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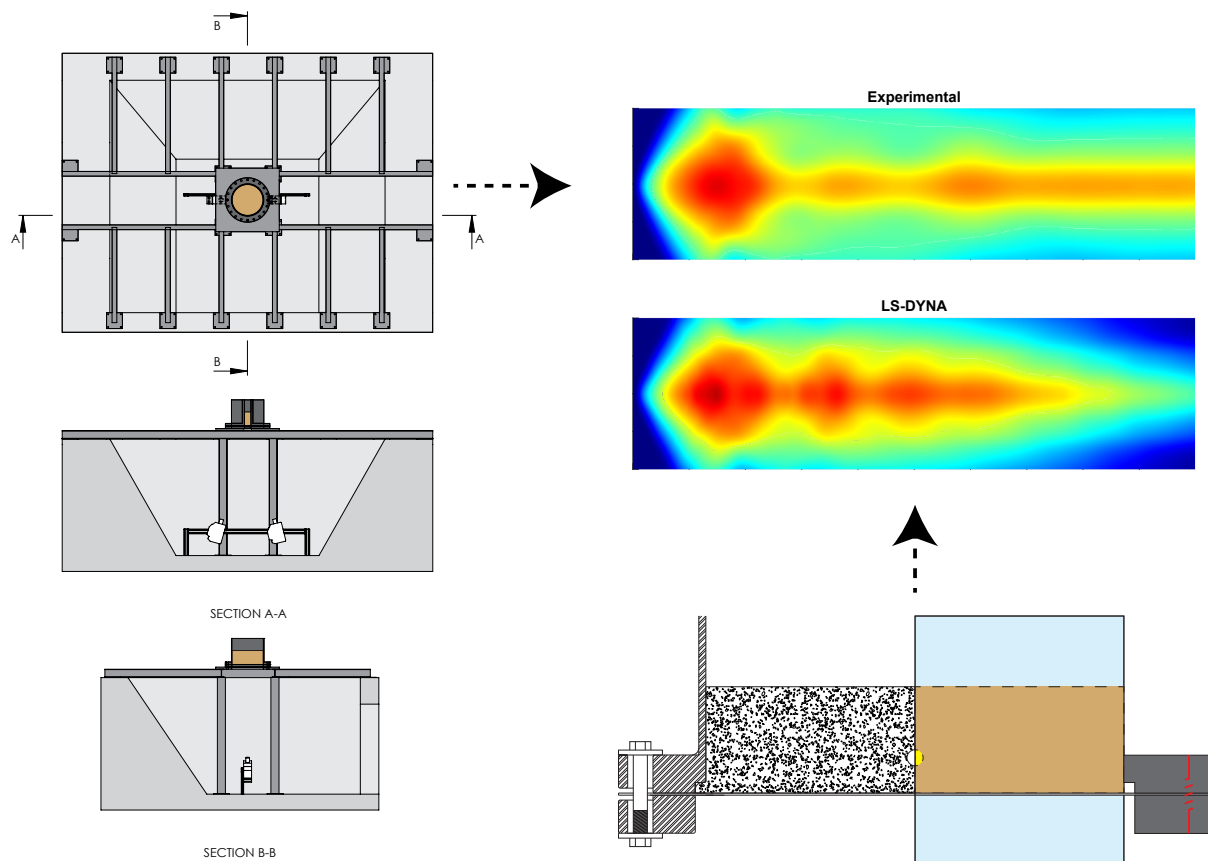
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Experimental Characterisation of Blast-Induced Ground Shock by High-Speed Digital Image Correlation of Structural Response

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Abstract

Subsurface explosions result in the propagation of stress waves through the soil, known as ground shock. This can damage buried and surface structures in events such as the detonation of unexploded ordnance from previous conflicts or deliberate attack. However, understanding of the mechanisms and magnitude of loading imparted to buried structures is limited by a lack of high-quality experimental data. Past testing utilised discrete measurement devices within the soil; results have often been characterised as incident, free-field values, when in fact they are influenced by the materials, size and shape of the measurement device. This paper describes an innovative approach to experimental determination of ground shock loading on a buried structure.

Uniquely, digital image correlation from high-speed video at 100k frames per second has been utilised to determine ground shock loading at reduced scale on a deformable plate. Testing was conducted using a steel pipe with a deformable aluminium base plate, filled with varying depths of uniform sand at a consistent bulk density and moisture content. A constant scaled burial depth of $0.434 \text{ m/kg}_{\text{TNTe}}^{1/3}$ was maintained using spherical PE10 charges with a varying stand-off distance. Two charge sizes were used, with nine tests conducted in total.

Displacement histories from over 2500 locations across the plate were extracted from each test, greatly increasing the resolution of data capture possible. The experimental data

has been used to validate numerical modelling approaches, using an existing soil material model. The numerical results show good agreement with the experimental data.

Keywords: Blast, DIC, Ground shock, UXO

1. Introduction

Unexploded ordnance (UXO) forms a serious threat to infrastructure and the built environment, as well as potentially causing injury or death to civilians. Approximately ten percent of the seventeen thousand tons of bombs dropped on London during the Second World War did not detonate [1], representing a large quantity of UXO. The UK Construction Industry Research and Information Association (CIRIA) introduced guidance for all construction sites to assess the risk from UXO in 2009 [2], with this continuing to pose a considerable threat to the built environment today. Therefore, it is important for the effects of an explosion in the ground to be well understood, to better inform the design of suitable mitigation measures upon discovery of UXO.

In the event of an explosion in soil, stress waves propagate through the soil in all directions. In the extreme near-field surrounding a charge, hydrodynamic behaviour occurs resulting in complex fluid-like behaviour of the soil [3], as individual particles are fractured by the high-magnitude stress. When deeply buried, this can result in the formation of a camouflet (a void in the soil formed by the rapid expansion of detonation product gases and high stresses of the explosion), or when shallow-buried, the stress wave reaches the soil-air interface and ejects the soil, resulting in crater formation – as explained by Grujicic et al. [4]. At further distances, stress propagates through the soil via approximately elastic-compressive particle vibrations and collisions, resulting in a radial particle acceleration and velocity – similar to the propagation of a P-wave from a seismic event (described as a condensational wave by Oldham [5]).

A wide array of experimental testing has been collated to produce empirical relationships, which have been used for decades for the prediction of ground shock loading within

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TM5-855-1 Fundamentals of Protective Design for Conventional Weapons [6] (as implemented in the computer program CONWEP), with this often seen to be the standard method of calculating ground shock in the literature. This document is not available for general access, but is known to re-produce the equations presented by Drake and Little [7] for ground shock. These formulae describe free-field (incident) behaviour where shock is transmitted through a continuous soil medium, with work needed to investigate the applicability of these to structures in the near-field range of a charge, such as building foundations subjected to blasts from accidental detonation of UXO.

1.1. Historic determination of loading

Ground shock loading has been experimentally measured in various ways. Lampson [8] established the validity of cube-root scaling for ground shock transmission through a large quantity of experimental work carried out in various soil types during the Second World War, including a large-scale test series using charge sizes from 8lbs (3.6kg) of TNT to 4000lb (1814.4kg) general purpose bombs. Lampson [8] also established empirical formulae for the prediction of ground shock loading parameters, from around ten thousand individual records of various experimental transient and permanent quantities.

