulder and soil. Once a barrier design has passed a threshold criterion numerically, field-scale testing according to American Society for Testing and Materials (ASTM) Standard Test Method for Vehicle Crash Testing of Perimeter Barriers (F2656-07) (ASTM 2007) is conducted to assess the physical performance of the barrier.

Two important factors play a deciding role in the design of LVAR barriers including constructability and constraints of the site where the barrier or barrier system will be installed. The constructability is an important design consideration because often materials are unavailable in certain areas of the world as well as specialized equipment to install the barriers. With that, the overall size and weight of the barrier system need to be constrained by the equipment available to lift and install the system. By limiting the overall size and weight of a barrier, a larger demand is created on the foundation to resist the overall impact force from a vehicle with a moderate foundation deformation. With a higher demand on the foundation and allowing a moderate deformation, traditional methods using Finite Element Method (FEM) for modeling of soil may become inaccurate and lead to overconfidence in the predicted performance of a barrier system. This overconfidence can lead to a failed field-scale test. Introducing a meshless formulation, such as the Smoothed Particle Hydrodynamics (SPH) method, to the numerical model may be advantageous for simulating the response of complex, physical systems involving large deformations.

The second factor that influences the overall design of a LVAR barrier system is the site location and the constraints each location proposes. As the world continues to
urbanize and sprawl, the need for security barriers in confined areas increases. The site location directly affects the size and depth of the barrier or foundation, soil conditions, and whether or not sub-grade connections can be made for systems of barriers. Taking each of these into consideration, the demand for smaller boulders with smaller foundations is often needed. Development of new materials to be used for foundations that can absorb larger amounts of energy compared to traditional soils is becoming a primary need. These new materials need to be adequately modeled numerically, particularly the strain-rate effects on their mechanical response, first before they can be implemented into LVAR barrier designs.

1.2 Objectives

The primary objective of this research is to develop numerical models that can accurately predict the response of vehicle barrier systems and geo-materials, including single boulders embedded in standard AASHTO gravel soil and expanded polystyrene crushable concrete, under impact loading by calibration and validation using experimental testing. The secondary objective is to develop modeling procedures, including loading and unloading with material memory, and recommendations for soil and crushable concrete that interact with rigid bodies and undergo large deformations under impact loading.
1.3 Scope

The scope of this study was to investigate the system response and modeling techniques of a vehicle barrier system involving a boulder embedded in compacted AASHTO soil, and the mechanical behavior of expanded polystyrene crushable concrete subjected to impact loadings. These numerical models were based on full-scale experimental tests. A numerical model was calibrated to predict the response of a single boulder embedded in compacted AASHTO coarse aggregate fill subjected to a vehicular impact. This model was then validated using a full-scale experimental field-test. Since large deformation is not uncommon in barriers subjected to vehicular impacts, modeling techniques using Smoothed Particle Hydrodynamics (SPH) were developed to accurately predict large deformations under impact loading. Lastly, the development of a material constitutive model with strain rate effects, capable of absorbing large amounts of energy, was developed due to constraints placed on barrier designs.

1.4 Dissertation Organization

This thesis is presented in five chapters. Chapter 1 includes background information, motivation for this research and objectives. Chapter 2 presents field tests and numerical modeling of vehicle impacts on a boulder embedded in compacted fill. This chapter calibrates a soil model using Mohr-Coulomb failure criterion and validates a field-scale test using the calibrated model. Chapter 3 further develops the model calibrated and validated in Chapter 2 to incorporate high-fidelity modeling techniques using Smoothed Particle Hydrodynamics (SPH) to accurately predict large soil deformations. Chapter 3
provides modeling recommendations to accurately predict large deformations seen in optimized LVAR barrier systems. With optimization of LVAR barriers, the need for new materials used in the foundation for greater energy absorption are needed. Chapter 4 develops a material model for expanded polystyrene crushable concrete, a proven energy absorbing material, which accounts for rate effects that would be encountered in LVAR barrier systems. The material constitutive model also accounts for loading and unloading along the stress-strain curve with permanent deformations. Chapter 5 summarizes the findings of each of the studies and provides suggestions for future work.
Chapter 2

Field Tests and Numerical Modeling of Vehicle Impacts on a Boulder Embedded in Compacted Fill

2.1 Abstract

Landscape Vehicular Anti-Ram (LVAR) systems are a group of protective barriers that effectively protect sensitive structures against threats using natural materials (e.g., boulders) and, as a result, can be aesthetically pleasing. This study presents two consecutive vehicular crash tests hitting the same single boulder embedded in AASHTO coarse aggregate fill. The field-scale tests were instrumented with high speed cameras and pressure cells. A LS-DYNA model was developed to simulate the field-scale tests. A readily available truck model from the National Crash Analysis Center was modified and implemented in the LS-DYNA model. The boulder and surrounding soil were modeled using Mohr-Coulomb failure criteria model. The model parameters were calibrated using results from Test 1 with a truck traveling 30 mph. The calibrated model was then used to simulate Test 2, which involved a truck traveling at 50 mph without resetting the boulder or soil. The calibrated model was able to yield the global response of the system including the time-history of the translational displacement and rotation of the boulder and was in good agreement with field-scale test results. This suggests that the overall global response was dominated by the dynamic behavior of the truck and boulder system upon impact. Hence, a simple material model for soil and boulder is sufficient for simulating the tests conducted.
2.2 Introduction

The development of perimeter barriers has become critical for protecting buildings and facilities against vehicular impacts. Landscape Vehicular Anti-Ram (LVAR) systems are a group of protective barriers that effectively protect against threats using natural materials (e.g., boulders) and, as a result, can be aesthetically pleasing. Typical bollard systems are a collection of anti-ram barriers that typically use bollards with a deep, stiff foundation to achieve a proper standoff distance. The development process of these systems includes field-scale testing and numerical modeling to effectively design and validate these systems subjected to vehicular impacts. Field-scale tests involve using medium-duty trucks for vehicular impacts into these systems to validate each of the systems designed based on numerical modeling. Abundant field-scale testing and numerical modeling have been conducted on typical bollard systems (O’Hare et al. 2012, Uzzolino et al. 2012, Omar et al. 2007, Krishna-Prasad 2006, Hu et al. 2011, Liu et al. 2008, and Ferdous et al. 2011); however, limited research exists for LVAR systems (Reese et al. 2012).

In order to adopt a unified test method to test the crashworthiness of vehicle barriers such as anti-ram devices, the U.S. Department of State (DOS) established a standard for device designs, the Specification for Vehicle Crash Test of Perimeter Barriers and Gates (SD-STD-02.01) (U.S. Department of State 1985) in 1985. SD-STD-02.01 specified preliminary criteria related to the test vehicle (diesel or gas engine medium-duty truck (6800 kg)), ballast attachment, impact velocity and penetration distance. With the use of small sized cars in more recent terrorist attacks, the American Society for Testing and
Materials (ASTM) established a new standard—*Standard Test Method for Vehicle Crash Testing of Perimeter Barriers* (F2656-07) (ASTM 2007). This test method provides a structured procedure to establish a penetration rating for perimeter barriers subjected to vehicle impact as well as site condition requirements during installation and testing and standard vehicular energy requirements. ASTM F2656-07 test standards have been used in numerous research projects including those tested at the Larson Transportation Institute (O’Hare et al. 2012, Uzzolino et al. 2012, and Reese et al. 2012) and the Texas Transportation Institute (Briad et al. 2010).

Numerical modeling of anti-ram barrier systems has been generally conducted using LS-DYNA (Hallquist 2013), which is known to be a reliable program for modeling vehicular impact and contains a library of constitutive models developed for high-rate loads for a number of materials. LS-DYNA also has several different contact formulations that are effective in high-strain rate impacts including single-surface, one- and two-way contacts. These common contact formulations accurately maintain compatibility between parts within the model. Numerous researchers have investigated typical bollard systems using LS-DYNA as a means to validate field-scale tests (O’Hare et al. 2012, Uzzolino et al. 2012, Omar et al. 2007, Krishna-Prasad 2006, Hu et al. 2011, Liu et al. 2008, and Ferdous et al. 2011). O’Hare et al. (2012) developed Streetscape Vehicular Anti-Ram (SVAR) systems with shallow foundations, including street benches, bus stops and street signs, which were able to conform to site restrictions, use cost effective standard materials, and accommodate varying architectural aesthetics. The design of the SVAR systems was optimized using LS-DYNA and then validated through field-scale testing. Other researchers validated DOS standard impact conditions for anti-ram bollards known as K4,
K8, and K12 through field-scale testing; using LS-DYNA, researchers were able to offer recommendations and modifications for each of the barriers after validation since the model simulations predicted the standard impact conditions (Omar et al. 2007).

Research conducted at the Larson Transportation Institute, affiliated with the Pennsylvania State University, on the design and performance of a LVAR system consisting of a boulder embedded in compacted fill will be summarized herein. Two field-scale tests were conducted and used to rate the performance of barrier systems against M30 and M50 impacts. Due to minimal movement of the boulder and displacement of the surrounding soil at M30, a M50 test was conducted to assess the performance of the system at a high rate without replacing the boulder or surrounding soil. Using high-speed cameras, pressure cells, and field surveys the boulder movement and soil pressure were obtained from each test. A LS-DYNA model was developed to predict the overall global response of the system to vehicular impact and the simulated results were compared to the collected field data. In the following sections, the field-scale tests are described followed by descriptions of the LS-DYNA model and selection of model parameters. Lastly, results and discussions on the capability of the LS-DYNA model in capturing response of the LVAR system are presented. This study focuses on the numerical predictions and measured performance for a LVAR system in the field-scale. The results therefore provided useful basis for evaluating the predictive capabilities and limitations of the numerical analyses.
2.3 Field-Scale Testing

Vehicular impact tests were conducted according to ASTM F2656-07 - Standard Test Method for Vehicle Crash Testing of Perimeter Barriers (ASTM 2007), which establishes a penetration rating for perimeter barriers subjected to a vehicle impact. The test facility uses a rigid rail to provide vehicle guidance, a reverse towing system to accelerate the test vehicle to the required speed, and a release mechanism that disconnects the tow cable and steering guidance prior to impact. The propulsion system used to bring the test vehicle up to the desired impact speed consists of a tow vehicle, a tow cable, two re-directional pulleys anchored to the ground, a speed multiplier pulley attached to the tow vehicle, a quick-release mechanism, and a ground anchor. For a detailed description of the system, please refer to Reese et al. (2012).

Two consecutive, full-scale field tests were conducted and consisted of the same Rockville White (RW) granite boulder embedded in AASHTO uniformly-graded coarse aggregate soil. Test 1 was for a M30 impact (vehicular speed of 48.3 km/hr (30mph)) while Test 2 was for a M50 impact (vehicular speed of 80.5 km/hr (50mph)). Between the tests, the RW granite boulder was not reset and surrounding soil was not recompacted. Field surveys were completed before and after each test to determine the final placement of the boulder and each truck.

The test vehicles used in this study were medium-duty diesel trucks. The truck used in Test 1 was a 1999 International 4700 single-unit flatbed truck. Barrels with ballast were secured on the bed of the truck making the test inertia weight 6745 kg. The height to the lower edge of the front bumper was 0.48 m and the height to the upper edge of the front
bumper was 0.75 m. The second truck used was a 1999 International 4700 single-unit flatbed truck. Test inertia weight of the vehicle was 6827 kg. The height to the lower edge of the front bumper was 0.48 m and the height to the upper edge of the front bumper was 0.76 m.

According to ASTM F2656-07 (2007) standards, during the install of the RW boulder, the excavation should extend along the horizontal direction behind the boulder to a distance equal to 1.5 times the boulder embedment depth (ASTM 2007). As shown in Figure 2-1, this criterion is satisfied. Prior to each test, the extent of excavation was surveyed and the geometry was utilized as an input parameter for the numerical simulations to be discussed later. Prior to boulder placement, the AASHTO uniformly-graded coarse aggregate soil was placed into the excavated pit to the desired elevation in lifts of approximately 0.2 m and compacted using a tamping compactor. The boulder was then lowered into the pit and centered for the impact. Similar procedures were followed to backfill the pit with compacted aggregates.

The RW granite boulder was approximately 1.98 m wide (W) x 1.68 m long (L) x 3.43 m high (H) as shown in Figure 2-1. The total embedment depth (D) was 2.03 m. The approximate weight of the boulder was 27,200 kg. No natural fissures or joints were observed in the boulder. Based on the information provided by the quarry, the bulk density and compressive strength of the boulder were approximately 2696 kg/m3 and 142 MPa, respectively (Cold Spring Quarry 2014). Small-scale testing was completed to confirm these values and will be briefly discussed in a subsequent section.
The field tests were instrumented with high-speed cameras to record the motions of the truck and boulder immediately prior to and after the crash and pressure cells were embedded in the soil to monitor pressure variations during the tests. The high-speed camera systems are fully autonomous and can capture images up to 29,000 frames-per-second with a maximum resolution of $1016 \times 1016$. Three cameras (see Figure 2-2) were used during each test and they were positioned (1) at a $90^\circ$ angle and 20.5 meters to the center of the test article to measure dynamic penetration; (2) at a $90^\circ$ angle and 15.9 meters above the center of the test article to capture enough surface area prior to and after impact to determine impact speed, impact angle, exit angle and debris field; and (3) behind the test article at a distance of 32.2 meters centered along the guidance rail to record the approach of the test vehicle to track its alignment with respect to the center of the test article. Specific points on the impacted system were tracked using fiducials that allow tracking of critical points during impact. High-speed video and still images from Camera 1 were utilized to determine boulder movement time history for comparison with LS-DYNA simulations.
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Figure 2-2. Location of high-speed video cameras during field-scale test (not to scale).
Figure 2-3. Still images from high-speed camera 1 at: (a) $t = 0$ sec; (b) $t = 0.1$ sec; (c) $t = 0.2$ sec; and (d) $t = 0.3$ sec.

Hydraulic-type pressure cells (Geokon 2011) were embedded inside the compacted aggregates to monitor earth pressure time histories at the front and back of the boulder during each impact. The cells use two flat plates, welded together at their periphery, and are separated by a small gap filled with a fluid. When a load is applied to the cell, the hydraulic fluid is squeezed between the two plates and a pressure build occurs with the
change in pressure being converted to an electric signal. Resulting pressure-time histories can provide insight into the dynamic soil-boulder response upon impact and are utilized for numerical comparison to LS-DYNA simulations. General locations of the pressure cells are shown in Figure 2-1.