Ingram [9] discussed the development of pressure transducers for recording stress propagation in soil, with stress measurements being accurate to $\pm 20\%$. They contended that the use of a stiffer material than the soil for a gauge results in it picking up more stress than the free-field, so it must be calibrated to account for this. On the other hand, a less stiff inclusion would under-register as soil would “arch around it”. Van Dongen and Weerheijm [10] introduced a pressure transducer with an integrated accelerometer, to allow for adjustment of the sensor’s structure-type behaviour to that of a free-field behaviour, tested with charges from 125 g to 500 g of TNT over 12 tests. Correcting for transducer motion brought tests at the same scaled distance into alignment. It was found that the pressure transducer alone measured “a peak pressure which was about 30 percent higher than the adjusted actual free field peak pressure.” Comparing the adjusted values to TM5-855-1[6], it was found that using an attenuation factor that was 8% lower than the recommended value gave better pre-

dictions of the incident pressure in this testing. The issue of sensor-soil interaction was also approached by Wang et al. [11], who explored the use of accelerometers encased in foamed concrete in an attempt to match the acoustic impedance of sand, with this tested using three small-scale charge masses. Acceleration profiles showed a higher peak value when the sensor was encased in the foamed concrete, which they interpreted as delivering a more accurate representation of the acceleration of the soil due to a reduced impedance mismatch.

Ingram [12] later described testing and resulting waveforms from large-scale buried explosive tests with varying ground conditions and degrees of containment. From this it could be seen that geotechnical properties have a large effect on ground shock propagation. Coupling factors were introduced by Ingram et al. [13] (similarly to in earlier work by Lampson [8]), according to testing in sandstone, to describe the proportion of explosive output when compared with a reference condition – i.e. for airblast this is against a free-air charge, and for ground shock and cratering this is against a deeply-buried charge. High sensitivity to confinement was found in (approximately) radial velocity, but not in velocity perpendicular to the direction of propagation. Bergeron et al. [14] conducted testing with 100 g C4 charges at varying depths of burial. They reported specific impulse, pressure, and arrival time for each, the latter two of which have been compared against the results from this study in Sections 2.4.5 and 4. Ehrigott [15] carried out testing with buried accelerometers and soil pressure gauges. Three soil types were investigated: clay, silty-sand and uniform sand. It was found that “the times of arrival of the stress occur slightly earlier than the times of arrival of the motion”, as would be expected due to the inertial resistance of the sand particle masses, so it is important to distinguish between these arrival times.

Melosh [16] investigated the rise time of shock waves from underground nuclear explosions, finding that the rise time is related to the material properties of the intervening material between the explosion source and the measurement point, rather than a feature of the explosive yield (provided that a “quasisteady” wave state is achieved, usually by a yield of 1kT or greater). Wu et al. [17] measured ground shock from large-scale conventional explosive trials involving charge weights of 0.5, 2.5 and 10 tons, buried within tunnel structures in rock, overlain with soil. Peak particle velocity, acceleration and principle frequency were

recorded in free-field rock, at the rock-soil interface, and at the soil surface. Transmission media had a large influence on the attenuation of the stress waves, and it was found that large charges resulted in increased damage to the rock, resulting in a lower peak particle acceleration in free-field rock at the same scaled distance as smaller charges. Xia et al. [18] carried out large scale experimental testing with 1t and 10t TNT surface charges. Velocity and acceleration sensors were buried in boxes 0.5m below the surface. Only data from the 1t test was recovered and reported.

Small-scaled centrifuge tests were carried out by Charlie et al. [19] (350mg and 1.031g PBX 9407 detonators (with a TNT equivalence factor (TN_{Te}) of 1.14)), and compared against 8.5kg Powermite high explosive water gel (TN_{Te} 0.82) ‘prototype’ tests. They found similarity in results at the same scaled distance for peak particle velocity, stress and scaled acceleration – as well as agreement with Drake and Little [7] predictions, showing that these predictions are suitable for use over a large array of charge masses. They also found that velocity, stress and acceleration are affected by soil saturation, with the lowest measurements seen in dry soils. Hansen [20] also carried out testing in a centrifuge, measuring ground shock in “dry sand, partially saturated sand, and clay-sand” with charges of 0.2–1.0 g of Detasheet and C4. Acceleration and stress-time histories were recorded, with peak values reported. Additionally, curve fits for acceleration and stress were derived for each soil type (Eqns. A.6 & A.11 in Appendix A for 10% saturated sand). Guan et al. [21] extended this testing of gram-scale charges under hypergravity conditions in a centrifuge, by varying depth of burial (DOB) in dry sand in order to understand the relationship between DOB and coupling of explosive energy with the ground. It was found that tests at a scaled DOB of $0.4 \text{ m/kg}_{\text{TNTe}}^{1/3}$ had a coupling factor of 0.35, whilst those at $0.8 \text{ m/kg}_{\text{TNTe}}^{1/3}$ were fully coupled (in comparison to a deeply-buried charge condition).

1.2. Empirical prediction

Many empirical formulae have been introduced for the prediction of ground shock loading, often tuned to the results of a particular experiment or study. Those discussed herein are outlined in Appendix A.

Westine [22] presented dimensionless empirical expressions for ground shock loading (Eqn. A.1 and others), based on experimental data from testing involving yields of 0.01kg to 34.9kT. Westine and Friesenhahn [23] went on to present a dimensionless empirical expression for ground shock pressure in unsaturated sand / silt (Eqn. A.7), and another for saturated clay. For the saturated case, a “modified hydrodynamic solution” was used with clay acting as a high-density “heavy water”.

Drake and Little [7] gathered data from over 100 tests from the Second World War up to 1983 to derive empirical expressions (Eqns. A.2, A.4, A.8 and others), which were subsequently included in TM5-855-1 [6]. They discussed seismic velocities and ground shock attenuation for various soil types, finding that “in granular media” a higher relative density results in weaker attenuation. It was established that saturation has a large effect on ground shock transmission in cohesive and loose cohesionless soils, but dense granular soils are largely unaffected as inter-particle contacts contribute the majority of the soil stiffness which influences shock propagation. Also, the coupling factor (f), discussed in [13], was developed for soil with varying depth of burial of charge centre.

Drake et al. [24] later tuned these predictive expressions, “using a more sophisticated material model”, in order to improve accuracy, particularly at near-charge positions (Eqns. A.3, A.5, A.9 and others). They considered a “primary loading wave velocity” at “higher stress levels”, as well as seismic velocity. It was described how for the nearest $0.155\text{m/kg}^{1/3}$ there is an “expanded cavity” (camouflet) “in most rocks”, where more characteristically hydrodynamic behaviour is exhibited.

Leong et al. [3] proposed a new dimensionless formula for the prediction of peak ground shock pressure (Eqn. A.10), which better represents the data gathered in their experimental testing. They carried out 2 experimental tests (with pressure data recorded from the equivalent of 3 trials in the second test) using PETN charges of varied masses, burial depths, and geometries; one in “saturated sandy clay”, and one in 85% saturated “dense sand”. It was found that the TM5-855-1 [6] predictions overestimated all pressure recordings when using the recommended attenuation factor and acoustic impedances from soil descriptions. If the actual recorded acoustic impedances were used, better agreement was reached between

174 predictions and measured pressures.

175 1.3. Computer modelling

176 Lu [25] used experimental data from large scale testing in rock and numerical modelling
177 in Autodyn3D, to gather data for the training of an artificial neural network (ANN) for the
178 prediction of a ground shock loading event. It was found that two ANNs were required, one
179 to predict the horizontal component of ground shock, and one for the vertical component.
180 Thus, the angle of a point from the charge centre is an important variable in determining
181 the direction of acceleration, and therefore the horizontal and vertical components.