2.4 LS-DYNA Model

The LS-DYNA (Hallquist 2013) research/commercial code was utilized to perform Finite Element Method (FEM) simulations to model the vehicular impact tests. This code was chosen due to its proven capability in modeling impact events (O’Hare et al. 2012, Omar et al. 2007, Liu et al. 2008, Reese et al. 2012, and Briad et al. 2010). The FEM model is shown in Figure 2-4, which details the different mesh densities used for the truck, boulder, and soil domain. This section will briefly discuss the truck, boulder, and soil constitutive models used in the numerical simulation as well as stress initializations used to simulate the initial condition of both tests under gravity loading and the initial condition of the M50 test after the M30 impact.
2.4.1 Truck Model

Field-scale frontal impact tests against anti-ram barriers have indicated that a truck typically absorbs approximately 70% of the impact energy for a M30 rated test (Omar et al. 2007). Therefore, a realistic truck model that can capture the correct amount of energy absorption during an impact event is essential. The truck model used for the simulations, shown in Figure 2-4, was modified from a model readily-available in the National Crash Analysis Center (NCAC) database (Mohan et al. 2003 and NCAC 2008). The FEM model was based on a 1996 Ford F800 single unit truck as shown in Figure 2-5(a) and was
developed to meet the needs of roadside hardware research and development community while reducing computational requirements. The NCAC truck model was developed to ensure that the load transfer between the truck and hardware, the deformation of the truck, and the overall behavior of the truck during impact simulations would be as accurate as feasible given the model computational requirements.
Figure 2-5. FEM truck model modifications: (a) NCAC truck model; and (b) Modified truck model.

The truck requirement for ASTM F2656-07 is a Ford F650 or Ford F750, which was different than the NCAC truck model. Field-scale tests, however, utilized a flatbed medium duty truck with ejectable added cargo mass in the form of 55-gallon barrels filled with ballast and secured towards the front of the truck’s bed with ratchet straps.
Modifications to the truck model, as shown in Figure 2-5(b), were needed to meet the criteria of the standard. Joint revolution constraints were added to allow the wheels to rotate and truck ramping. The closed bed on the original truck was removed, leaving an open truck bed for the added ballast barrels. The ballast barrels were added to meet the required weight according to ASTM F2656-07. An accelerometer was added at mass center of the truck to monitor the response time history to compare vehicle dynamics from field-scale tests and numerical simulations. Last, definitions of conservative failure strains for all truck’s materials were added to prevent unrealistic energy absorption due to extreme deformation.

The modified truck model consisted of 1606 eight-noded constant stress solid elements, 20,318 four-noded Belytschko-Tsay shell elements and 201 Hughes-Liu with cross section integration beam elements. Three validation checks were conducted to ensure the modified truck model could capture the global response of the ASTM-specified truck under impact. First, the truck model was validated using checks of equilibrium by preloading the truck with a gravity load using LS-DYNA. The normal force on the rigid wall beneath the truck was then compared to the weight of the truck to ensure the mass of the truck satisfied ASTM F2656-07. Second, conservation of energy principles were used to ensure that excessive internal energy, such as hourglass energy, did not contribute to spurious numerical results. Third, using the numerical simulation from Test 1 (speed of 48.3 km/hr), the ratio of the internal energy of the truck after impact to the initial energy of the system (e.g., kinetic energy of the truck) indicated that the truck absorbed approximately 69% of the impact energy, which is consistent with the field-scale test
conducted by Omar et al. (2007). The absorbed energy is due to plastic deformation and fracturing of various truck components.

### 2.4.2 Constitutive Model for Boulder and Soil

The LS-DYNA Material Type 173, “Mohr-Coulomb (M-C)” (Hallquist 2013 and Livermore Software Technology Company (LSTC) 2006) was utilized to model the boulder and soil behavior in the FEM simulations. M-C model was used to represent the soil and boulder due to its ability to effectively and simplistically capture impact conditions present in experimental testing. Although M-C constitutive model is simple and cannot capture sophisticated post-failure constitutive behaviors such as the strain softening response and fracture propagation, it is considered sufficient to model the boulder behavior during the field tests as no damage to the boulder was observed and it essentially behaved as a rigid mass. The M-C is also considered sufficient in modeling the constitutive behaviors of the compacted granular fill as small deformations of the fill were observed after the impacts. The Mohr-Coulomb characterizes failure of a material based on its cohesion, normal stress on an element and friction angle as follows (Hallquist 2013):

\[
\tau_{\text{max}} = c + \sigma_n \tan(\phi)
\]  

(2.1)

where \( \tau_{\text{max}} \) is the shear strength on any plane, \( \sigma_n \) is the normal stress on that plane, \( c \) is cohesion and \( \phi \) is the friction angle. Model parameters for the boulder and soil are discussed later.
2.4.3 Preload and Stress Initialization

A springback analysis procedure readily available in LS-DYNA was utilized to preload the system with gravity so that the stress-dependent soil strength and conditions prior to impact were modeled correctly. Once a steady-state condition was met, information from the model was outputted and used for all subsequent modeling using a sequential analysis. Pressure distributions in the soil with respect to depth and total body equilibrium from the steady-state condition were checked to ensure that gravity was applied correctly.

A stress initialization procedure in LS-DYNA was utilized to model consecutive impacts on the boulder that, as discussed earlier, occurred without resetting the boulder or recompressing the soil. For the simulation of Test 1, sufficient analysis time was utilized to allow the system to reach a steady-state condition. Then, pertinent information from the end of the simulation, which included the deformed geometry of the soil-boulder system, residual stress and strain patterns within the soil-boulder elements, and boundary conditions of the soil domain, were carried over to the beginning of Test 2 simulation. An undeformed truck, using Test 2 speed, was then used in the numerical model for the Test 2 simulation.

2.5 Model Parameters

Data collected from Test 1, such as recorded boulder translations and rotations, were utilized to calibrate input parameters for the LS-DYNA model including constitutive parameters for the boulder and soil, contact algorithms, and domain size. Test 1 was chosen
for calibration due to the minimal rotational and translational movement of the boulder during impact. This section will discuss general parameters for contact algorithms used in the simulation, boulder material model to determine key parameters, and calibration of soil material parameters. The calibrated model is subsequently utilized to predict the global response of the LVAR system for Test 2 and compare with data collected from high-speed cameras and pressure cells.

2.5.1 Boulder Model

Laboratory tests were conducted at CITEL to determine the RW boulder material properties for numerical simulation under different conditions (e.g., quasi-static and high-strain rates) that would be subsequently implemented into computational models. These tests included uniaxial compression, split tension, Chevron bend, split Hopkinson pressure bar tests, and Schmidt hammer (International Society for Rock Mechanics (ISRM) 2007, Xia et al. 2011, and Katz et al. 2000). Uniaxial compression tests determined the ultimate compressive strength; split tension tests measured the uniaxial tensile strength; Chevron bend tests determined the fracture toughness of rock material and its Young’s modulus (ISRM 2007). In addition to these quasi-static tests, split Hopkinson pressure bar (SHPB) tests were conducted to evaluate the rate effects of material behavior (Xia et al. 2011). The Schmidt Hammer test was conducted in the field to quickly determine the compressive strength of rock material (Katz et al. 2000). Based on the tests conducted an initial shear modulus of 9480 MPa was determined, which is consistent with typical properties of granite published in literature (Gere and Timoshenko 1984 and Wyllie and Mah 2004).
The boulder domain was constructed using eight-noded constant stress solid elements. Each element was 100 mm x 100 mm x 100 mm. This mesh size was selected to create a smooth transition from the boulder domain to the soil domain, which is described below, and also to create adequate nodal contacts between the two contact surfaces.

A single variant analysis was conducted to determine the sensitivity of the numerical simulation to the initial shear modulus \( G \) of the boulder, which demonstrates that the boulder displacement was relatively insensitive to the change of boulder stiffness. For example, Figure 2-6 shows the effect of changing the boulder’s shear modulus by \( \pm 25\% \) on boulder displacement time history while keeping other parameters constant. Additional sensitivity analyses also confirmed that other global responses of the system (e.g., boulder rotation and truck penetration) are insensitive to the boulder’s shear modulus. This is likely due to several reasons. First, the boulder’s response was largely elastic during the field tests as no fracture of the boulder was observed after the tests. Second, the boulder rotation and displacement were likely governed by the surrounding soil, which has considerably lower stiffness and strength than the boulder. Consequently, the boulder essentially behaved as a rigid mass transferring the impact momentum and energy to the surrounding soil. The selected boulder properties were summarized in Table 2-1.
Figure 2-6. Effect of varying boulder elastic shear modulus on Test 1 center of gravity (C.G.) displacement.

Table 2-1. Summary of boulder and soil parameters used in M-C model.

<table>
<thead>
<tr>
<th></th>
<th>Density (tonne/mm$^3$)</th>
<th>Elastic Shear Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
<th>Friction Angle (degrees)</th>
<th>Cohesion (MPa)</th>
<th>Dilation Angle (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boulder</td>
<td>3.056x10$^{-9}$</td>
<td>9480</td>
<td>0.25</td>
<td>37.8</td>
<td>18.3</td>
<td>0</td>
</tr>
<tr>
<td>Soil</td>
<td>2.1x10$^{-9}$</td>
<td>20</td>
<td>0.25</td>
<td>45</td>
<td>0.0048</td>
<td>15</td>
</tr>
</tbody>
</table>
2.5.2 Soil Model

The soil domain, representing the AASHTO uniformly-graded coarse aggregate, was constructed using eight-noded constant stress solid elements. The extent of the soil domain needs to accurately predict the global response of the impact event. Since the impact event occurred over a short duration, the extent of the modeled soil domain does not need to be expansive as long as the reflected wave does not influence the simulation. Based on these considerations, the soil domain extends one embedment depth from the impact face, sides and beneath the boulder and three embedment depths behind the boulder as shown in Figure 2-4. This domain was computationally efficient and accurately predicted the global response. Boundaries of the modeled soil domain were constrained based on anticipated motion of each side of the exterior soil domain. For example, the sides of the soil domain were restrained against translation normal to their exterior surfaces. These imposed boundary conditions adequately constrained the soil domain when attempting to predict global response of the system because of the impact event’s short duration.

In conjunction with model calibration, a sensitivity study was conducted to determine an optimal mesh density for the soil with respect to model accuracy and computational efficiency. The mesh transitioned from the highest mesh density (i.e., finest) near the boulder to the lowest density (i.e., coarsest) at the exterior of the soil domain as seen in Figure 2-4. This transition was based on anticipated distribution of strain intensity within the soil domain. To realistically model the soil as a continuum, each soil element should represent a sufficient number of soil particles, and, for the soil used for
these tests (AASHTO uniformly-graded coarse aggregate), the smallest realistic soil element size was determined to be 50 mm in each local Cartesian direction based on the average particle size of the AASHTO uniformly-graded coarse aggregate. A mesh sensitivity study was conducted to investigate the effect of the smallest element size of 50 mm, 75 mm, and 100 mm on the numerical simulation to determine the highest possible mesh density near the boulder, where large displacements were anticipated, to increase computational efficiency while maintaining an accurate global response of the system. Results of this parametric study indicate that the smallest element size of 75 mm provided optimum combination of accuracy and computational efficiency. The largest mesh density on the exterior of the soil domain remained constant at 250 mm. Hence, the optimum mesh was determined to be 75 mm near the boulder-soil interface and smoothly transitioning to 250 mm at the exterior of the soil domain.

Calibration of M-C constitutive parameters for the backfill soil was conducted based on the results of Test 1. These parameters included shear modulus \( (G) \), friction angle \( (\phi) \), dilation angle \( (\psi) \), and cohesion \( (c) \). Based on the gradation and angularity of the AASHTO coarse aggregate the friction angle is estimated to be 45° and the dilation angle is estimated to be 15° (Bolton 1986). A sensitivity analysis indicated that shear modulus and cohesion had a greater influence on the global response of the system than the friction and dilation angles. Figure 2-7 shows a comparison of simulated boulder displacement versus time based on varied values of \( G \) and \( c \) with recordings from Test 1. Figure 2-7 shows that the boulder translational displacement increases as the shear modulus of soil decreases. A combination of \( G = 20 \text{ MPa} \) and \( c = 4.8 \text{ kPa} \) yielded an excellent
match with field-scale recordings. Hence, this combination was used for subsequent simulations. Table 2-1 summarizes material constitutive properties used in this study.

![Graph showing effect of varying elastic shear modulus and cohesion on Test 1 C.G. displacement.](image)

**Figure 2-7.** Effect of varying elastic shear modulus and cohesion on Test 1 C.G. displacement.

### 2.5.3 Contact Algorithms

Contact algorithms were used to mimic frictional interaction between the boulder and soil, truck and LVAR system, and self-contact of truck components. One- or two-way contact sliding interfaces, which prevented nodes of selected elements from penetrating the surfaces of others, were used to represent boulder-to-soil and vehicle-to-LVAR barrier contact. The one-way formulation checked to determine if the nodes of a defined slave part penetrated the surfaces of a defined master part while two-way formulation checked
penetrations in both directions (LSTC 2006). The static contact friction angle between the boulder and soil was selected to be 22.5° (i.e. \(0.5 \times \phi\)) (Das 2011) which gives a friction coefficient of 0.414. The dynamic coefficient of friction is assumed to be the same as the static coefficient. The static coefficient of friction between the truck and LVAR system was 0.4 (O’Hare et al. 2012). A standard penalty formulation that places interface springs normal to all penetrating nodes and contact surfaces was chosen to represent interaction between the truck and boulder because the stiffness of each was approximately the same order of magnitude. A soft constraint penalty formulation, which calculates the stiffness of the interface springs based on stability considerations during a given time step, was chosen for contact between the boulder and soil due to large differences in their stiffness magnitudes (Hallquist 2013). These penalty-based formulations were useful because no special treatment of intersecting interfaces is required. In addition, this approach minimizes hourglassing that could potentially occur in elements representing the soil due to large deformations that are anticipated in the soil during an impact event (LSTC 2006).

2.6 Results and Discussion

Two field-scale tests were conducted and used to rate the performance of LVAR barrier systems against M30 and M50 impacts. Using the data collected from the field and calibrated soil and boulder material models the results are discussed in the following section. The calibrated material models and soil domain from Test 1 were then applied to Test 2 to determine the accuracy and efficiency of the numerical simulations in predicting boulder displacement and rotation of the field-scale test. To model the second test, the
simulation began with the final position of the boulder and soil for Test 1 and all residual stresses and strains at the end of simulation of Test 1 were carried over to the Test 2 simulation. The final position of the truck after impact from Test 1 and Test 2 and snapshots of their respective LS-DYNA simulations are shown in Figure 2-8. From this qualitative comparison, the only observed differences were that the truck hood detached in both field-scale tests and the barrels did not eject in Test 1 when compared against the simulation. The amount of truck deformation was very similar between the field-scale test and respective simulations. Quantitative investigations that follow revealed that these observed differences played a minor role in relation to overall performance and the LS-DYNA model was determined to have accurately predicted actual impact behavior.
Figure 2-8. Comparison of final impact condition of field-scale tests and LS-DYNA simulations; (a) Field-Scale Test 1; (b) Simulation Test 1; (c) Field-Scale Test 2; and (d) Simulation Test 2.

Figure 2-9 presents a comparison of simulated boulder translational displacement versus time with test recordings based on fiducial tracking for Tests 1 and 2. Figure 2-10 shows the corresponding plot for boulder rotation. Both plots show good agreement between the test and simulation, although there is a slight disparity for boulder rotation in Test 2, observed in Figure 2-10. This disparity was attributed to soil particles in the numerical simulation unable to fill the void created behind the bolder after the impact when the boulder settles reaches equilibrium during a simulation. In the field-scale test, loose soil particles were able to fill the void created behind the boulder after impact when the boulder was rebounding. Nevertheless, Figure 2-9 and Figure 2-10 demonstrate that LS-
DYNA adequately captured the overall dynamic response of this LVAR system under vehicular impact. The calibrated soil properties also predicted the response of Test 2, which was rated for M50 as discussed previously.

Figure 2-9. Displacement of tracking fiducial on side of boulder for Test 1 and 2.
Figure 2-10. Rotation of tracking fiducial on side of boulder for Test 1 and 2.

Figure 2-11 shows the comparison of the simulated pressure-time histories at the location of the pressure cells with those recorded by the pressure cells for Test 1. Figure 2-12 shows the corresponding plot for Test 2.
Figure 2-11. Pressure-time history comparison for Test 1 of field-scale test and LS-DYNA simulation.
Figure 2-12. Pressure-time history comparison for Test 2 of field-scale test and LS-DYNA simulation.

These figures show that the LS-DYNA model can capture the magnitude of the pressure experienced by the pressure cells during impact testing. In both figures, the simulated pressure-time history starts out at some value while the recorded pressure-time history starts near zero. This is attributed to the type of pressure cell used in the field-scale tests. The pressure cell only measures the change in pressure and has a zero reading at constant pressure, while the LS-DYNA simulations measure the pressure with respect to depth. The disparity between the simulated and recorded pressure-time histories might be due to a combination of several reasons. First, the effect of soil-pressure cell interaction in the field test was not considered in the LS-DYNA simulations since the pressure cells were not modeled. The simulated pressure-time histories in LS-DYNA were obtained by monitoring the pressure-time histories of the elements at the corresponding locations of the
pressure cells. Second, the soil parameters were calibrated against the global response of Test 1, for which the response of the truck and boulder may have played a dominate role. Therefore, the calibrated soil parameters may not be able to accurately reflect the pressure-time history within the soil; however, Figure 2-11 and Figure 2-12 indicate that the magnitudes of soil pressures were captured.

2.7 Conclusions

There is a lack of LVAR field-scale tests and corresponding numerical modeling to investigate the performance of these systems under vehicular impact. This study presents two field-scale tests with corresponding LS-DYNA models to predict the global response of the system under vehicular impact. The LVAR system consisted of a single granite boulder embedded in AASHTO coarse aggregate subjected to M30 and M50 impact tests. Using high-speed cameras and pressure cells, information collected was used to calibrate both soil and boulder material parameters. This study suggests that: 1) a realistic truck model is imperative to accurately predict the global response of an LVAR system; and 2) using a simplistic material model for boulder and soil, LS-DYNA simulations were able to predict the global response such as boulder translational displacement and rotation under the impact events. Lastly, this study was able to calibrate soil and boulder material properties from Test 1 and accurately predict the response of a vehicular impact at a higher speed on a single boulder embedded in compacted AASHTO coarse aggregate fill.


Chapter 3

Field-Scale Testing and Numerical Investigation of Soil-Boulder Interaction under Vehicular Impact using FEM and Couple FEM-SPH Formulations

3.1 Abstract

A computational approach that couples the Finite Element Method (FEM) and the Smoothed Particle Hydrodynamics (SPH) method may be advantageous for simulating the response of complex, physical systems involving large deformations. However, comparisons of this modeling technique against field-scale test data are remarkably sparse in literature. This study presents three field-scale tests involving vehicular impact into three landscape vehicular anti-ram (LVAR) barriers. Each barrier consisted of a single boulder embedded in compacted AASHTO soil and physical testing resulted in one of the following outcomes: minimal boulder/soil movement (Test 1); moderate boulder/soil movement (Test 2); and severe boulder/soil movement and vehicle override (Test 3). For each test, two LS-DYNA models were developed: a model using a traditional FEM approach for the entire soil region along with a model using a hybrid FEM-SPH approach where the near-field soil region was simulated using SPH. For Tests 1 and 2 both the traditional FEM approach and the hybrid FEM-SPH approach were able to accurately match data collected from the field tests. However, for Test 3, the FEM-only approach was not able to accurately predict the global response of the system under vehicular impact. On the other hand, the hybrid FEM-SPH approach was able to capture global response of the system including boulder rotation, soil upheaval, and vehicle override.
3.2 Introduction

Large deformation in geomaterials has been modeled using advanced elasto-plastic constitutive models and numerical techniques such as the finite element method (FEM) (Finn et al. 1986) and finite difference method (FDM) (Bathurst and Simac 1994). Numerical models have the ability to capture the initiation and subsequent progressive deformation of geomaterials; however, the ability of these models to capture post-failure large deformation remains to be a critical issue. Since FEM and FDM are grid-based numerical methods, they generally have difficulty modeling large deformation of geomaterials. In FEM, each node of the computational mesh follows the assigned material during its motion. External surfaces and contact interfaces are easily tracked. Due to the fact that contact algorithms for the standard FEM are well defined, this method has been the predominate one used for modeling geomaterials. The method fails when excessive distortions in elements cause spurious behaviors. Adaptive remeshing (Khoei and Lewis 1999) and the Arbitrary Lagrangian-Eulerian (ALE) method (Hughes et al. 1981) have been used to model large deformations in FEM. These remeshing techniques, however, are problematic when addressing highly distorted meshes, particularly when the material behavior is highly nonlinear (Belytschko et al. 2000). Particle based, mesh-free methods were subsequently developed to handle these issues as a means of tracking materials using a set of interacting particles. Among these methods, the Smoothed Particle Hydrodynamics (SPH) method (Lucy 1977; Gingold and Monaghan 1977) has been shown to be a relatively mature and reliable method for accurately predicting large deformations of geomaterials.
SPH, a mesh-free method, is capable of handling large deformation without severe element distortion problems and has been used to model geomaterials post-failure (Chen and Qiu 2012 and 2014; Bui et al. 2008). Several researchers have successfully employed SPH to study the dynamic response of structures under high-velocity impact loads (Liu and Liu 2003; Schwer 2009; Jackson and Fuchs 2008; Aktay et al. 2005; Jankowiak and Lodygowski 2013; Swaddiwudhipong et al. 2010). This method is, however, usually less computationally efficient when compared to FEM and suffers from certain instability problems (Mohotti et al. 2015). Combining both approaches, using SPH to model the regions where large deformation is expected and using FEM elsewhere seems to be a logical application to address high-velocity impact with large soil deformations. Xu and Wang (2014) introduced different interactions methods available within LS-DYNA (Hallquist 2013; LSTC 2006) for SPH formulations and numerous researchers have incorporated a combined model of SPH and Lagrangian formulations (Jankowiak and Lodygowski 2013; Swaddiwudhipong et al. 2010; Thiayahuddin et al. 2012).

Landscape Vehicular Anti-Ram (LVAR) barriers play a crucial role in the protection of critical assets against terroristic threats. These barriers are designed to stop vehicular threats based on ASTM F2656: Standard Test Method for Vehicle Crash Testing of Perimeter Barriers (2007) with a design criterion of P1 rating. P1 criterion indicates that the front edge of the truck bed cannot pass beyond 1 meter behind the inside edge of the barrier during an impact event. Because these barriers are often comprised of a single boulder embedded in compacted soil, the optimization of the boulder is an important aspect to keep the barrier economically feasible and installation procedures relatively simple. All of these barriers are designed and optimized using LS-DYNA before any full-scale testing.
commences so it is imperative that the boulder and soil are modeled as accurately as possible to predict the translation and rotation of the boulder and the displacement of the soil during a vehicular impact. Inaccurately modeling the soil can lead to a false confidence in the barrier performance and result in failure to obtain a P1 rating of the barrier.

Several full-scale tests of single barriers embedded in soil with comparison to FEM have been published but lack to incorporate advanced modeling techniques like the addition of coupled FEM-SPH simulations. Asadollahi Pajouh et al. (2014) conducted full-scale tests and corresponding LS-DYNA simulations on a group of piles embedded in loose sand with no compaction. The full-scale test resulted in large soil deformations and the simulation over predicted the pile deformation using traditional FEM analysis (Asadollahi Pajouh et al. 2014). Ren and Vesenjak (2005) investigated a crash analysis of road safety barriers using LS-DYNA and compared to a full-scale test. The LS-DYNA numerical model consisted of traditional FEM and used springs to represent the stiffness of the soil. From the analysis of the full-scale test, the posts embedded in soil showed large deformation as well as soil upheaval. However, analyzing the soil as spring elements does not allow the user to see the soil deformation post impact (Ren and Vesenjak 2005). Wu and Thomson (2007) conducted full-scale tests and numerical simulations on a guardrail post and studied the interaction between the post and soil during quasi-static and dynamic loading. The guardrail was embedded in gravel and the authors used two different soil material models to represent the granular material. Simulations were conducted on quasi-static and dynamic loading conditions and both show severe element distortion in the soil elements near the guardrail. The authors also failed to show the comparison between full-
scale tests and numerical analyses to assess the accuracy of the numerical models (Wu and Thomson 2007).

Published comparisons between traditional FEM and coupled FEM-SPH simulations for soil-structure interaction involving large soil deformation are remarkably sparse in literature, particularly when this comparison is validated using instrumented, field-scale tests (Reese et al. 2012 and 2014; Zhou et al. 2016). Keske et al. (2015) investigated a low-order modeling technique of vehicle impacts upon boulders embedded in cohesionless soil. The authors were able to predict the response of a large boulder with little soil movement using their low-order model compared to data collected from an instrumented field-scale test. However, a barrier system that exhibited large boulder translation and rotation as well as large soil deformation could not be accurately predicted using their low-order model (Keske et al. 2015). The need to use higher order modeling techniques is required.

Research conducted at the Larson Transportation Institute, affiliated with The Pennsylvania State University, on the design and performance of three Landscape Vehicular Anti-Ram (LVAR) systems all consisting of a single boulder embedded in compacted fill will be summarized herein. Three field-scale tests were conducted and used to rate the performance of the barrier systems against M30 [vehicular speed of 48.3 km/hr (30mph)] impacts. Using high-speed cameras and field surveys, boulder movement was obtained from each test and was used to compare with model simulations. Each field-scale test was simulated using two LS-DYNA models. For the first model, the entire soil domain was modeled using traditional FEM formulation. For the second model, a coupled FEM-SPH formulation was used to model the soil domain, where SPH was used for the region
near impact (large deformation was expected) and FEM was used to model regions beyond the impact zone. The optimal size of the SPH zone for accurate prediction of boulder translational and rotational movement (i.e. global response) was determined. A sensitivity study of SPH particle spacing was also conducted. In the following sections, the field-scale tests are first described followed by descriptions of the LS-DYNA models and selection of model parameters and formulations. Lastly, results and discussions on the capability of the LS-DYNA models for capturing global response of the LVAR systems are presented. This paper focuses on the numerical predictions, including the use of FEM and coupled FEM-SPH formulations, and measured performance of LVAR systems in the field-scale tests. These results, therefore, provide a useful basis for evaluating predictive capabilities and limitations of the FEM and coupled FEM-SPH formulations in LS-DYNA when applied to LVAR barrier design.

3.3 Field-Scale Testing

Vehicular impact tests were completed according to ASTM F2656-07: Standard Test Method for Vehicle Crash Testing of Perimeter Barriers (2007), which establishes a penetration rating (desired P1) for perimeter barriers subjected to a vehicle impact (M30 designation). The test facility uses a rigid rail to provide vehicle guidance, a reverse towing system to accelerate the test vehicle to the required speed, and a release mechanism that disconnected the tow cable and steering guidance prior to impact. For a detailed description of the system, please refer to Reese et al. (2012 and 2014).
3.3.1 LVAR Barriers

Three different full-scale field tests were conducted for this research. Test 1 consisted of a Rockville White (RW) granite boulder approximately 1.98 m wide (W) × 1.68 m long (L) × 3.43 m high (H). The total embedment depth (D) was 2.03 m. The approximate weight of the boulder was 27,200 kg. Based on the information provided by the quarry that supplied the boulder, its bulk density and compressive strength were approximately 2696 kg/m$^3$ and 142 MPa, respectively (Coldspring Quarry 2014). Test 2 consisted of an American Black (AB) granite boulder approximately 0.98 m (W) × 1.05 m (L) × 3.0 m (H). The total embedment depth (D) was 1.95 m. The approximate weight was 9770 kg. The bulk density and compressive strength of the boulder were approximately 3165 kg/m$^3$ and 300 MPa, respectively (Rock of Ages Quarry 2014). Test 3 consisted of a different AB granite boulder approximately 1.19 m (W) × 0.78 m (L) × 2.44 m (H). The total embedment depth (D) was 1.38 m. The approximate weight was 5580 kg. Figure 3-1 summarizes the general dimensions for each of the tests described above. No natural fissures or joints were observed on the outer faces of the boulders. Along with material information provided from each quarry, small-scale testing, including uniaxial compression, split tension, Chevron bend, split Hopkinson pressure bar tests were completed to confirm supplied material property values (Reese et al. 2014; Xia et al. 2011; Gere and Timoshenko 1984). Table 3-1 summarizes the material properties of each of the granites used in the tests. Based on the tests conducted an initial shear modulus of 9480 MPa was determined, which is consistent with typical properties of granite published in literature (Gere and Timoshenko 1984; Wyllie and Mah 2004). Reese et al. (2014)
conducted a single variant analysis to determine the sensitivity of the numerical simulation to the initial shear modulus ($G$) of the Rockville White granite boulder, which demonstrated that the boulder displacement was relatively insensitive to the change of boulder stiffness.

<table>
<thead>
<tr>
<th>Granite Type</th>
<th>Compressive Strength (MPa)</th>
<th>Tensile Strength (MPa)</th>
<th>Fracture Toughness (MN/m$^{1.5}$)</th>
<th>Split Hopkinson Pressure Bar (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rockville White</td>
<td>45.2 ± 17.8</td>
<td>7.4 ± 0.8</td>
<td>1.5 ± 0.2</td>
<td>133.1</td>
</tr>
<tr>
<td>American Black</td>
<td>168.2 ± 38.6</td>
<td>13.8 ± 1.0</td>
<td>2.5 ± 0.3</td>
<td>180.0</td>
</tr>
</tbody>
</table>

Table 3-1. Summary of small-scale material testing on boulders
Figure 3-1. General layout of boulder embedment: (a) plan view; and (b) elevation view (Dimension of excavated pit to scale, Dimension of boulder not to scale).