182 Laine and Larsen [26] utilised a multi-material Euler solver in AUTODYN, with axisym-
183 metric 2D modelling of shock propagation in soil for 4 varied soil types, taken from those
184 defined in [27] (from dry sand to fully-saturated clay), and 13 burial depths. They found
185 that the coupling factor, f , tends to 1 at $d = 0.6 \text{ m/kg}^{1/3}$ (as in Figure 1, with different cou-
186 pling factor curves shown for pressure and kinetic energy distribution). Further, they found
187 that a reduced scaled distance to a target results in a greater coupling factor, and varying
188 the angle from the charge to a target location (horizontal, 45 degrees down, and vertically
189 down) also alters the coupling factor. Laine et al. [28] later used Autodyn to model a “sus-
190 pended piston-spring system buried in sand subjected for ground shock from an explosive
191 charge”, to compare against testing from Hultgren [29]. The numerical model was used to
192 understand work done in the system, with it being found that work done is not inversely
193 proportional to piston mass (as would be expected for an impulsive load) and thus indicat-
194 ing that the mass of soil between the charge and the target has a large effect on structural
195 displacement. They described that the pressure experienced by the piston was made up of
196 two regimes, an initial inertially-controlled peak pressure, followed by a stiffness-dominated
197 longer-term pressure. Laine et al. [30] extended this by introducing a 2 degree-of-freedom
198 model with a mass of sand between the charge and the face of the piston in a cone shape,
199 with this compared against numerical modelling results. They argued that the much lower
200 spring stiffness compared to sand stiffness results in the continued movement of the sand
201 into the easily-compressed piston. Laine et al. [31] further investigated this, showing that

normalised particle velocity increases by about 60-70% after the shock reaches the piston at varying scaled distances, as the stiffness of the piston is considerably less than that of free-field soil.

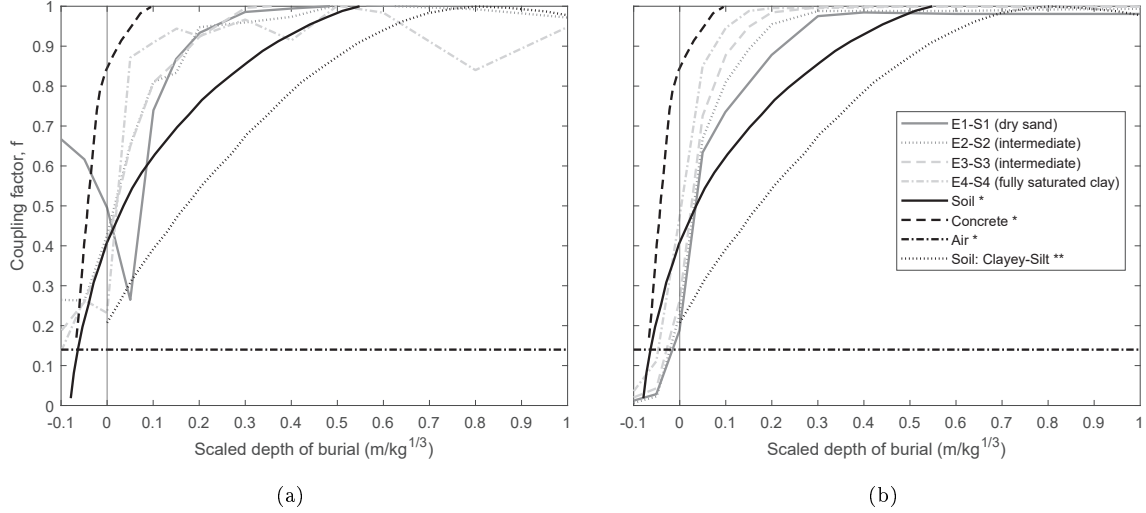


Figure 1: Coupling factor curves from Laine and Larsen [26], with * Drake et al. [24] and ** Lampson [8] curves for comparison, plotted against scaled depth of burial: (a) Pressure based coupling factor (at $Z = 3.69 m/kg^{1/3}$, angle -45 degrees), (b) Kinetic-energy based coupling factor (at 3ms analysis time)

Shelton et al. [32] investigated the Drake and Little [7] and Drake et al. [24] coupling factor suitability for various ground shock parameters. They suggested that new coupling factor curves are needed for pressure and velocity separately. This work utilised experimental data and corresponding prediction curves generated by the computer program FOIL [33]. Additionally they used Zapotec Eulerian/Lagrangian numerical modelling, validated against the experimental data. This suggested that the existing coupling factor curve represents a lower-bound on peak stress and peak particle velocity, and upper-bound curves are needed for the design of protective structures. Laine et al. [34] also investigated the Drake and Little [7] and Drake et al. [24] empirical expressions, comparing against 1-D AUTODYN modelling of ground shock tests in dry soil. They found that the Drake et al. [24] velocity curve was accurate enough to be retained as-is, but proposed modifications to the displacement curve.

1.4. Influence on structures

Various studies have investigated the effects of ground shock loading on structures, with complex building structures often investigated through numerical modelling. Wu and Hao [35] investigated combined ground shock and air shock loading from a surface charge in Autodyn2D. Loading was either separated or combined as a result of the “time lag between ground shock and airblast pressure arrival”. Wu and Hao [36] went on to enhance this work, by numerically modelling a 1-storey masonry-infilled reinforced concrete frame structure under separate and combined air shock and ground shock loading using LS-DYNA. They found that at small scaled distances, airblast dominates, but at large scaled distances, due to faster attenuation of air shock than ground shock, the “ground shock will have more pronounced influence than airblast load.” Single degree-of-freedom models were used by Tolani et al. [37] to understand effects of separate and combined air blast and ground shock loading on multi-storey buildings from surface blast. It was found that for shorter structures (less than six storeys), air shock dominates, but for a 12-storey building, ground shock has a strong influence up to a range of 10 m for a 1000 kg TNT charge ($Z = 1 \text{ m/kg}^{1/3}$). Thus, for tall, “flexible buildings” (20-30 storeys), ground shock may have a significant effect on structural response at shorter standoff distances.

Lu et al. [38] used an electromagnetic shaker to create ground shock type loading for 1:12 scale models of 2 and 3 storey reinforced concrete frames, as a simulation of ground shock from a blast. They investigated how the frequency spectra of ground shock loading affects the structural response, finding that higher frequencies (30 Hz and above) result in local mode resonance being more critical than global structural resonance.

Zhou et al. [39] described how in-structure shock is influenced by acceleration-time history, rather than specific impulse (which influences deflection of a single structural member or panel). In-structure shock is characterised by the failure of equipment due to excessive acceleration, with limited damage to the main structure. Zhou et al. [40] further worked on understanding and predicting acceleration-time histories of ground shock loading, to determine global response of a buried structure. Theoretical predictions were produced for global displacement, velocity and acceleration. Experimental testing was conducted in sand, with

245 a hollow steel box buried at a radial position, at the same DOB as the charge. Ground
246 shock pressure was recorded in the free-field, and near the box, with two accelerometers
247 installed within the box. Numerical modelling in ANSYS/LS-DYNA was also conducted to
248 validate the findings. Inclusion of rise time gave a better representation of true acceleration
249 in the analytical prediction, falling much closer to the measured acceleration. Additionally,
250 increasing mass per unit area of the structure resulted in exponentially reduced peak global
251 accelerations. Further, it was found that peak acceleration scaled linearly with seismic ve-
252 locity of the soil medium. They also found that for the same scaled stand-off distance, but
253 a greater charge mass, peak acceleration also reduced non-linearly.