According to ASTM F2656-07 standards, during the installation of each barrier, excavation should extend along the horizontal direction behind the boulder to a distance equal to 1.5 times the boulder embedment depth or 0.6 m, whichever is greater up to a maximum of 1.8 m (ASTM 2007). As shown in Figure 3-1, this criterion is satisfied. Prior to each test, the extent of excavation was surveyed. Prior to boulder placement, an American Association of State Highway and Transportation Officials (AASHTO) uniformly-graded coarse aggregate soil was placed into the excavated pit to the desired elevation in lifts of approximately 0.2 m and compacted using a tamping compactor to a density of not less than 90% maximum dry density in accordance with Test Method D6938-15 (ASTM 2015). The aggregate used was 2B gravel with a density of approximately 2100 kg/m$^3$, a porosity of approximately 0.35, an elastic shear modulus of approximately 20 MPa, and a gradation in general accordance with AASHTO M147-65 specifications. The boulder was then lowered into the pit and centered for impact. Similar procedures were followed to backfill the pit with compacted aggregates.
3.3.2 Test Results

High-speed cameras were implemented during testing to record pertinent information, such as boulder translation, displacement and rotation and global response of the system, which included truck behavior. Figure 3-2 shows where the high-speed cameras were located to record the motions of the truck and boulder immediately prior to and after the impact. Camera 1 was positioned at a 90° angle to the center of the test article to measure dynamic penetration. Camera 2 was positioned at a 90° angle above the center of the test article to capture enough surface area prior to and after impact to determine impact speed, impact angle, exit angle and debris field. Camera 3 was positioned behind the test article centered along the guidance rail to record the approach of the test vehicle to track its alignment with the center of the test article during impact. The distance from each camera to the center of the test article is summarized in Table 3-2. Each distance was used to calculate critical information from field-scale tests, including impact speed, angle of approach, truck dynamic penetration and truck offset at impact.

Table 3-2. Distance from center of test article to each camera

<table>
<thead>
<tr>
<th>Camera 1 (m)</th>
<th>Camera 2 (m)</th>
<th>Camera 3 (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 1</td>
<td>20.55</td>
<td>15.94</td>
</tr>
<tr>
<td>Test 2</td>
<td>25.04</td>
<td>6.76</td>
</tr>
<tr>
<td>Test 3</td>
<td>20.57</td>
<td>15.77</td>
</tr>
</tbody>
</table>
Test 1 resulted in minimal movement (both translation and rotation) of the RW granite boulder. The front end of the truck rebounded after impact. Based on analysis of the high-speed video, the approach speed of the truck at impact was 52.3 km/h. Analysis of data from Camera 3 determined the centerline of the truck impacted the test article 14 cm to the left (to the side of Camera 1) of the critical impact point that was defined as the centerline of the boulder. Test 2 resulted in moderate boulder translation and rotation as well as soil upheaval behind the boulder during impact. From high-speed video analysis, the approach speed of the truck was 48.3 km/h and the center of the truck impacted 6.57 cm to the right of the center of the AB boulder. During Test 3 vehicle impact, the AB granite boulder translated and rotated out of the ground upon impact. The truck ramped over the flipped boulder and landed with the back axle sitting on top of the boulder. During this test, a malfunction of the high-speed video trigger resulted in recording post-test data and not the actual impact. Based on the stationary radar system [Stalker Speed Sensor (S3)] the approach speed at impact was 54.7 km/h. This system determines the test vehicle speed during the towing process using a stationary Doppler radar speed sensor operating at a frequency of 34.7 GHz and communicating through a RS-232 port (Stalker Traffic
Technologies 2014). Inspection after each impact showed that none of the boulders showed any signs of fracture or cracking. Figure 3-3 shows the final position of the boulder and truck for each completed test. In particular, Figure 3-3c shows the truck override and boulder displacement due to the vehicle impact.
The LS-DYNA research/commercial code (Hallquist 2013 and LSTC 2006) was utilized to perform simulations to model the vehicular impact tests. In this section, the numerical model is first discussed, including a detailed description of SPH formulation as well as material properties, followed by a description of contact algorithms used within LS-DYNA to connect SPH particles to FEM segments.
3.4.1 Numerical Model

Two numerical models of each field-scale test were created to compare their performance at capturing global response of the LVAR system when varying magnitudes of soil deformation occurred. The first model consisted solely of finite elements for the LVAR barrier and soil domain as shown in Figure 3-4a; whereas the second model used a hybrid approach for modeling soil. In the near-field soil region, SPH formulations were used, while solid finite elements were used in the far-field, as seen in Figure 3-4b, to take advantage of SPH’s capabilities in modeling large deformations and FEM’s computational efficiency. Each model consisted of two parts: (1) medium-duty truck, and (2) LVAR barrier consisting of a boulder embedded in soil.

![Figure 3-4. LS-DYNA models: (a) FEM-only model; (b) hybrid FEM-SPH model.](image)

The truck model used for the simulations was modified from a model readily-available in the National Crash Analysis Center (NCAC) database (NCAC 2013; Mohan
The NCAC truck model was developed to ensure that the load transfer between the truck and hardware, the deformation of the truck, and the overall behavior of the truck during impact simulations would be as accurate as feasible given model computational requirements. Based on requirements from ASTM F2656-07, the modified truck model consisted of 1,606 eight-node constant-stress solid elements, 20,333 four-node Belytschko-Tsay shell elements, and 377 Hughes-Liu beam elements with cross section integration. The truck model was validated as reported in Reese et al. (2014) using checks of equilibrium, conservation of energy principles, and the amount of energy absorption that occurred through plastic deformation of truck components when hitting a rigid wall.

Each LVAR device was comprised of a single boulder embedded in compacted AASHTO soil as shown in Figure 3-4. Eight-node constant stress cubic solid elements were used for the boulder in both of the models. The solid element size was 100 mm based on Reese et al. (2014). The soil domain, representing an AASHTO uniformly-graded coarse aggregate, was modeled using two different approaches. In the FEM-only model, eight-node constant stress solid elements were used and varied in size from 75 mm near the boulder to 250 mm at the exterior of the soil domain. A mesh convergence study was conducted in Reese et al. (2014) for the FEM-only model by changing the mesh size nearest the boulder and transitioning to the exterior the soil domain. The mesh distribution described previously resulted in convergence of the mesh (Reese et al. 2014). The hybrid approach used a combination of SPH particles near the boulder connected to eight-node constant stress solid elements in the far-field as seen in Figure 3-4b. Based on SPH model parameters discussed in the subsequent sections, the size of the FEM elements (for the
hybrid approach) were constant at 150 mm. Detailed discussion of SPH formulation and model parameters of the hybrid approach are presented in subsequent sections. The extent of the soil domain, based on embedment depth of the boulder, remained the same between models and can be seen in Figure 3-1 (Reese et al. 2014).

The size of the soil domain was selected to negate any effects from reflected compression waves that would interfere with the impact response. Normal translation of the exterior and bottom boundaries of the soil domain were the only constrained degrees of freedom. An investigation of the size of the soil domain, to negate any boundary effects, was conducted by changing the size of the soil domain (based on the embedment depth) and compares the barrier response for Test 1. Four domain sizes were investigated: 2D – ½ embedment depth in the front and sides of the boulder and two times the embedment depth behind the boulder; 3D – one embedment depth in the front and sides and three times the embedment depth behind the boulder; 4D – 3/2 embedment depth in the front and sides and four times the embedment depth behind the boulder; and 5D – two times the embedment depth on the sides and front of boulder and five times behind the boulder. The translational displacement of the top corner (furthest point from impact) of the boulder was monitored for convergence of the soil domain. Figure 3-5 shows displacement of the boulder while varying the soil domain. There was a 5.3 percent change when expanding the soil domain from 2D to 3D, a 6.4 percent change when expanding from 3D to 4D, and a 1.6 percent change when expanding from 4D to 5D. Therefore, based on this analysis, a soil domain of 3/2 times the embedment depth around the front and sides of the boulder and four times the embedment depth behind the boulder is used for all subsequent analyses.
3.4.2 SPH Formulations for Soil

SPH is a mesh-free, Lagrangian, particle-based method developed by Lucy (1977) and Gingold and Monaghan (1977). In SPH simulations, the computational domain is discretized into a finite number of particles, each representing a certain volume and mass of the material (fluid or solid) and carrying field variables such as velocity, acceleration, density, and pressure/stress. Therefore, the SPH method is a continuum-scale numerical method. The material properties, $\Pi^h f(x)$, at any point $x$ in the simulation domain are then calculated according to an interpolation process over its neighboring particles that are within its influence domain $\Omega$ through:

$$\Pi^h f(x) = \int_{\Omega} f(y)W(x - y, h)dy$$  \hspace{1cm} (3.1)
where $W$ is the smoothing kernel function, which is a weighting function and $h$ is the smoothing length that is a unit measure of the sub-domain of influence of function $W$. Figure 3-6 illustrates the underlying principle behind the interpolation process.

![Figure 3-6. Particle approximation based on kernel function $W$ in influence domain $\Omega$ with radius $\kappa h$.]

The kernel function $W$ is defined using the function $\theta$ by the relation

$$W(x, h) = \frac{1}{h(x)^d} \theta(x) \quad (3.2)$$

where $d$ is the number of space dimensions. It should be noted that $W(x, h)$ should be a centrally peaked function. The most common smoothing kernel is the cubic B-spline which is defined as:
where \( C \) is a constant of normalization that depends on the number of space dimensions.

LS-DYNA computes the initial smoothing length, \( h_0 \), for each SPH part by taking the maximum of the minimum distance between every particle. Every particle has its own smoothing length which varies in time according to the following equation:

\[
\frac{d}{dt}[h(t)] = h(t) \nabla \cdot \nu
\]  

(3.4)

where \( h(t) \) is the smoothing length and \( \nabla \cdot \nu \) is the divergence of flow. The smoothing length increases when particles separate from each other and reduces when the concentration of particles increases. It varies to keep the same number of particles in the neighborhood. The smoothing length varies between the minimum and maximum values:

\[
H_{MIN} \times h_0 < h(t) < H_{MAX} \times h_0
\]  

(3.5)

where \( H_{MIN} \) and \( H_{MAX} \) are scale factors for the smoothing length. The smoothing length has significant impacts on the overall numerical behavior (e.g., accuracy and efficiency). SPH particles interact with each other only if they are within each other’s influence domain, otherwise, they are independent from each other as shown in Figure 3-6. Therefore, larger smoothing length (i.e., larger influence domain) generally results in smoother or more continuous behavior as the SPH particles are more interdependent; whereas smaller smoothing length generally yields more discrete behaviors as the SPH particles are more independent from each other. Since the smoothing length is a function
of the individual particle and time, the constant applied (i.e. $\kappa$ in Figure 3-6) to the smoothing length is an important parameter. Sakakibara et al. (2008) recommends using a smoothing constant of 1.05 instead of the recommended value of 1.2 from LS-DYNA. This research compared FEM and SPH simulation of the same model and varied several different SPH parameters, including particle spacing, smoothing length constant, and effect of renormalization in an attempt to determine the most efficient and accurate method when using SPH particles. These studies will be discussed later in this paper.

### 3.4.3 Material Properties

The LS-DYNA Material Type 173, “Mohr-Coulomb (M-C)” (Hallquist 2013) was utilized to model the boulder and soil behavior in all the simulations. The M-C model was used to represent the soil and boulder due to its ability to effectively and simplistically capture impact conditions present in the field testing (Reese et al. 2014). The Mohr-Coulomb model characterizes failure of a material based on its cohesion, friction angle, and normal and shear stresses at a point as follows (Hallquist 2013 and LSTC 2006):

$$
\tau_{\text{max}} = c + \sigma_n \tan(\phi)
$$

(3.6)

where $\tau_{\text{max}}$ is the shear strength on any plane, $\sigma_n$ is the normal stress on that plane, $c$ is cohesion and $\phi$ is the friction angle. Model parameters for the boulder and soil were calibrated and validated in Reese et al. (2014) and material properties are summarized in Table 3-3. Although different boulders were used in these three tests, the boulders behaved
essentially as a rigid mass and, hence, the same boulder properties were used for all numerical simulations. This approach is justified in Reese et al. (2014).

<table>
<thead>
<tr>
<th></th>
<th>Density (ton/mm$^3$)</th>
<th>Elastic Shear Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
<th>Friction Angle (degrees)</th>
<th>Cohesion (MPa)</th>
<th>Dilation Angle (degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Boulder</td>
<td>3.056×10$^{-9}$</td>
<td>9480</td>
<td>0.25</td>
<td>37.8</td>
<td>18.3</td>
<td>0</td>
</tr>
<tr>
<td>Soil</td>
<td>2.1×10$^{-9}$</td>
<td>20</td>
<td>0.25</td>
<td>45</td>
<td>0.0048</td>
<td>15</td>
</tr>
</tbody>
</table>

### 3.4.4 FEM-SPH Coupling

Both SPH and FEM formulations in LS-DYNA are based on the Lagrangian approach. Therefore, it is possible to link both methods at an interface. The interface ensures continuous coupling of the two methods. At the interface, the SPH particles are constrained and move with the elements. The influence domain of the particles at/near the interface zone, such as that of the particle $i$ (see Figure 3-6), covers both the FE mesh and SPH particles and certain considerations are required in the computation. For strain and strain rate calculation of each particle, only those from the SPH particles within the influence domain are considered, whereas the contributions from both SPH particles and elements inside the influence domain are included to calculate forces (Johnson 1994 and LSCT 2006).
LS-DYNA allows mesh-based and mesh-free techniques, such as SPH, to exist and interact in one simulation, thereby allowing users to take advantage of both procedures. The interaction or coupling between SPH particles and FEM elements can be defined using traditional tied- or penalty-based contact definitions (Beal et al. 2013). Since there is no mesh connectivity for the SPH particles, it is imperative that only “nodes_to_surface” contact definitions are utilized in which SPH is always defined to be the slave node \( n_s \) and finite elements are defined to be the master surface.

Tied-based contact consists of “tying” SPH slave nodes to FEM surfaces to connect the two domains. LS-DYNA ties translational degrees of freedom of nodes to a specified surface. The constraints are only imposed on the slave nodes, so the more coarsely meshed side of the interface should be the master surfaces (i.e. FEM) (LSTC 2006). Ideally, each master node should coincide with a slave node to ensure complete displacement compatibility along the interface, but this is difficult, if not impossible, to achieve.

Penalty-based contact consists of placing normal interface springs, stiffness factor equal to \( k_i \), between all penetrating nodes and the contact surface. Standard penalty formulation was utilized for this study. In standard penalty formulation, the interface stiffness is chosen to be approximately the same order of magnitude as the stiffness of the interface element normal to the interface particle. In applying the penalty method, each slave node is checked for penetration through the master surface. If the slave node does not penetrate, contact force is not applied. If it does penetrate, the contact force is calculated based on the amount of penetration and contact stiffness, which is then applied to the slave node and contact point (LSTC 2006).
The stiffness factor, $k_j$, is determined in several ways including: the minimum of the master segment and slave node stiffness, the master segment stiffness, the slave node stiffness or the area-/mass-weighted slave node value. Since the boundary between SPH particles and solid FEM segments is the same material with identical stiffness, a minimum of the master segment and slave node stiffness was used.

Penalty-based automatic “nodes-to-surface” contact was used for each interface with SPH particles including: boulder and FEM soil domain. The contact between the boulder and SPH soil domain used a static and dynamic coefficient of friction of 0.414 determined by the friction angle of the AASHTO soil. The static and dynamic coefficient of friction for the SPH and FEM soil domain was 1.0. Figure 3-7 displays the contact between the SPH nodes and the surface of the FE model. The SPH nodes are always the slave while the finite elements are always the master surface.
3.5 Model Parameters

Several parameters affect the overall response of the near-field soil region simulated using SPH particles, including particle spacing, type of formulation, smoothing length, and size of the region. Small-scale SPH simulations were conducted to determine the particle spacing. The formulation type and smoothing length were determined from literature and discussed below. These small-scale SPH simulations were then compared to a model comprised of Lagrangian finite elements to determine the accuracy of the SPH model. Lastly, Test 3 was used to determine the appropriate extent of the near-field SPH
soil region needed to accurately predict the behavior resulting from large soil deformations exhibited in the field-scale test.

3.5.1 SPH Particle Spacing

Particle spacing plays an integral role in accurately predicting the overall response of the system as well as enhancing model computational efficiency. An increase in particle spacing decreases computational time but decreases model accuracy. It is generally recommended for a model to have densely packed SPH particles with a constant initial distance between them in all directions (Bojanowski and Kulak 2010; Bojanowski 2014; Kulak and Schwer 2012). It is also recommended for stability of the calculations to have at least four SPH particles per face of the Lagrangian element in contact with those SPH particles.

A small-scale numerical study was conducted to investigate the optimal SPH spacing for contact between SPH and FEM domains. This small-scale study consisted of a confined compression test of soil as shown in Figure 3-8. The test was simulated using two formulations: a FEM-only model where the top loading ram, confinement cylinder, base and soil were modeled using traditional FEM formulations; and a coupled FEM-SPH model where the soil was modeled using SPH formulations. Soil properties were from Reese et al (2014). In the numerical simulations, the loading ram moved vertically and the displacement and reaction force from the soil were tracked. Element sizes of 18.4 and 25.4 mm were used for the FEM models and showed convergence as seen in Figure 3-9. Based on the larger element size, the SPH model was created using four SPH nodes per element
side (a four by four configuration) as seen in Figure 3-8b and five nodes per element side (a five by five configuration) as shown in Figure 3-8c. The simulated force versus displacement curves from the SPH are shown in Figure 3-9 for the four by four and five by five SPH arrangements. There is a good agreement and convergence between the FEM and the five by five SPH arrangement. The four by four arrangement did not converge with the FEM model, mostly likely due to the insufficient number of particles per FEM mesh. From this comparative analysis, a grid of five by five SPH particles was used where SPH particles were in contact with a FEM domain.
Figure 3-8. Plan and elevation view of small-scale confined compression test using: (a) Finite Elements: 25.4 mm; (b) SPH Particles: 4×4 configuration; and (c) SPH Particles: 5×5 configuration.
3.5.2 Formulation and Smoothing Length

There are several types of SPH formulations within LS-DYNA that determine particle approximation, including a default formulation and a renormalization approximation. Sakakibara et al. (2008) conducted analyses of the effects of changing formulation type while keeping the smoothing length constant. Based on the analysis using the default formulation, the particles at the boundary edge had lower stress due to a truncated boundary and reduced number of particles within each edge particle’s influence domain. To account for the issue of the FEM mesh truncating the influence domain of SPH particles in the vicinity of a FEM-SPH boundary, the renormalization technique readily available in LS-
DYNA proved to correct the stress inaccuracies. Therefore, renormalization approximation (FORM 1) was used for all SPH analyses in this study.

Smoothing length is another SPH parameter that directly affects the influence domain of the particles. The smoothing length constant can be varied between recommended values of 1.05 and 1.3 within LS-DYNA. Comparisons of four different SPH models with increasing particle density were analyzed to see the effects of varying the smoothing length constant. Three smoothing lengths (1.05, 1.2 (default), and 1.3) were examined (Sakakibara et al. 2008). A smoothing length constant of 1.05 was found to be the most accurate in all models regardless of the particle density. In all cases, a higher smoothing length caused a weaker response of the model and material was less stiff. Based on this analysis, a smoothing length constant of 1.05 was used in all subsequent analyses.

3.5.3 Near-Field SPH Soil Region

Based on the above parameters, the extent of the near-field SPH soil region was determined as a function of the embedment depth of the barrier based on observations seen for Test 3 and shown in Figure 3-3c. Using an entire soil domain consisting of SPH would not be computationally efficient and Reese et al. (2014) showed regions of little soil disturbance could be accurately predicted using traditional FEM formulations. Therefore, three different near-field SPH soil domains, based on Test 3 as described in a previous section, were constructed based on a proportion of the embedment depth of the barrier (¼, ½, and ¾) as shown in Figure 3-10a-c. The SPH regions were located around the boulder where large soil deformation was expected. Each model was compared to the final
boulder/truck position (Figure 3-3c) to determine the extent of the near-field SPH soil region that most accurately predicted the response. Figure 3-10d shows the final resting position of the truck and boulder for a SPH region extending ¼ of the embedment depth of the boulder. The computational time for this analysis was 3 hours 42 minutes. Figure 3-10e shows the corresponding plot for a SPH region equal to ½ of the embedment depth away from the boulder and its computational time was 5 hours 44 minutes. Figure 3-10f shows the corresponding plot for a SPH region equal to ¾ of the embedment depth away from the boulder and its computational time was 12 hours 8 minutes. Analyzing the distance the truck traveled past the boulder was used as a basis to determine which SPH soil domain size to use. By visual inspection the ¼ embedment depth model does not accurately predict the final position of the truck ramping over the boulder. The boulder sticks to the bottom of the truck and digs into the FEM soil domain causing the truck to stop short of ramping entirely over the boulder. In the ½ embedment depth model, the truck traveled 2210 mm farther than the ¼ embedment depth model and more accurately predicts the final position of the truck. The ⅔ embedment depth model shows the truck traveled approximately 540 mm farther than the ½ embedment model. Both the ½ and ¾ embedment depth models capture the overall behavior of Test 3 with little quantitative difference in results, but a dramatic difference in computational efficiency. The ¾ embedment depth model takes over two times the computational time for little difference in results. Therefore, the ½ embedment depth model is used to represent the near-field SPH soil region.
Figure 3-10. Extent of near-field SPH soil domain for Test 3: (a) $\frac{1}{4}$ Embedment Depth; (b) $\frac{1}{2}$ Embedment Depth; (c) $\frac{3}{4}$ Embedment Depth; Post Impact condition for: (d) $\frac{1}{4}$ Embedment Depth; (e) $\frac{1}{2}$ Embedment Depth; and (f) $\frac{3}{4}$ Embedment Depth

3.6 Results and Discussion

The final position of the truck after impact from Tests 1, 2, and 3 and snapshots of their respective LS-DYNA simulations using FEM-only and hybrid FEM-SPH formulations are shown in Figure 3-11. From this qualitative comparison, Test 1 showed a good agreement between the field-scale test and both of the LS-DYNA models. Test 2 showed a similarly good agreement between the field-scale test and the LS-DYNA models, but the hybrid model showed slight truck ramping that wasn’t present in the FEM model or the field-scale test. The largest qualitative difference between LS-DYNA models was
for Test 3, shown in Figure 3-11h and Figure 3-11i. The traditional FEM model did not predict the overturn of the boulder and truck override while the hybrid model did predict the override, results that could lead to false confidence in a barrier meeting P1 (a penetration distance of less than 1 m) rating for a M30 impact based on FEM model prediction.

Figure 3-11. Comparison of final impact condition of field-scale tests and LS-DYNA simulations; (a) Test 1; (b) FEM Test 1; (c) Hybrid Test 1; (d) Test 2; (e) FEM Test 2; (f) Hybrid Test 2; (g) Test 3; (h) FEM Test 3; and (i) Hybrid Test 3

Figure 3-12 presents comparison between simulated boulder translational displacement (FEM and hybrid) versus time against test data taken from fiducial tracking of the boulder for Test 1. Figure 3-13 shows the corresponding plot for boulder rotation. Both plots show a good agreement between the test and simulations. Figure 3-13 shows
the FEM model accurately predicts boulder rotation, however, the hybrid model over predicts the rotation in the late stage of impact. Nevertheless, Figure 3-12 and Figure 3-13 demonstrate that LS-DYNA adequately captured overall dynamic response of Test 1 with minimal boulder and soil movement using two different modeling techniques embedded within the program.

![Figure 3-12. Center of gravity displacement for Test 1: field data, FEM, and hybrid model](image-url)
Figure 3-13. Center of gravity rotation for Test 1: field data, FEM, and hybrid model

Figure 3-14 presents comparisons of boulder translation parallel to the impact direction for Test 2 tracking fiducials from the field-scale test. Test 2 had moderate boulder and soil movement but the comparison of the FEM and hybrid simulations to the field data shows a good agreement. Both of the simulations over-predicted boulder translation. This may be due to using soil material properties taken from Test 1 even though this installation may have produced slightly different compaction levels for the AASHTO soil. Figure 3-15 shows the corresponding plot for boulder rotation. The FEM simulation was able to capture boulder rotation during impact. The hybrid simulation was able to capture the boulder rotation up to a certain point and then predicts a larger final rotation of the system. Regardless of minor installation differences that could have influenced material properties
for the compacted AASHTO fill, both LS-DYNA simulations were able to adequately capture overall dynamic response under vehicular impact.

Figure 3-14. Center of gravity displacement for Test 2: field data, FEM, and hybrid model
For Test 3 the high-speed cameras were not triggered prior to impact and, hence, high-speed video analysis was not available for boulder translation and rotation data. Instead, a qualitative analysis using images captured from a digital camera is presented as well as a comparison of the truck velocity from a low-resolution camera was used to compare Test 3 to FEM and hybrid numerical analyses. Figure 3-16 shows the qualitative comparison of the still images captured during Test 3 compared to the two LS-DYNA simulations. From Figure 3-16b it can be seen that the FEM simulation under-predicted boulder translation and rotation, as well as truck ramping. This is most likely due to FEM being a grid based formulation that has difficulty simulating large deformations. On the other hand, the hybrid model predicts global response of the system, including boulder
translation and rotation as well as truck ramping, relatively well as shown in Figure 3-16c. The hybrid simulation did exhibit some boulder “sticking” underneath the truck which caused the boulder to drag after truck ramping. This “sticking” did not change the overall performance of the hybrid simulation.

Figure 3-16. Comparison of digital camera images and LS-DYNA simulations at different simulation time steps: (a) Test 3; (b) FEM model; and (c) hybrid model

Vehicular data was processed from a low-resolution hand-held camera for Test 3. The truck velocity in the direction of impact was analyzed and compared to the numerical simulations. Figure 3-17 shows a screenshot from the video analysis program. Photron
FASTCAM Analysis software (2014) was used to analyze the video data based on a dimensional scale, stationary point, and tracking point shown in Figure 3-17. The length of the boulder was used as the dimensional scale for each frame to determine the movement of the tracking point from one frame to the next. A stationary point was needed because the hand-held camera had a lot of movement throughout the impact. The stationary point was used to calibrate the tracking point, which was a point on the side door of the truck, to remove any artificial movements of the camera during the filming process. The same point on the truck was used to compare the truck velocity in the FEM and hybrid analyses. Figure 3-18 shows velocity versus time for the field-scale test, FEM simulation, and hybrid simulation. From this figure, it is clear that the FEM simulation is inaccurate and actually predicted the truck to stop at 0.42 seconds which was not the case in the field-scale test. The comparison between the field-scale test and the hybrid simulation shows good correlation. There is some error but this could be due to the quality of the video. For example, the truck shows an increase in velocity at certain points during impact which is physically impossible. Even with the poor quality video, the hybrid simulation predicts the truck behavior during impact.
Figure 3-17. Video analysis of Test 3 showing length scale, stationary point, and tracking point

Figure 3-18. Truck velocity in direction of impact for Test 3: field data, FEM, and hybrid model
From the above figures, it can be seen that there are advantages and disadvantages for both FEM and hybrid FEM-SPH modeling approaches. The obvious advantage of using a FEM formulation is its proven computational efficiency. However, if a model takes advantage of strategically placing more complex modeling techniques (i.e. SPH) in areas that are suspected to produce large deformations, a model can be computationally optimized while producing more accurate results. In all three tests, the hybrid approach was able to accurately predict the global response of the system under a vehicular impact while the FEM-only model for Test 3 failed to predict the response due to the large deformation near the area of impact. Another advantage of using a hybrid modeling approach is the ability to optimize the barriers, including size, weight, and embedment depth of the barrier, without the fear that the model is inaccurate. By optimizing the barrier, the economic benefits are tremendous. The cost of the boulder will decrease as well as the need to have special equipment onsite during the installation phase. These cost savings may be small relative to a single barrier but when hundreds are being installed around the perimeter of a critical asset that small savings add up.

3.7 Conclusions

This paper presents three field-scale tests conducted and used to rate the performance of LVAR barrier systems against M30 impacts. Each barrier consisted of a single boulder embedded in compacted AASHTO aggregate. For each test, two LS-DYNA models were created to predict the global response of the system under vehicular impact. The first LS-DYNA model used traditional finite elements while the second model consisted of a hybrid
FEM-SPH approach. Test 1 resulted in minimal boulder and soil movement; Test 2 resulted in moderate boulder and soil movement; and Test 3 resulted in excessive boulder rotation, large soil deformations, and truck override. For Tests 1 and 2 both the traditional FEM approach and the hybrid FEM-SPH approach were able to accurately match data collected from the field tests. However, for Test 3 the traditional FEM approach was not able to accurately predict global response of the system under vehicular impact. The hybrid approach was able to capture global response of the system including boulder rotation, soil upheaval and truck override. This research suggests that a hybrid FEM-SPH approach is advantageous in simulating the field performance of embedded structures under impact loading involving large deformation of soil.