254 Hacıfendioğlu and Alpaslan [41] investigated application of “simulated blast-induced
255 ground motions” on a masonry chimney structure, founded on rock, by numerical modelling
256 in ANSYS. They found significant deflections and stresses were generated causing cracking.
257 Ibrahim and Nabil [42] studied blast loading of pile foundations using finite element mod-
258 elling in ABAQUS, with a raft and piles modelled in layers of silty-clay and stiff clay, and a
259 457 kg TNT above-ground charge implemented using CONWEP. They investigated various
260 load mitigation techniques including barrier walls of varying materials and an open trench
261 surrounding the raft.

262 McKenzie et al. [43] considered ground shock in the context of structural damage from
263 IEDs or VIEDs – (Vehicle) Improvised Explosive Devices, investigating the combined loading
264 from an air blast wave and ground shock propagation. They examined this in the context of
265 the 1995 Oklahoma City bombing, finding that ground shock is predicted to have contributed
266 a peak displacement of 350 mm and a peak velocity of 11 m/s at a column at 4.75 m radius
267 from charge centre (values calculated by CONWEP) - demonstrating that it was a major
268 contributor to the collapse. Jahami et al. [44] simulated the ground shock effects of the
269 2020 Beirut port explosion on the pile foundations of the nearby grain silos using coupled
270 Eulerian-Lagrangian numerical modelling in ABAQUS, validated against in-the-field findings
271 of crater size and silo damage. It was found that the top of the piles experienced significantly
272 more lateral displacement than the bottom. Additionally, the piles nearest to the explosion
273 centre failed in tension (in keeping with the uplifting of the raft foundation observed on-site),

with the furthest piles failing in compression (again observed on-site with the raft settling).

Le and Chin [45] proposed two energy equivalent impulse methods to simplify non-uniform ground shock loading for the estimation of steel plate deformation from near-field charge locations into an equivalent UDL across the plate. The deformation resulting from these approximations was compared in LS-DYNA against that from the original loading, found using the 1D hydrocode FoilW, with reasonable accuracy achieved at medium to long standoff distances ($Z > 0.4 \text{ m/kg}^{1/3}$).

At much longer distances, McCaughey [46] stated that building damage is often mistakenly alleged by lay people to result from explosions due to exaggerated perceptions of explosion effects. Peak particle velocity limits were discussed, for the avoidance of damage to different classes of structures, which could be used to determine if damage was likely to have been caused by a given explosion event. It was observed that the author perceived an “appreciable” magnitude of vibration from an explosion at a large distance, as a result of the “startling” loud noise, but when compared against vibration from other sources such as a person jumping (measured as a higher magnitude of vibration), the floor vibration was “not appreciated as unusual”. This effect may lead a non-expert to conclude that building damage, such as cracking, only noticed after a distant explosion, was resultant from the explosion, rather than some other (often true) cause.

1.5. Summary

Much has been done to understand the transmission of ground shock through soils, usually finding incident, free-field quantities. These require often speculative assumptions of soil-structure interaction mechanisms to be made to establish the loading impinging upon a structure (especially in the near-field). This study aims to improve confidence in our ability to accurately quantify and predict (via numerical modelling and/or simplified expressions) the loading that can be expected in a ground shock structural interaction situation. In order to achieve this, new experimental approaches are required to better inform our understanding of the mechanisms of ground shock transmission and the associated transfer of loading to a structure. The following sections describe how this has been accomplished through a

simplified experimental setup, wherein a deformable plate represents a buried structure, the recorded deformation of which has been used to validate a numerical model, in turn used to extract pressures within the soil. This has then been compared against the existing literature predictions to understand the validity of their use for design and hazard mitigation purposes.

2. Experimental testing

2.1. Context

2.1.1. Blast quantification with digital image correlation (DIC)

Digital image correlation is an established technique which uses digital videography to determine the displacement of a target specimen by the correlation of locations of markers (spread over the target as a ‘speckle pattern’) against a reference position in digital images [47]. In this work, three-dimensional DIC is utilised via a stereo high speed video (HSV) camera setup in order to capture out-of-plane deformations. Reu and Miller [48] discussed the use of DIC at high speed to capture extremely fast transient events, such as explosive loading of a plate. At the time, the unavailability of HSV cameras that were capable of recording at extremely high frame rates, with a suitable number of frames captured and a reasonable image resolution, limited the use of this technique.

However, more recently, testing to quantify plate deformation under blast loading using DIC has been conducted by Curry and Langdon [49]. This testing utilised varied charge masses and standoff distances (SOD) with cylindrical PE4 charges in air in the near-field of 3 mm thick high-strength steel plates. Recordings were made at 30,000 frames per second (FPS) at a resolution of 1024 x 76 pixels. This was later extended by Rigby et al. [50] to allow for direct comparison between inferred specific impulse from plate velocity and areal density (from DIC recordings) against recorded pressure-time histories from a direct-measurement technique utilising Hopkinson Pressure Bars. Curry et al. [51] compared the uptake and distribution of specific impulse between DIC results and discrete location recordings for identical tests, also using the Hopkinson Pressure Bar technique. DIC from ultra-high speed video at 100,000 FPS was used to establish the loading on 3 mm Strenx 700

steel plates from PE10 charges of masses from 10 g to 60 g in air. Inherent errors due to image noise in two different camera systems were also established, with work conducted to ensure that the setup was optimised to reduce these effects.

3D DIC has also previously been applied to shallow-buried charge testing in soils (a landmine-type threat where the load is measured against a structure above the surface) by Tiwari et al. [52] with very small charges (1 g) at a resolution of 256 x 256 pixels at 26143 FPS and 128 x 128 pixels at 61538 FPS over 150 x 150 mm or 75 x 75 mm of a plate respectively. This was furthered by Curry [53], wherein a deformable plate subjected to buried charge loading was recorded at a resolution of 1024 x 76 pixels at 30000 FPS over 300 x 25 mm. Elgy et al. [54] also applied DIC with a resolution of 1024 x 1024 pixels at 20000 FPS for the validation of aluminium honeycomb crush-block measurements of plate deflection from 1 kg PE4 buried charges.

Seo et al. [55] investigated using DIC to take measurements of ground shock induced vibrations in a rock column in an in-situ mine environment. Recordings were made during blasting, at a point approximately 40 m from a 95.8 kg ANFO explosive charge in a hole drilled in limestone rock. They found that DIC measurements correlated well with recordings from a conventional contact vibration meter. However, this technique has not previously been applied to the determination of ground shock loading on a plate or buried structure.

2.1.2. Control of geotechnical properties

Various investigations of loading from shallow buried charges have been carried out over recent decades. Clarke et al. [56, 57, 58] demonstrated that the impulse which impinges upon a target above a soil surface is dependent on moisture content and particle size distribution (PSD), in addition to bulk density as identified in the predictive formula of Westine et al. [59]. It was found that a well-graded soil (i.e. Stanag as specified for landmine-analogue testing by NATO [60]) results in a much wider spread of results than a uniform soil such as Leighton Buzzard 14/25 sand (LB) [58, 61], with the former contributing highly variable impulse through discrete particle strikes due to the large range in particle sizes [62, 63, 64]. Although for a different experiment type, these works give credence to the importance of controlling

geotechnical conditions in blast testing as much as practicable to ensure consistency in results. Thus, it was decided to use LB sand of a consistent moisture content and bulk density (and thus a consistent dry density) for all tests in this series, in order to allow for direct comparison of results with minimal variance due to geotechnical conditions.

2.2. Experimental setup

The experimental setup consisted of a soil container with a deformable base plate positioned on a purpose-built steel frame, suspended over a concrete pit structure, with two Photron SA-Z high speed video (HSV) cameras mounted below (see Figure 2).