Several parameters were determined to help accurately predict global response of the system when implementing a hybrid FEM-SPH modeling approach. First, there needs to be at least five SPH particles per finite element length at all FEM-SPH boundaries. Second, the extent of near-field SPH soil domain needs to be proportional to ½ the embedment depth of the boulder for adequate computational efficiency and accuracy.
References


Photron FASTCAM Analysis (PFA) Ver.1.2.0. 2014.


Chapter 4

Development of Material Model for Expanded Polystyrene Crushable Concrete through Static and Dynamic Testing, CT Imaging and Numerical Simulations

4.1 Abstract

Tremendous energy is dissipated during a vehicular impact or blast load on a security barrier. Expanded polystyrene crushable concrete has been proven to dissipate large amounts of energy by cell crushing and increased impact response time. In this study, a crushable concrete mix design was tested where expanded polystyrene was used as the primary cell material. Quasi-static and dynamic drop weight tests were conducted on fully confined specimens to help determine material constitutive properties for numerical modeling. A LS-DYNA model, using Material Type 16, “Pseudo Tensor”, was developed to simulate both the quasi-static and dynamic drop weight tests. The numerical model was able to capture the overall responses of the specimens during the quasi-static testing and consecutive impacts from the dynamic drop weight tests. Several compressed specimens from the dynamic drop weight test, representing different strain levels, were studied using non-destructive X-ray CT imaging to ascertain damage levels. From the CT scans, the volume of concrete with respect to voids along the height of each specimen could be determined. As the level of dynamic compression increased, the volume of concrete increased and the voids decreased, showing that the polystyrene beads were being crushed. Also, the bottom of the specimen exhibited higher amounts of crushing of the polystyrene
beads. This was due to reflective waves from the bottom steel platen used to support and confine the concrete specimen.

4.2 Introduction

In recent years, terrorist attacks and security measures to stop or mitigate the damage they cause have been widely discussed and researched. One of the primary areas of research on this topic has been perimeter security systems. The goal of these perimeter security systems is to absorb large amounts of kinetic energy from vehicular impact (Krishna-Prasad 2006; Ferdous et al. 2011; O’Hare et al. 2012; and Uzzolino et al. 2012). Due to increased requirements to minimize barrier size, it has become more difficult to design a barrier capable of stopping an attack vehicle. One method for dissipating some of the vehicle’s high kinetic energy is to incorporate crushable materials in the barrier design. These materials will deform during impact, dissipating some of the vehicle’s energy and reducing load transmitted to other parts of the barrier. A crushable material that is capable of dissipating large amounts of energy could potentially reduce the size and cost of a barrier without any loss of performance with respect to stopping moving objects.

For this study, a crushable concrete mix design was tested where expanded polystyrene (EPS) was used as the primary cell material. Polystyrene is a synthetic polymer made from monomer styrene. EPS is a very good thermal insulator and has a weak crushing strength along with low density (Baxter and Jones 1972; Premier Industries 2005). Polystyrene is also very inexpensive to produce, making it an excellent cellular material in energy absorbing crushable concrete. Disadvantages related to using EPS in
Crushable concrete exist that must be considered, including: 1) polystyrene is an extremely lightweight, hydrophobic material that is subject to segregation between the cement mixture and beads; 2) its hydrophobic nature causes a reduction in interfacial bond strength between the polystyrene aggregate and the cement matrix; and 3) since the polystyrene aggregate has virtually no compressive strength, it offers little resistance to the cement matrix’s tendency to shrink (Chen and Liu 2004). Figure 4-1, taken from a CT scan, shows the typical micromechanical makeup of a polystyrene concrete. Black circles represent voids or cellular material (expanded polystyrene beads) and white represents the cement matrix.

**Figure 4-1. CT image of EPS concrete showing expanded polystyrene beads and cement matrix**

In general, polystyrene concrete that is loaded to high strains behaves similarly to other cellular foams (Santagata et al. 2010). First, the material exhibits a linear-elastic
phase that is similar to normal concrete. The initial elastic phase defines the elastic modulus, yield stress and yield strain (Gibson and Ashby 1982). As the crushable concrete reaches its relatively low plateau stress, it begins to enter its crushing phase, where it continues deforming at a relatively constant stress. The cells made of polystyrene begin to progressively collapse. During this progressive collapse, energy is dissipated from two mechanisms: brittle fracture of the cement matrix and plastic deformation of the polystyrene cells (Bassani et al. 2012). Finally, as the last of the cells finish collapsing, there is no more room for continued collapse. At this point, the crushable concrete has reached its densification phase. In this phase, the concrete behaves in a much stiffer manner and the load will increase rapidly with increasing strain. With respect to the three phases of response for compression in the foam rise direction, the initial linear elasticity is generated by cell wall bending and cell face stretching while the plateau phase is associated with the collapse of cells by elastic buckling, formation of plastic hinges, and crushing in the cell walls. Figure 4-2 shows an idealized stress-strain curve for a cellular concrete showing the linear-elastic, crushing and densification phases, where $\sigma_{pl}$ and $\varepsilon_D$ denote plateau stress and densification strain, respectively. Although deformation in the loading direction can be very severe, deformation transverse to this is almost negligible, i.e. Poisson’s ratio is approximately zero and volume is not conserved (Tu et al. 2001).
Figure 4-2. Idealized stress-strain curve for a cellular concrete (after Santagata et al. 2010)

A vehicular impact event is characterized by relatively high contact forces acting on a small contact area over a very short time duration. Most material characterization of crushable concrete has been performed under quasi-static conditions to determine material characteristics that do not incorporate strain rate effects (e.g., Baxter and Jones 1972; Gibson and Ashby 1997; Babu and Babu 2003; Chen and Liu 2004; Bouvard et al. 2007; Barsotti 2011; and Bassani et al. 2012). Rizov (2007) conducted low velocity impact tests on closed-cell cellular polyvinylchloride (PVC) foam with a mass of 6.4 kg at varying drop-heights to have a range of impact energies. All of the tests were conducted on unconfined beam and panel specimens measuring impact damage by recording contact force-time histories and visual inspections of crater development, including zones of
crushed and compacted foam. Sri Ravindrarahaj and Lyte (2008) tested several mix designs, varying percentage of fine aggregate replaced by polystyrene aggregate, and compared impact resistance to that of normal weight concrete. A 75 kg weight was dropped freely from 462 mm and impacted a bearing plate placed over an unconfined concrete cylindrical specimen resting on a load plate. Impact velocity was maintained at 3.01 m/s. The researchers determined that impact response time of the polystyrene concrete increased with an increase in polystyrene content and more energy absorbing capacity was achieved when compared against normal weight concrete. Sabaa and Sri Ravindrarajah (2000) observed that the impact resistance of EPS concrete was increased by the addition of polypropylene fibers by increasing the strains of first cracking from 13 to 40%. Sabaa and Sri Ravindrarajah (2000) and Sri Ravindrarahaj and Lyte (2008) both investigated energy absorbing capabilities of polystyrene concrete by changing the concrete density and aggregate types.

Shah and Topa (2014) conducted both quasi-static and dynamic drop weight tests on unconfined expanded polystyrene crushable foam and completed numerical simulations of quasi-static compression tests using LS-DYNA. The drop weight tests were carried out using a long rod projectile with semispherical end that penetrated into an unconfined EPS foam block. The researchers used the MAT_CRUSHABLE_FOAM material model within LS-DYNA (Hallquist 2013; LSTC 2006) to represent expanded polystyrene foam. Based on comparison between numerical simulations and the drop weight tests, to avoid negative volume errors in the simulations resulting from large deformations, the stress-strain curve needed to be extended exponentially at large strains. Selected solid elements utilized one-point integration with hourglass LS-DYNA control type 2, which is a viscosity-based
control that helps reduce zero energy modes of deformation. To avoid mesh tangling in high compression areas, interior contact was utilized between solid elements within EPS foam which allowed larger element deformation and decreased distortion and negative volume. Even though researchers were able to enhance material model effectiveness within LS-DYNA for higher strain rates, they only qualitatively compared data from simulations to the experimental results. Du Bois et al. (2006) used a suite of tests, both quasi-static and dynamic, to develop material characteristics for polypropylene foams for use in LS-DYNA. The researchers used four different material models readily available in LS-DYNA to represent four foam classes: MAT_FU-CHANG and MAT_VISCOUS_FOAM were used to represent elastic foams and MAT_CRUSHABLE_FOAM and MAT_BILKHU-DUBOIS_FOAM were used for foams featuring permanent deformations. For numerical stability, it was essential to smooth the definition of the stress-strain curve. Size and shape dependency of the material characteristics were seen in soft foams most likely due to the outflow of air from the low strength open cell foam, a process that takes more time as the sample size increases.

To adequately ascertain damage levels from repeated impact within a specimen, a proven, nondestructive, experimental technique is needed. One such technique is X-ray Micro-Computed Tomography (CT). CT allows for non-invasive, three-dimensional (3D) evaluation of materials, including concrete that are used for various purposes (Bouvard et al. 2007). The most common technique, named transmission tomography or attenuation tomography, is based on the measurement of X-ray attenuation along a set of directions throughout the evaluated object (Baruchel et al. 2000). Yang et al. (2013) conducted in-situ micro CT tests of concrete specimens under progressive compressive loading to gain
a better understanding of 3D fracture and failure mechanisms at the meso-scale. Using an imaging analysis program, AVIZO, the researchers were able to segregate the aggregates, mortar, cracks and voids and track the crack evolution and damage using consecutive CT scans.

This paper combines dynamic testing of EPS crushable concrete with CT imaging and numerical simulations to develop a material model for use in impact simulations that accounts for strain rate effects. The material model can be used to assist with design and simulation of perimeter barrier systems to ascertain absorbed energy that will possibly protect them against purposeful or, more importantly, accidental impacts without compromising barrier structural integrity. Detailed discussion of the expanded polystyrene crushable concrete mix, dynamic testing, CT imaging and analysis, numerical simulations using LS-DYNA, and recommended material constants will be discussed in the following sections.

4.3 Expanded Polystyrene Crushable Concrete

All concrete test specimen mixing and curing was performed in accordance with the American Society for Testing and Materials (ASTM) C192 (2013) standard with a few noted exceptions to accommodate mixing with expanded polystyrene. A detailed description of the mixing procedure is outlined in Doyle (2015). Standard specimens with a diameter of 101.6 mm and a height of 203.2 mm were tested at 28 days to ensure proper strength gain from curing.
Based on literature, the polystyrene concrete’s properties will be considerably more consistent and predictable if cement paste fully covers each polystyrene sphere, creating a closed-cell foam (Baxter and Jones 1972; Chen and Liu 2004). This can only be achieved if the polystyrene’s volume fraction in the concrete mix design is smaller than the polystyrene’s bulk density percentage when it is unconfined. A volume fraction of 40% was used for testing, well below the minimum bulk density of 52% for loosely-packed uniform spheres, to ensure that the polystyrene concrete behaved as a closed-cell foam. The mix design used a lower amount of air entrainer and a water-to-cement ratio of 0.4 based on suggestions from Doyle (2015) and Doyle et al. (2016). Other details of the mix design are shown in Table 4-1.

<table>
<thead>
<tr>
<th>Component</th>
<th>Quantity (kg)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polystyrene</td>
<td>16.7</td>
</tr>
<tr>
<td>Water</td>
<td>335</td>
</tr>
<tr>
<td>Cement</td>
<td>837.5</td>
</tr>
<tr>
<td>Air Entrainer</td>
<td>1.64</td>
</tr>
<tr>
<td>Water Reducer</td>
<td>2.35</td>
</tr>
</tbody>
</table>

4.3.1 Quasi-Static Testing

Unconfined and confined quasi-static uniaxial compression tests were completed on the polystyrene concrete to determine baseline material properties. Unconfined uniaxial
compression tests were performed to determine the stress-strain behavior in accordance with ASTM C39 (2012) with a minor modification. Because post peak-stress behavior was important to obtain for the planned material utilization, samples were loaded at a constant strain rate instead of the constant stress rate stipulated in the ASTM C39 standard. Figure 4-3 shows the unconfined stress-strain curve for three polystyrene concrete specimens with peak compressive strengths of 0.90, 1.41, and 1.30 MPa, respectively. Confined compression tests were also performed to determine confined stress-strain behavior under quasi-static loading. The confined tests were performed by placing a specimen in a steel tube (inner diameter of 103.2 mm; wall thickness of 12.7 mm) with a steel bottom plate and loading piston on top. The steel chamber provided stiff confinement to prevent shear failure and maximize polystyrene cell crushing during testing. A detailed description and accompanying figure of the confining cylinder test configuration will be given in the next section. Figure 4-4a displays the 28-day quasi-static confined stress-strain curve for polystyrene concrete cylinders showing the entire test data. Figure 4-4b shows the enlarged region (grey circle in Figure 4-4a) of the stress-strain curve showing the linear-elastic phase of EPS concrete. Using both stress-strain curves in Figure 4-4, all three phases discussed previously can be viewed. It can also be seen that when polystyrene concrete is confined, there is a dramatic strength increase because shear is not the controlling failure mechanism (Doyle et al. 2016).

For the confined configuration tests, two different loading rates were investigated. Specimens 1 and 2 were loaded at a rate of $3.48 \times 10^{-4}$ mm/mm/sec (relative strain of $1 \times$) while Specimens 3 and 4 were loaded at rate of $3.48 \times 10^{-3}$ mm/mm/sec (relative strain of
From Figure 4-4, a small effect can be seen between the two loading rates, especially at higher strain levels.

Figure 4-3. Quasi-static unconfined stress-strain curve for EPS concrete
4.3.2 Dynamic Testing

EPS crushable concrete cylindrical specimens having a diameter of 101.6 mm and an approximate height of 190.5 mm were placed in a confining steel tube with a base plate on the bottom and a loading piston on top as seen in Figure 4-5. A 304.8 mm long cylinder with a diameter of 88.9 mm, weighing 14.7 kg, was dropped on each specimen through a clear cylinder PVC pipe guide hitting a 96.5 mm high loading piston. The loading piston was used to distribute the load evenly on the concrete specimen and to confine the top of the specimen. The drop weight was released from two different heights (ground was the reference point) to achieve impact speeds at approximately a ratio of 1:2: Height 1 was set at 0.76 m and Height 2 at 2.90 m. Theoretically, the impact speed at Height 1 and 2 was
approximately 3.87 m/s and 7.54 m/s, respectively, and were confirmed using high speed video analysis as described below. Consecutive drops were conducted on each sample to achieve different strain levels including 8, 14 and 18 percent strain. These strain levels were chosen to represent a range of overall deformation to show the evolution of cracks in the concrete and regions of compacted polystyrene beads. Eighteen percent strain is the maximum strain that can be achieved with the current drop-test experimental setup.