The soil container (bin) was made up of a 592 mm internal diameter steel pipe, with a wall thickness of 10 mm, either 0.5 or 1 m in length, containing a specified depth of soil. This pipe was welded to a stiff steel flange to allow a 3 mm circular aluminium (Al-6082-T6) plate to be rigidly clamped at its base, as in Figure 3. The plate was clamped between the pipe flange and an identical separate flange via 16 M20 bolts around the periphery of the plate (see Figures 3 & 14), with an additional four bolts at quarter points used to secure the whole soil bin to a 30 mm thick steel plate, rigidly fixed to the testing frame. Before securing in the clamping apparatus, the aluminium plate was lightly sanded and cleaned, then sprayed with a white primer paint, after which a 0.20" (5.08 mm) speckle pattern was applied using a Correlated Solutions, Inc. roller. This equated to a typical speckle size of seven pixels on the resulting video, ensuring all speckles were above the minimum of three pixels suggested by Reu [65]. It should be noted that the pipe structure represents an approximately rigid boundary on the soil domain, which is likely to have a greater influence (as a result of reflections from the pipe wall etc.) as the charge is moved away from the plate. For this reason, numerical modelling in this study takes account of the finite width of the soil domain.

The soil, Leighton Buzzard 14/25 sand (LB – with PSD as in Figure 4), was prepared to a bulk density of $1.65 \text{ g/cm}^3 \pm 0.01 \text{ g/cm}^3$ for all tests. Moisture content was controlled at a constant $5\% \pm 0.2\%$, with consistency throughout samples achieved by thoroughly mixing sand with the required volume of water before placement in the bin. Moisture content

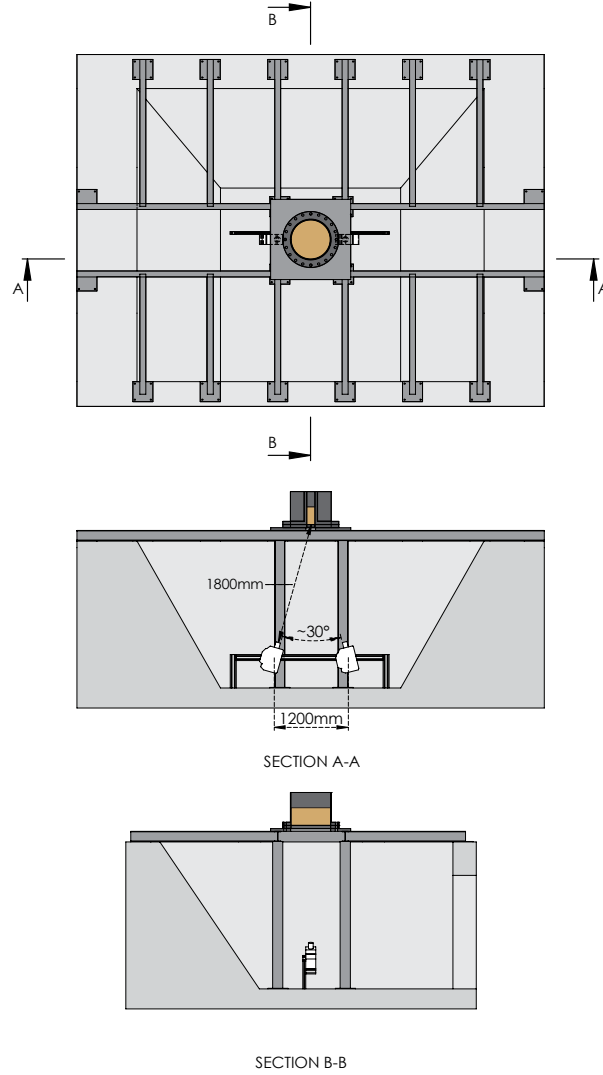


Figure 2: Diagram of the experimental setup (nominal dimensions)

385 measurements were made on small soil samples by recording the mass lost during rapid
 386 drying in a microwave oven. Once the appropriate moisture content was achieved, the soil
 387 was placed in the bin in a number of lifts, with each compacted to the desired bulk density.
 388 During the compaction process, the deformable aluminium plate was supported by a rigid
 389 steel insert with a rubber top pad to avoid deflection. In this paper, moisture content (m/c)
 390 is defined as the proportion of water by mass of the wet soil. In Table 1, ω is also reported
 391 as the mass of water divided by the dry mass of soil for each test.

392 Within the soil, a hand-moulded spherical PE10 charge (nominally 86% PETN with 14%

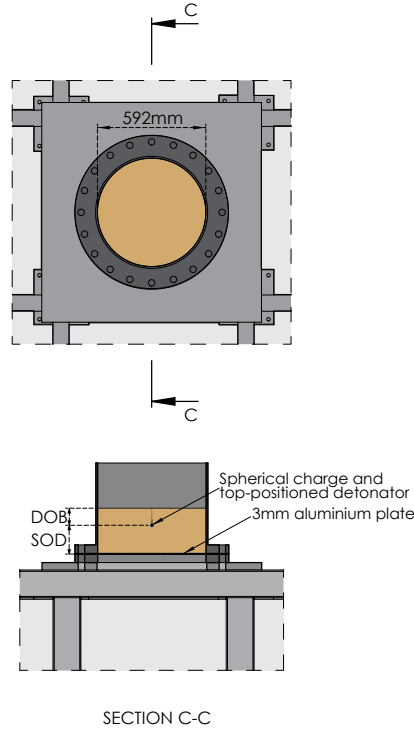


Figure 3: Closer view of the soil bin

binders at a density of 1.5 g/cm^3 , with this composition also known as ‘X-Dough’, ‘Polar Dough’, ‘Seismoplast’, ‘NSP711’ and ‘m/46’) was positioned in an excavated central cavity at a specified centroidal depth of burial from the soil surface (DOB) and stand-off distance from the aluminium plate (SOD). TNT equivalency for PE10 was taken, for simplicity, as a constant 1.22 for this study – this was established in the far-field regime by Farrimond et al. [66], although measuring explosive effectiveness can be seen to be more complex at close distances [67] and when interacting with media such as soil. After charge and detonator placement, the cavity was backfilled with the excavated sand. The charge was top-detonated (using a Euronel 2 non-electric detonator) for all tests to remove the influence of detonator placement on loading. A single-strand enamelled breakwire was embedded within the charge, with the opening of this circuit at the instant of detonation triggering the HSV camera recording.

The two cameras were mounted to a single support frame, isolated from the concrete

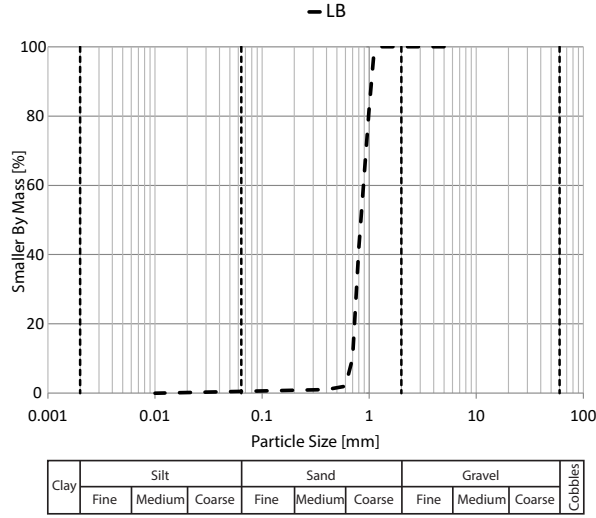


Figure 4: Particle size distribution curve of the LB soil used in testing

406 pit by a layer of sand and separate from the main steel structure to reduce transfer of in-
 407 structure vibrations during testing. Nominally, there was a spacing of 1800 mm from the
 408 front of the camera to the target plate, with each camera mounted approximately 15 degrees
 409 from vertical (thus allowing for a 30 degree stereo-angle between the two cameras, falling
 410 within the range recommended by Reu [68]). Matching Nikon 50 mm lenses were used at an
 411 aperture of $f/4$ and a shutter speed of $3.75 \mu\text{s}$, with additional illumination provided by two
 412 high-intensity GS Vitec MX LED lights. The cameras were placed inside protective boxes,
 413 with a layer of transparent polycarbonate placed in front of the lenses to protect them
 414 from damage. This setup allowed for a field of view of approximately 450 mm x 200 mm
 415 on the plate, from a sensor resolution of 640 x 280 pixels on each camera (corresponding
 416 to approximately 1.4 px/mm) at 100,000 frames per second (for transient DIC capture), as
 417 shown in Figure 5. For residual displacement measurement, full-frame images were captured
 418 of the entire exposed plate area.

419 Before testing, calibration of the camera system was carried out using a target board
 420 from Correlated Solutions, Inc. – wherein approximately 40 images were captured with the
 421 board at varied angles and positions across the capture area. This allowed the software to
 422 recognise the spatial layout of the specific setup (taking account of small changes in camera

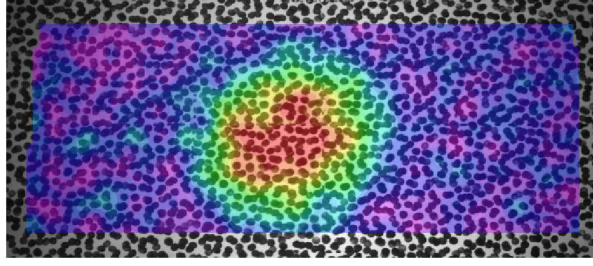


Figure 5: Typical camera field of view (FOV) with DIC data area overlaid

position etc.) by calibrating against a known target.

2.3. Experimental test summary

Nine experiments were conducted, with parameters varied to investigate their effects on loading (Table 1). Tests A – H varied stand-off distance to the plate, measured from the charge centre to the plate surface, for the same 10 g charge mass and same 100 mm centroidal depth of burial. Test I utilised an increased charge mass of 50 g at the same scaled SOD and scaled DOB as test D in order to investigate the validity of cube-root scaling on ground shock effects. Transient DIC data was captured for tests C – I, although only residual displacement was captured in tests A & B due to initial problems with triggering the cameras.

Residual displacement profiles were captured for all tests except test C. TNT equivalent charge mass, and scaled distance as a function of TNTe mass, is reported herein to allow for comparison to other data captured from alternative explosive types.

2.4. Results and Discussion

For each test, ten thousand captured pairs of images taken during the period of plate motion (0.1 seconds of test data) were processed using VIC-3D software [69] with a subset size of 29 pixels (chosen to achieve reduced uncertainty over a large area of the plate for all tests) and a step size of 7 pixels. This resulted in over 2500 data capture locations across the plate, for in excess of 25 million data points per test. Displacement at each point was measured for each frame, with the initial deflected shape (from sand mass under gravity) acting as a zero-reference. From the transient displacement, velocity and acceleration were calculated as first and second derivatives with respect to time. For reference, assessing a

ID	Charge mass [g]	TNTe mass [g _{TNTe}]	SOD (centroidal) [mm]	DOB (centroidal) [mm]	Scaled SOD [m/kg _{TNTe} ^{1/3}]	Scaled DOB [m/kg _{TNTe} ^{1/3}]	Bulk density [g/cm ³]	Moisture content [%]	Data captured	Peak pre-test deflection [mm]
A	10	12.2	50	100	0.217	0.434	1.641	4.85 (5.10)	Residual	4.8
B	10	12.2	50	100	0.217	0.434	1.647	5.11 (5.39)	Residual	4.4
C	10	12.2	50	100	0.217	0.434	1.652	5.12 (5.40)	Transient	–
D	10	12.2	100	100	0.434	0.434	1.649	5.01 (5.27)	Transient & residual	5.0
E	10	12.2	250	100	1.086	0.434	1.649	5.04 (5.31)	Transient & residual	4.9
F	10	12.2	250	100	1.086	0.434	1.645	4.89 (5.14)	Transient & residual	4.0
G	10	12.2	350	100	1.520	0.434	1.647	4.99 (5.25)	Transient & residual	4.6
H	10	12.2	500	100	2.172	0.434	1.645	4.84 (5.09)	Transient & residual	2.7
I	50	61.0	171	171	0.434	0.434	1.649	5.00 (5.26)	Transient & residual	3.8

Table 1: Details of experimental tests undertaken. Moisture content reported as m/c and (ω), i.e. percentage of total soil mass and percentage of dry soil mass. Pre-test deflection determined over a 500 mm circular region.

500 mm circular region using the full-frame images taken before the test, all recorded pre-test differential displacements in the out-of-plane dimension were less than 5 mm, as outlined in Table 1.

In order to account for errors in charge placement and differences in camera alignment between tests, the 2D coordinate system of the data (X and Y in the plane of the undeformed plate) was adjusted to align the origin with the point of maximum displacement of the plate for transient data from all tests. This allows for direct comparison between data from multiple tests.

2.4.1. Signal processing

The data produced by the digital image correlation process has an inherent uncertainty due to system noise, with uncertainty in the matches between the images from each of

the cameras. See Curry et al. [51] for a detailed discussion of noise characterisation in DIC measurements in blast testing. In the present study, this represented a very small proportion of the displacement–time data, evident in a peak standard deviation of transient out-of-plane displacement from all of the tests of 0.07 mm; the standard deviation formed a maximum of 0.6% of peak transient displacement for any test.

However, there was high-frequency noise present in the velocity and acceleration data (due to the differentiation of the noise on the displacement–time data), so low-pass filtering was investigated to address this. A fourth-order Butterworth filter with a cut-off frequency of 5 kHz (a method similar to that used in analysis of ground velocity waveforms from the 2022 Hunga Volcano eruption [70]) was applied to the velocity data on an exploratory basis. The filter in this work operates at a much higher cut-off frequency due to the presence of high-frequency vibrations in local explosive ground shock, as opposed to distant seismic loading; the Donner et al. [70] bandpass filter operated between 0.02 and 0.06 Hz for stations up to 10 000 km away from the source. The 5 kHz cut-off frequency in this work was selected based on a sensitivity analysis on the midpoint velocity data, wherein it was found that for Tests C, D and I (representing scaled SODs up to $0.434 \text{ m/kg}_{\text{TNTe}}^{1/3}$) this resulted in a maximum deviation in peak velocity from the raw data of 3%. For the further SOD tests, a maximum deviation was seen for Test H, with a 22% difference between the peak raw and filtered velocities, of 2.89 m/s and 2.25 m/s respectively. The difference in magnitude results primarily from noise in the raw signal, rather than a reduction in the true magnitude. Unfortunately the sensitivity analysis also showed that all levels of filtering on the velocity data (subsequently used to calculate acceleration) resulted in large reductions in peak acceleration. As such, velocity has been used unfiltered in this work, and acceleration has been calculated from the raw velocity data – under the caveat that at larger SODs it is not possible to ascertain the true value of acceleration due to the large proportion of noise on the velocity signal magnitude at these distances. It is evident that the decision to employ filtering in work of this type is dependent on the magnitudes of signals involved; if concentrating on only large SODs, filtering may be necessary.