Each drop test was recorded using a high-speed video camera to determine the speed of the drop weight at impact as well as the level of compaction of each polystyrene specimen. The high-speed camera system is fully autonomous and can capture images up to 29,000 frames-per-second with a maximum resolution of 1016 × 1016. Measurement of the height of the loading piston was also taken after each drop using calipers to calculate the cumulative strain of the confined concrete specimen. Data collected from each drop was used as a comparison for numerical simulations as discussed in a subsequent section.
Figure 4-5. Set up for drop test experiment

The first set of tests were conducted to observe general behavior of the EPS concrete samples and to ascertain whether the material behaved consistently under similar impact loads. As a result, repeatability tests were conducted on five samples at Height 2. Figure 4-6 shows the cumulative strain for each sample and the corresponding number of drops. The figure indicates that there is a very good repeatability among the tests, with some deviation existing for Test 2. The repeatability tests showed that expanded polystyrene crushable concrete specimens have predictability when subjected to higher strain rate loading, especially at higher strains.
After the repeatability tests were conducted, three specimens were tested at each of the drop heights for a total of six tests. Tests 2, 3, and 4 were conducted at Height 1 achieving 8%, 14%, and 18% strains, respectively; Tests 7, 8, and 9 were conducted at Height 2 achieving 8%, 14%, and 18% strains, respectively. When a specimen reached a particular level of strain (8, 14, or 18 percent), it was removed from the confining chamber by pneumatically pushing it out from its base. Figure 4-7 shows the compaction of each specimen with respect to the number of drops. From this figure it can be demonstrated that at Height 2 material response has less deviation between specimens than Height 1.
Figure 4-7. Cumulative strain of concrete specimens versus number of drops for Height 1 and Height 2

High-speed video analysis was conducted on Test 9 to determine the impact speed, maximum deformation, and total deformation of the specimen after each impact. This test was selected because the specimen had a higher strain rate at impact (i.e., Height 2) than Height 1 and achieved the highest strain level (i.e., 18%) when compared against all other specimens. Photron FASTCAM Analysis (2014) was used to track displacement of the drop weight and loading piston and calculate their average velocity and acceleration. Figure 4-8a shows video analysis of one drop that tracks displacement of the drop weight (two data points) and the loading piston (one data point). Figure 4-8b plots absolute displacement of each of the tracked data points with respect to time. Table 4-2 summarizes information collected from the high-speed video analysis, including the speed of the drop weight before contact with the loading piston, and the compaction for the specimen after
each drop. From this table it can be seen that there is a variation in impact speed likely due to frictional contact between the drop weight and guide tube. Also, occasionally during free fall of the drop weight it would contact the sides of the guide tube, thereby causing a decrease in the impact speed. Even with these discrepancies, the numerical model was able to be adjusted accordingly for each drop to accurately represent the impact speed of the drop weight. The data from Table 4-2 will be used as input into the numerical model of the drop weight tests as well as comparing the specimen compaction after each impact.

After each set of drop weight tests were completed, specimens were removed and stored for CT imaging. Figure 4-9 shows the eight specimens after CT imaging with Test-1 and Test-6 specimens representing undisturbed specimens (i.e., 0% strain) prior to impact testing. It can be seen that there was a fair amount of damage at the top and bottom of each specimen caused by removal from confinement chamber, transportation, and general handling.
Figure 4-8. Photron FASTCAM analysis of data points including drop weight and loading piston: (a) data point tracking from high-speed video; and (b) absolute displacement versus time for selected data points
Table 4-2. Video analysis results from Test 9 including drop weight speed at impact and compaction of specimen

<table>
<thead>
<tr>
<th>Drop</th>
<th>Drop Weight Speed at Impact (mm/s)</th>
<th>Final Compaction (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>7132</td>
<td>5.0</td>
</tr>
<tr>
<td>2</td>
<td>6892</td>
<td>4.96</td>
</tr>
<tr>
<td>3</td>
<td>6935</td>
<td>5.2</td>
</tr>
<tr>
<td>4</td>
<td>7201</td>
<td>5.3</td>
</tr>
<tr>
<td>5</td>
<td>6647</td>
<td>4.5</td>
</tr>
<tr>
<td>6</td>
<td>7550</td>
<td>4.2</td>
</tr>
<tr>
<td>7</td>
<td>7307</td>
<td>4.1</td>
</tr>
<tr>
<td>8</td>
<td>7543</td>
<td>3.8</td>
</tr>
<tr>
<td>9</td>
<td>7364</td>
<td>2.7</td>
</tr>
</tbody>
</table>
Figure 4-9. Expanded polystyrene crushable concrete specimens pre- and post-drop weight testing at 4 strain levels (0, 8, 14, and 18 percent) for: (a) drop Height 1; and (b) drop Height 2
4.4 LS-DYNA Numerical Models

Two numerical models were created using LS-DYNA to replicate quasi-static and dynamic drop weight tests described in the previous sections. Using dimensions described previously, individual models were created for each test using the same selected constitutive model for expanded polystyrene crushable concrete. Figure 4-10 details each of the numerical models which will be discussed in more detail in subsequent sections. Figure 4-10a shows quasi-static numerical simulations without strain rate effects incorporated in the material model, while Figure 4-10b shows dynamic drop weight test simulations that incorporate strain rate effects. Sections that follow will describe constitutive models selected for all parts of the model, with an emphasis on the constitutive model used for the expanded polystyrene crushable concrete, and a detailed description of results from quasi-static numerical models and dynamic drop weight tests.
4.4.1 Material Constitutive Model

LS-DYNA Material Type 16, “Pseudo Tensor” (Hallquist 2013; LSTC 2006) was utilized to model the expanded polystyrene crushable concrete. This material model was selected because it is the next progression from LS-DYNA Material Type 5, “Soil and Foam” and only constitutive model for foam to incorporate strain rate effects. From previous quasi-static simulations completed by Dubois et al. 2006, Material Type 5 was
successfully used to predict force-displacement curves for unconfined and confined compression tests obtained from laboratory testing. Two different response modes exists for Material Type 16: (1) Mode I, which is well suited for implementing standard geological models; and (2) Mode II, which uses two shear strength curves with damage and is best used for modeling concrete. Response Mode I was selected herein because it implements well founded geological models like Mohr-Coulomb yield surface with a Tresca limit as shown in Figure 4-11 (Hallquist 2013).

![Figure 4-11. Mohr-Coulomb failure surface with Tresca limit](image)

Tabulated values for the selected constitutive curve follow Equation (1) (Labuz and Zang 2012):
\[
\sigma_1 = \sigma_3 \tan^2\left(45^\circ + \frac{\phi}{2}\right) + 2c \tan(45^\circ + \frac{\phi}{2})
\]

where \( \sigma_1 \) is the major principal stress, \( \sigma_3 \) is the minor principal stress, \( \phi \) is the friction angle, and \( c \) is the cohesion for the material.

In addition to Mode I using a tabulated stress difference versus pressure (Figure 4-11) for the shear failure surface, it also uses a compaction curve (volumetric strain versus pressure) via implementation of an Equation of State (EOS) technique. LS-DYNA EOS Form 8 with compaction is required with this constitutive model. This EOS follows the general equation below for pressure in the loading phase:

\[
P = C(\varepsilon_v) + \gamma T(\varepsilon_v)E
\]

where \( P \) is pressure, \( C(\varepsilon_v) \) is the volumetric strain as a function of compression, and \( \gamma T(\varepsilon_v)E \) is the volumetric strain as a function of tension. Volumetric strain is defined as negative in compression and pressure is defined as positive in compression. Material unloading occurs along the unloading bulk modulus curve to a defined pressure limit. Reloading follows the unloading path to the point where unloading initiated and continues on the loading path as pressure increases. Figure 4-12 shows pressure versus volumetric strain for the expanded polystyrene crushable concrete as load increases. This curve was determined from static confined compression tests described in a previous section.
The last critical parameter for the “Pseudo Tensor” material model is defining a strain rate sensitivity multiplier for the principal material, which, for this study, is a multiplier of the yield strength determined by the strain rates experienced by the material. A sensitivity analysis was conducted to determine strain rate multipliers for different strain rates for the expanded polystyrene concrete. Two points were used to determine the strain rate sensitivity multiplier with the first point being 1 to match the quasi-static tests (i.e. no rate effects) and the second point was optimized to match the drop weight test numerical model. Figure 4-13 shows the selected sensitivity multiplier for Material Type 16.

**Figure 4-12. Pressure versus volumetric strain curve for LS-DYNA EOS Form 8 with compaction**
4.4.2 Quasi-Static Simulation

The quasi-static numerical model, shown in Figure 4-10a, was created to calibrate the developed static material model discussed in the previous section. The expanded polystyrene crushable concrete was modeled using eight-node fully-integrated solid elements to eliminate hourglass modes during analysis. The mesh was discretized using 48 elements in the circumferential direction and 32 elements along the length of the cylinder. The steel base, confinement cylinder, and loading ram were all modeled as rigid elements. The loading ram was assigned a prescribed motion with a constant loading rate, similar to what was observed during physical testing.
Material properties used in Material Type 16 for the expanded polystyrene crushable concrete are summarized in Table 4-3. The friction angle, which helps determine the pressure versus yield stress curve, and Poisson’s ratio were determined from a previous parametric study that used Response Surface Methodology (RSM) to optimize those variables (Box and Wilson 1951). RSM method uses a sequence of designed experiments to determine which variables affect the response variable of interest and then use a second-degree polynomial to obtain an optimal response. In this case, the optimal response was defined as minimizing the difference between force-displacement behavior observed during the physical compression test and that predicted by the numerical simulation. Results from these comparisons will be discussed in the next section.

Table 4-3. Summary of expanded polystyrene concrete material parameters

<table>
<thead>
<tr>
<th>Density (ton/mm³)</th>
<th>Shear Modulus (MPa)</th>
<th>Poisson’s Ratio</th>
<th>Tensile Cutoff (MPa)</th>
<th>Friction Angle (deg.)</th>
<th>Bulk Modulus (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.122×10⁻⁹</td>
<td>85.3</td>
<td>0.038</td>
<td>1.18</td>
<td>46.2</td>
<td>50</td>
</tr>
</tbody>
</table>

4.4.3 Drop Weight Simulations

Based on the drop weight testing protocol described in the Dynamic Testing Section, a numerical model was created as shown in Figure 4-10b. The model matched the physical test with respect to size and geometry of the specimen, confinement chamber, and drop weight. For the simulations, the loading piston was larger in diameter than the
specimen to create a uniform pressure load during impacts. Lastly, the drop weight sits directly on top of the loading piston.

The expanded polystyrene crushable concrete, base, cylinder, loading ram and drop weight were presented using the same approach described earlier for the quasi-static test. The constitutive model utilized the strain rate sensitivity multiplier shown in Figure 4-13 for EPS concrete. The initial velocity assigned for each respective drop is listed in Table 4-2.

A spring-back analysis in LS-DYNA was utilized to model consecutive drops onto the specimens. For each drop, sufficient analysis time was utilized to allow the system to reach a steady-state condition. Then, pertinent information from the last time step within the current simulation, which included the deformed geometry of the expanded polystyrene crushable concrete and residual stresses and strains developed within its elements, was utilized as initial conditions for the next drop.

4.5 Results and Discussion

This section will present and compare numerical model results for quasi-static and dynamic drop weight tests against laboratory test data to assess the efficiency and accuracy of Material Type 16 in predicting the response of EPS concrete subjected to different loading rates. Lastly, discussion of the segmentation and visualization of the CT scans for each of the specimens from Drop Height 2 are qualitatively analyzed.
4.5.1 Quasi-Static Test

A comparison between the experimental confined quasi-static test and numerical simulation was completed to ensure Material Type 16, “Pseudo Tensor” was capable of predicting a comparative quasi-static response. The numerical model used the same prescribed motion of the loading ram that was implemented for the laboratory tests. Figure 4-14 shows the load-deflection curve for the experimental test and the numerical simulation. There is a good agreement between the curves and both show a yield stress of approximately 10 kN. The numerical model does exhibit softening post yield, but it is within the deviation of the experimental data collected.
4.5.2 Dynamic Drop Weight Tests

A comparison between experimental tests and numerical simulations was conducted using the numerical model shown in Figure 4-10b. Experimental specimen displacement was monitored as described above using high-speed cameras. For the numerical model, average displacements in the z-direction of the top and bottom of the specimen were tracked for comparison. Figure 4-15 plots the entire displacement time-histories for the experiments and numerical models. Termination of the numerical simulation was at 0.05 seconds, where specimen dynamic equilibrium was satisfied. The figure shows that however, experimental dynamic equilibrium took much longer to achieve. This was attributed to the drop weight and loading piston repeatedly rebounding.
off of the specimen until reaching final equilibrium at 0.25 seconds. For comparison purposes, a dashed line was used to extend the computational time domain to match that for the physical tests. From this figure, it can be seen that the material constitutive model was able to predict maximum and final displacement of the expanded polystyrene concrete for Drop 1. For all subsequent drops, only the first 0.05 seconds of the specimen displacement was displayed so maximum specimen response could be clearly indicated.

![Displacement-time history of Drop 1 for numerical simulation and experimental test](image)

**Figure 4-15. Displacement-time history of Drop 1 for numerical simulation and experimental test**

Figure 4-16 displays all dynamic experimental test results and their numerical simulations for Test 9. It can be seen from these figures that the constitutive model accurately predicts overall response of the expanded polystyrene concrete material under dynamic impacts. Some discrepancies do exist, especially for Drops 5, 8, and 9. For all
discrepancies, there is a larger maximum displacement numerically than experimentally. For Drops 5 and 8, the difference was attributed to the drop weight not hitting the loading piston squarely and load being transferred to the confinement steel and not directly to the specimen (confirmed using high-speed video); whereas numerically, the specimen is loaded perfectly every drop and all of the force from drop weight is transferred to the specimen. The last discrepancy was Drop 9 shown in Figure 4-16i. This discrepancy arises from the loading piston being almost flush with the confinement steel due to the amount of compression the specimen had already accumulated. Therefore, some of the energy from the drop weight was transferred to the confinement steel and the high-speed image analysis was not able to capture the loading piston as it traversed into the confinement steel. Besides these small discrepancies between the numerical and experimental results, the accuracy of the material constitutive model is considered as very good. Table 4-4 summarizes maximum and final displacements for the representative experiments and numerical simulations. The table also compares the cumulative strain with respect to the number of drops.
Figure 4-16. Comparisons of representative displacement-time histories for numerical simulations and experimental tests at Height 2 for: (a) Drop 1; (b) Drop 2; (c) Drop 3; (d) Drop 4; (e) Drop 5; (f) Drop 6; (g) Drop 7; (h) Drop 8; and (i) Drop 9
### Table 4-4. Summary of representative drop tests for experimental tests and numerical simulations

<table>
<thead>
<tr>
<th>Drop</th>
<th>Speed (mm/s)</th>
<th>Experimental Max. Displacement (mm)</th>
<th>Numerical Max. Displacement (mm)</th>
<th>Experimental Final Displacement (mm)</th>
<th>Numerical Final Displacement (mm)</th>
<th>Experimental Cumulative Strain (%)</th>
<th>Numerical Cumulative Strain (%)</th>
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<td>2.7</td>
<td>3.4</td>
<td>20.0</td>
<td>20.5</td>
</tr>
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</table>

#### 4.5.3 Image Analysis

The eight specimens, shown in Figure 4-9, were sent to General Electric’s Inspection and Non-Destructive testing facility for CT imaging to nondestructively evaluate the damage within each specimen. A Phoenix v|tome|x m scanner was used for all tests. The scanner was set at 230 kV and 220 mA for a pixel resolution of 2024 x 2024 for Specimens 1 – 4 and 6 – 8 and 240 kV and 250 mA for a pixel resolution of 2024 x 2024 for Specimen 9. Every scanning sequence covering the entire sample length was performed in six rotations, which were digitally stitched together for a full three-
dimensional reconstruction of the sample. X-ray CT images were processed using ImageJ and Avizo 3-D Visualization Framework (Avizo 8.0.0 Edition) for data cropping, segmentation, and comparison purposes. The original CT image data was partitioned into void and non-void spaces according to their signature CT registrations (determination of thresholds).

4.5.3.1 Determination of Thresholds

The Line-Probe command was used to determine the proper threshold greyscale value to segment each specimen into two phases: (1) concrete; and (2) voids. Straight lines (e.g., line segment AB) were drawn on 2D images of the sample cross-sections as seen in Figure 4-17a, and variations of greyscale values (i.e. absorption contrast) along the lines were obtained as seen in Figure 4-17b. These variations were then carefully compared with the 3D images to determine the appropriate threshold for each specimen. Figure 4-17 shows an example of the Line-Probe command for Specimen 1 and the corresponding greyscale values. From this figure there are two distinct thresholds: concrete (>19000) and voids (<19000). This process was completed to determine the threshold of concrete for each specimen and is summarized in Table 4-5. The full grey scale is 0~65535. After the thresholds were determined, a segmentation process could be completed to segment each specimen into two distinct parts: (1) concrete; and (2) voids.
Figure 4-17. Determination of thresholds for Specimen 1: (a) Line-probe path AB; and (b) Variation of grey values along AB
Table 4-5. Information collected from CT images using Avizo 8.0.0 for Height 2 specimens

<table>
<thead>
<tr>
<th>Specimen</th>
<th>Number of Images</th>
<th>Lower Threshold</th>
<th>Upper Threshold</th>
<th>Total Concrete Volume from Slices</th>
<th>Total Concrete Volume of Specimen</th>
<th>Percent Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>0% Strain</td>
<td>1710</td>
<td>18000</td>
<td>32000</td>
<td>804178.2</td>
<td>804478.9</td>
<td>0.037%</td>
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<tr>
<td>8% Strain</td>
<td>1563</td>
<td>27000</td>
<td>65535</td>
<td>825108.0</td>
<td>825823.6</td>
<td>0.087%</td>
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<td>14% Strain</td>
<td>1470</td>
<td>26000</td>
<td>65535</td>
<td>872017.7</td>
<td>872240.5</td>
<td>0.026%</td>
</tr>
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<td>18% Strain</td>
<td>1210</td>
<td>22000</td>
<td>65535</td>
<td>894452.2</td>
<td>893527.5</td>
<td>-0.103%</td>
</tr>
</tbody>
</table>

4.5.3.2 Segmentation

Thresholds, as discussed in a previous section, were used in Avizo to segment the CT images into two phases: (1) concrete; and (2) voids. Since concrete cracking and polystyrene compaction were of interest, each of the specimens were divided into 15 slices along their height to compare compaction of the polystyrene beads and air voids (an increase in concrete volume). The segmentation procedure was able to separate the specimens into two distinct phases and cracks were visualized directly via specifying a threshold as discussed above. Features below the segmentation threshold, such as narrow cracks and small voids, are not resolved, but did not contribute a large percentage to the overall volume. For each of the slices, Avizo calculated the volume of concrete. Figure 4-18 shows the percentage of concrete along the height of each specimen experiencing 0%, 8%, 14%, and 18% strain levels for drop Height 2. In this figure, $H$ represents the height...
of the specimen after compaction, $H_o$ represents the initial height of the specimen, and $H/H_o$ is the normalized height. The percentage of concrete was calculated by dividing the calculated volume of concrete by the total, original volume of each slice. The total volume of each slice was calculated assuming the original diameter of the specimen and the height of each slice that changes with strain. This figure clearly shows there is an increase in concrete volume (conversely a decrease in voids) with higher strain. The points located at the top and bottom of the specimen are not clear indicators of the compaction level because there was damage as described earlier and shown in Figure 4-9. Figure 4-18 also shows that there is greater compaction at the bottom of the specimen due to reflective waves at this boundary. Lastly, a comparison of the volume of the concrete from each slice to the volume of concrete for the entire specimen is summarized in Table 4-5. There was little disparity between the slice volume and the total volume for each specimen, suggesting that the image analysis is accurate for distinguishing concrete from voids.
4.5.3.3 Visualization

Using Avizo for visualization purposes, orthoslices of each specimen were created. The orthoslices provide a qualitative means of understanding Figure 4-18. Three-dimensional views of each specimen can be used to investigate the distribution of expanded polystyrene beads and concrete as a function of strain. Figure 4-19a displays an image of a specimen with no deformation (0% strain), which indicates that there is a relatively uniform distribution of expanded polystyrene beads throughout the section and very few air voids from mixing. There is an area of high concrete concentration approximately one-third the height of the specimen from the bottom. This can be seen in Figure 4-18 via an
increase in volume of concrete. Figure 4-19b shows an image of a specimen impacted until it achieved 8% strain. From this image, compaction of the polystyrene beads can be seen at the top and bottom of the specimen. The center of the specimen did not show any significant amounts of deformation and aligns with the quantitative data in Figure 4-18. Figure 4-19c shows a specimen compacted to approximately 14% strain and the compaction of the polystyrene can clearly be seen at the top and bottom. This figure also shows development of a shear failure line near the top of the specimen. Figure 4-19d shows a specimen compacted to approximately 18% strain. This image clearly shows regions where the polystyrene beads have been fully compacted and the only voids remaining in the matrix are located near the center of the specimen.
Figure 4-19. Orthoslices of Specimens at Height 2: (a) 0% Strain; (b) 8% Strain; (c) 14% Strain; and (d) 18% Strain
4.6 Conclusions

The use of innovative materials in perimeters barriers is one of the primary areas of research in anti-terrorism design. In this study, a crushable concrete mix design was tested where expanded polystyrene (EPS) was used as the primary cell material. EPS has been proven to be a great energy absorbing material but literature available for material constitutive models is lacking. Quasi-static and dynamic drop weight tests were conducted on specimens to determine material constitutive parameters for use in numerical simulations. All experimental tests were completed on confined specimens and the dynamic tests consisted of multiple impacts to determine rate effects and the evolution and propagation of cracks within the cement/void matrix.

A LS-DYNA numerical model was developed to simulate quasi-static and dynamic drop weight laboratory tests. Material Type 16, “Pseudo Tensor” was used to represent the expanded polystyrene crushable concrete because of its ability to incorporate strain rate effects and its more simplistic constitutive model that had already been proven to accurately represent the material in a quasi-static state. Once the constitutive parameters were determined, the numerical models were used to model the representative laboratory tests. The numerical model was able to represent both the quasi-static testing and the consecutive drops from the drop weight tests. Using a springback analysis, the state of the specimen after each drop was saved and used as the start of the subsequent drop. The material constitutive model is recommended for use in the design of vehicular anti-ram barriers since rate effects were captured in the constitutive model and can accurately represent impact loading.
Lastly, CT scans were completed on four different specimens from the drop weight tests representing four different strain levels: 0%, 8%, 14%, and 18% strain. These levels were chosen so the evolution and propagation of cracks within the specimen could be monitored as compression strain increased. From the CT scans, the volume of concrete with respect to voids along the height of the specimen could be determined. As the compression increased, the relative volume percentage of concrete increased and that of voids decreased showing that the polystyrene beads were being crushed. Also, the bottom of the specimen exhibited higher amounts of crushing of the polystyrene beads. This was likely due to reflective waves from the bottom steel plate on which the specimen sat. The center of the specimen did not experience significant amounts of compression. Shear failure lines developed in all of impacted specimens but as the compression increased, the shear failure lines reached further toward the center of the specimen.
References


Barsotti, M. (20110) “Comparison of FEM and SPH for modeling a crushable foam aircraft arrestor bed.” *11th International LS-DYNA Users Conference*, Detroit, MI.


Photron FASTCAM Analysis (PFA) Ver.1.2.0. 2014


Chapter 5

Conclusions and Recommendations

5.1 Conclusions

Experimental tests of geo-materials interacting with rigid bodies under impact loading were conducted to collect critical information about the behavior of such system. Full-scale vehicular tests were conducted to assess the performance and behavior of single boulders embedded in AASHTO uniformly-graded coarse aggregate soil as an anti-ram vehicle barrier, while drop weight tests were conducted on expanded polystyrene crushable concrete to develop a material model capable of incorporating strain rate effects for use in vehicle barrier designs. Numerical models were developed for each experimental test to calibrate material constitutive models. The following conclusions can be made about the performance and behavior of a single boulder embedded in compacted AASHTO uniformly-graded coarse aggregate soil as a vehicle barrier:

- LS-DYNA Material Type 173, “Mohr-Coulomb (M-C)” was able to accurately predict the response of boulders and soil subjected to impact loads with calibrated material parameters.

- Calibration of M-C constitutive parameters for the backfill soil was conducted based on the results of a single vehicular impact. A sensitivity analysis indicated that shear modulus and cohesion had a greater influence on the global response of the system than the friction and dilation angles for compacted AASHTO soil.
• The boulder displacement during impact was relatively insensitive to the change of boulder stiffness. This is likely due to several reasons. First, the boulder’s response was largely elastic during the field tests as no fracture of the boulder was observed after the tests. Second, the boulder rotation and displacement were likely governed by the surrounding soil, which has considerably lower stiffness and strength than the boulder. Consequently, the boulder essentially behaved as a rigid mass transferring the impact momentum and energy to the surrounding soil.

• The calibrated numerical model was able to yield the global response of the system including the time-history of the translational displacement and rotation of the boulder and was in good agreement with field-scale test results. This suggests that the overall global response was dominated by the dynamic behavior of the truck and boulder system upon impact. Hence, a simple material model for soil and boulder is sufficient for simulating the tests conducted.

• However, depending on the response of the barrier and the type of impact, traditional finite elements within the calibrated numerical model may not be sufficient. For impacts causing large boulder and soil deformations, FEM simulations under-predicted boulder translation and rotation during impact, as well as truck ramping. This is most likely due to FEM being a grid based formulation that has difficulty simulating large deformations.

• Therefore, development of a computational approach that couples the Finite Element Method (FEM) and the Smoothed Particle Hydrodynamics (SPH) was required. Using SPH particles in regions of large deformation (near-field) and
FEM in regions where there is little soil disturbance (far-field) is advantageous in simulating the field performance of embedded structures under impact loading involving large deformation of soil.

- The optimal near-field SPH soil domain was determined to be 3/2 times the embedment depth around the front and sides of the boulder and four times the embedment depth behind the boulder.
- Renormalization approximation (FORM 1) and a smoothing length of 1.05 were used for all SPH particles. Renormalization approximation accounts for stress inaccuracies along boundaries. Higher smoothing length coefficients caused a weaker response of the model and material was less stiff. Lastly, there needs to be at least five SPH particles per finite element length at all FEM-SPH boundaries.
- The hybrid model is a computationally efficient model that takes advantage of SPH’s capabilities to model large deformations in the near-field and FEM’s more efficient formulation in the far-field.

The following conclusions can be made about the performance and behavior of expanded polystyrene crushable concrete:

- LS-DYNA Material Type 16, “Pseudo Tensor” was capable of predicting the quasi-static and dynamic response of EPS crushable concrete.
- A comparison between the experimental confined quasi-static test and numerical simulation was completed to ensure Material Type 16, “Pseudo Tensor” was capable of predicting a comparative quasi-static response. There
was a good agreement between the curves and both show a yield stress of approximately 10 kN.

- The constitutive model accurately predicts overall response (maximum and final deformation) of the expanded polystyrene concrete material under dynamic impacts for each consecutive drop.

- Using CT scans of the dynamically impacted specimens, the evolution of damage based on the level of strain could be determined. From the scans, there is an increase in concrete volume (conversely a decrease in voids) with higher strain. The top and bottom of the specimens are not clear indicators of the compaction level because there was damage caused by removal from confinement chamber, transportation, and general handling. Also, there is greater compaction at the bottom of the specimen due to reflective waves at this boundary. The center of the specimen did not experience any significant amounts of compression. Shear failure lines developed in all of impacted specimens but as the compression increased, the shear failure lines reached further toward the center of the specimen.

### 5.2 Recommendations for Future Work

#### 5.2.1 Soil Modeling

This study showed that a simplistic material constitutive model could be used to predict the response and behavior of barriers embedded in compacted AASHTO soil
subjected to vehicular impacts. However, not all geographic locations will have compacted AASHTO soil available. Additional research could be performed to further understand the effects of changing the material constitutive model to represent sandy or clayey soils. For sandy soil, the extent of the SPH domain in the near-field region may be larger while for clayey soil it may require less. Also, the constitutive model required to represent sandy and clayey soil may have to be different than Mohr-Coulomb failure criterion based on the behavior of each.

5.2.2 Expanded Polystyrene Crushable Concrete

This study showed that expanded polystyrene crushable concrete can be accurately used in numerical models for both quasi-static and dynamic loading schemes. Additional research could be performed to further understand where the EPS concrete could be placed in vehicular anti-ram barriers. Optimization and methodologies for fabrication and installation are required. After determining how and where EPS could be utilized within an anti-ram barrier, a full-scale test could be conducted to validate the optimized barrier design. One suggested use of EPS concrete would be to create modular slabs that would be embedded in front of and behind the boulder/barrier. As the barrier was impacted, the modular slabs would dissipate energy while providing rotational resistance for the boulder.
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