Utilising a Fast Fourier Transform (FFT) analysis on the raw midpoint displacement data

484 highlighted a common high-amplitude response frequency between the nearer SOD (up to
 485 $Z = 1.086 \text{ m/kg}_{\text{TNTe}}^{1/3}$) tests of approximately 40 Hz (see Figure 6). A fully clamped (fixed
 486 edge) circular plate with the geometry and properties used herein (seen in Table 2) has a
 487 theoretical first natural frequency of 82 Hz, as calculated by Equation 1, reproduced from
 488 Carrington [71]. On the other hand, a simply supported circular plate has a first natural
 489 frequency of 40 Hz [72].

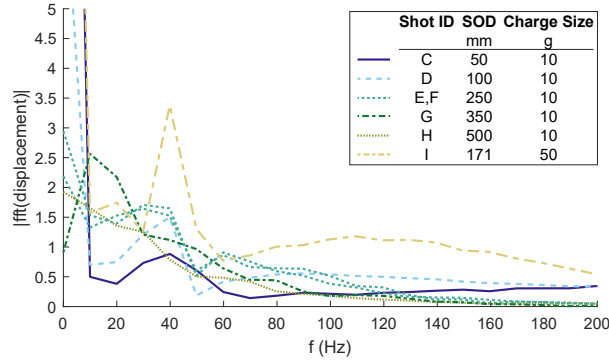


Figure 6: Frequency spectra of midpoint displacement up to 200Hz

Symbol	Plate property	Value
t	Thickness (mm)	3
a	Radius (mm)	308
E	Young's Modulus (GPa)	73
ρ	Density (kg/m ³)	2710
ν	Poisson's Ratio	0.33

Table 2: Plate properties used to calculate natural frequency. Radius taken as pipe internal radius plus distance to clamping surface, set back as seen in Figure 14. Material properties as used for numerical modelling, given in Appendix B.10.

$$n = \frac{tz^2}{4\pi a^2} \sqrt{\frac{E}{3\rho(1-\nu^2)}} \quad (1)$$

490 where n is the frequency of vibration, t is the plate thickness, a is the radius, E is the
 491 Young's Modulus, ρ is the density and ν is the Poisson's ratio. z is given by the roots of

Equation 2 from Carrington [71] for a clamped edge, or Equation 3 from Prescott [72] for a simply supported edge.

$$\frac{J_{n+1}(z)}{J_n(z)} + \frac{I_{n+1}(z)}{I_n(z)} = 0 \quad \frac{J_{n+1}(z)}{J_n(z)} + \frac{I_{n+1}(z)}{I_n(z)} = \frac{2z}{1-\nu}$$

(2) (3)

where n is the number of nodal diameters for plate vibration, and the root number is the number of nodal circles (for the first natural frequency, $n = 0$ and root number = 1). J_n is the Bessel function of the first kind, and I_n is the Modified Bessel function of the first kind.

Thus it is clear that the plate–soil system is not acting as a rigidly clamped, freely oscillating plate when vibrating after the initial loading period. For buried (or soil backed) structures the idealisation of simply supported or rigid boundary conditions fails to capture the complexities which exist in the system. In the current work, the complexities are found in the added soil mass and the compliance in the bolted clamp flanges that act to soften the rigid boundary condition to a damped frequency of approximately 40 Hz. Hence, the boundary conditions must be considered an integral part of the dynamic system. Test G (at $Z = 1.520 \text{ m/kg}_{\text{TNTe}}^{1/3}$) at greater SOD had a lower frequency peak at approximately 10 Hz, likely due to the initial loading velocity falling below that which is required to induce measurable oscillation at the natural frequency of the system. Due to the recording length it was not possible to see a clear response frequency of test H (at $Z = 2.172 \text{ m/kg}_{\text{TNTe}}^{1/3}$), which is likely to fall between 0–10 Hz.

2.4.2. Plate response

Due to the high volume of data available from the DIC results of each test, it is pragmatic to utilise only limited portions to infer the loading impinging upon the plate. In Figure 7 a profile taken along the long axis of the field of view, through the centre of displacement of the plate, is displayed for both displacement and velocity profiles of Test C. As can be seen in Figures 7a and 7c, the plate response is made up of an initial loading period, resulting in a plastic deformation over a few milliseconds, followed by a global elastic vibration of the plate. Figures 7b and 7d show the first 5ms of plate displacement and velocity respectively,

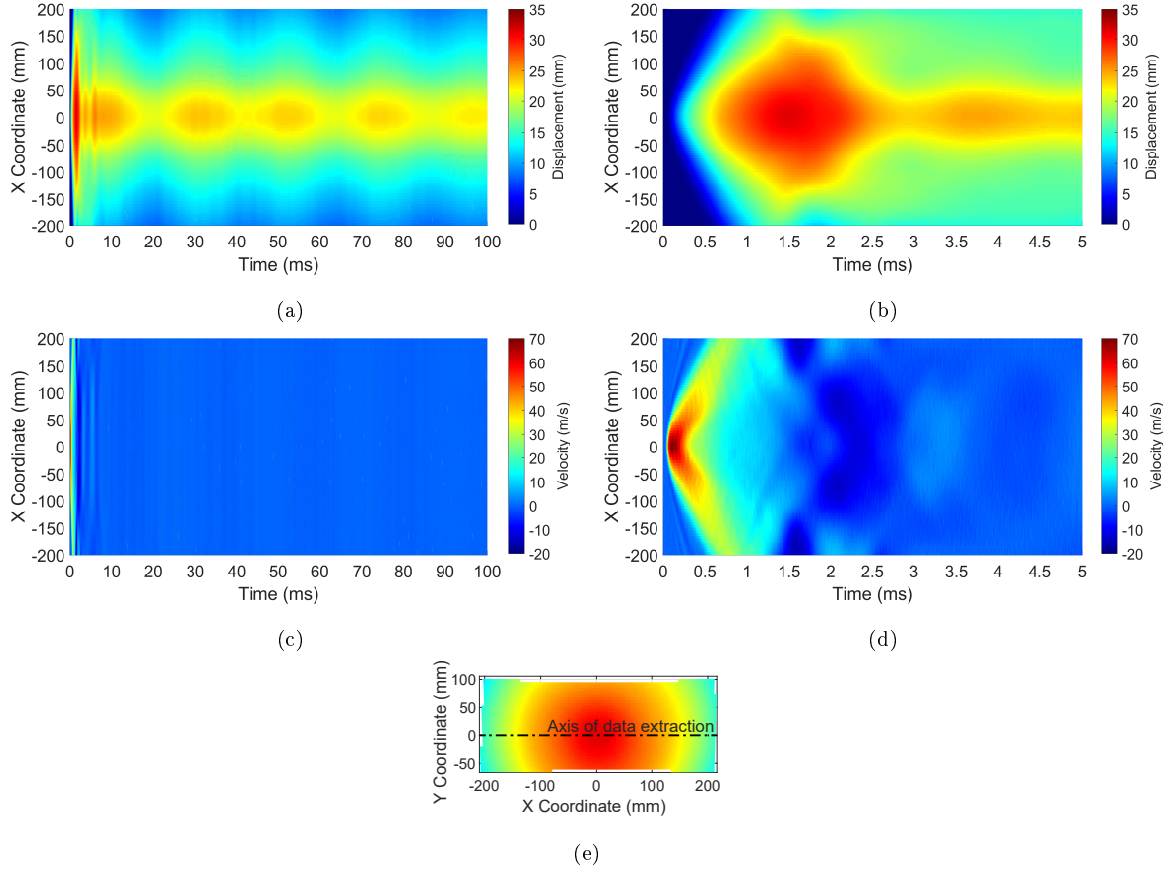


Figure 7: Test C plate transient behaviour extracted from DIC: (a) Displacement for 100ms, (b) Displacement for 5ms, (c) Velocity for 100ms, (d) Velocity for 5ms, (e) Axis of data extraction (overlaid on peak displacement surface)

demonstrating that at this close-in (50 mm) SOD, there is a centralised load applied quickly as a result of the ground shock wave.

2.4.3. Displacement measurements

Extracting the plate behaviour at a single (mid)point (the point of peak transient displacement) enables ease of comparison of transient quantities between tests, but does not demonstrate the distribution of these quantities over the surface of the structure. Figure 8 shows the midpoint transient displacement of the plate, scaled by the cube root of TNT-equivalent charge mass. It is evident that a decreasing SOD leads to an increased peak displacement, with an initial deformation takeup followed by elastic vibrations for all tests.

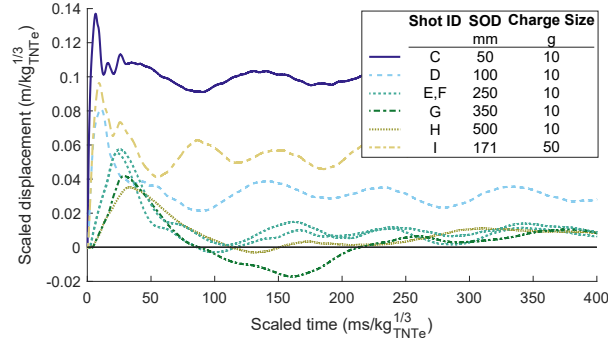


Figure 8: Midpoint scaled transient displacement of plate against scaled time

528 An increased charge mass for the same scaled SOD results in an increased absolute and
 529 scaled displacement, as observed between Test D (10 g PE10 / 100 mm SOD) and Test I
 530 (50 g PE10 / 171 mm SOD). Comparing these two tests with displacement scaled by charge
 531 mass results in a peak scaled displacement approximately 20% greater for the larger charge
 532 size test. This increased displacement is likely due to the more distributed loading from a
 533 charge at a greater SOD resulting from a lower scaled radial distance from the charge centre
 534 to a point away from the plate centre, leading to greater loading applied to the plate as a
 535 whole. This effect is demonstrated in Figure 9, where the radial scaled distance to a location
 536 on the plate from the charge centre, for any horizontal distance from plate centre, is shown
 537 for both charge sizes used. As can be seen, for a larger charge, positions away from the
 538 plate centre are a smaller scaled distance (Z) away than the same point for a smaller charge,
 539 resulting in higher loading at that point. Additionally, a lack of scaling of the plate thickness
 540 for the increased charge size scenario is likely to influence differences in scaled behaviour,
 541 with this effect discussed further in Section 4.

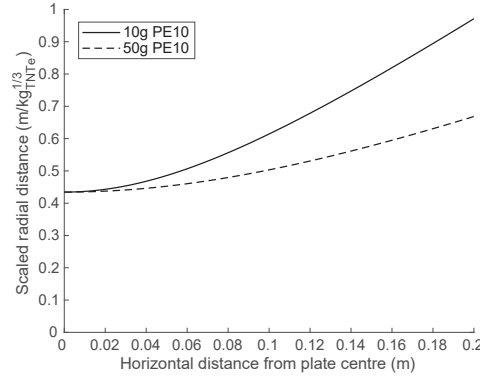


Figure 9: Effect of charge size on scaled radial distance across the plate

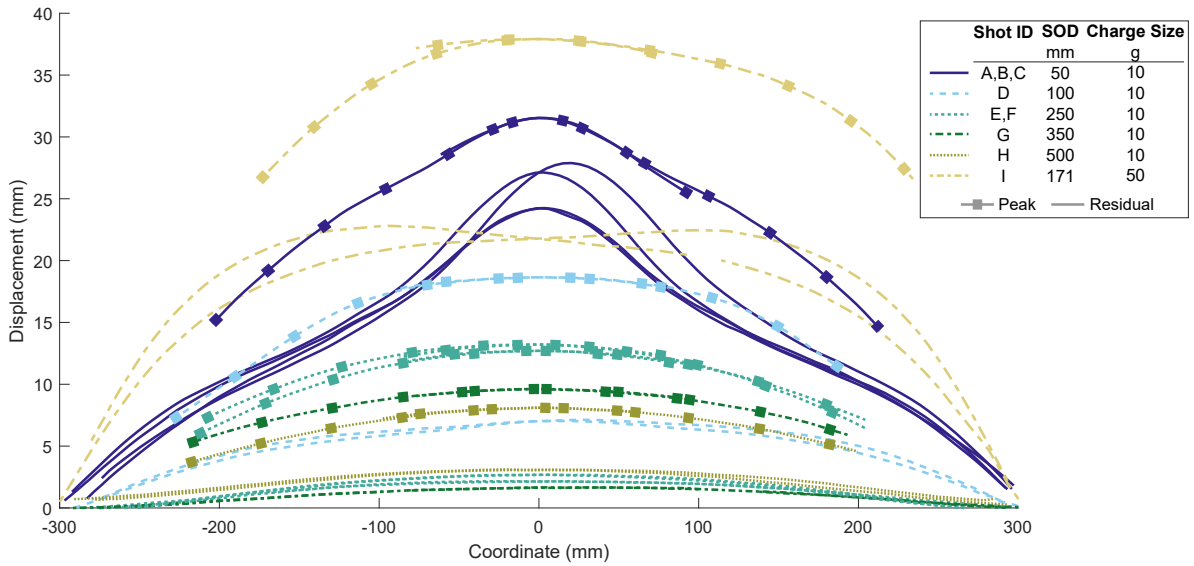


Figure 10: Plate displacement profiles from each test. Peak and residual displacement profiles are shown, for both the X and Y axes.

Absolute plate displacement profiles (along the X and Y axes) at the time of peak displacement, and long-term residual displacement, are shown in Figure 10. The transient profiles have been aligned with the point of peak displacement at the origin in X and Y, however the residual profiles retain their original exported positions. This shows that there is a centralised displacement profile for the 50 mm SOD (Test C), with a more distributed profile for charges at greater SODs. When comparing the peak transient displacement profiles against the residual displacement profiles, it is clear that the centralised loading

of Tests A, B and C contributes more heavily to plastic deformation than the distributed loading of other tests. This is especially clear when comparing Test A, B and C against Test I, where the peak transient displacement of Test I is considerably greater, but the peak residual displacement is lower. Also, this can be seen for the results of tests E, F, G, and H, where the residual displacement is greatest for test H, but the transient displacement is lowest. Thus, it is apparent that peak residual displacement is not necessarily an accurate indication of the transient displacement experienced.

2.4.4. *Velocity measurements*

Plate velocity can be seen to reduce with increased stand-off from the charge to the plate. For the same scaled distance, but an increased charge size, the same peak velocity is recorded, however the duration of the velocity decay is increased, as seen for Test I (50 g PE10 / 171 mm SOD) and Test D (10 g PE10 / 100 mm SOD). When plotted against scaled time (as in Figure 11), the decay period of these two curves become much more alike, showing that velocity-time profiles appear to scale by the cube-root of charge mass. Figure 11 shows the raw velocity traces (without filtering, as used to determine peak magnitudes in subsequent sections) overlaid by traces filtered according to the exploratory method described in Section 2.4.1. It can be seen that at the larger SODs, there is an initial rise in velocity due to the arrival of the incident wave, followed by subsequent increases resulting from reflections from the pipe wall.

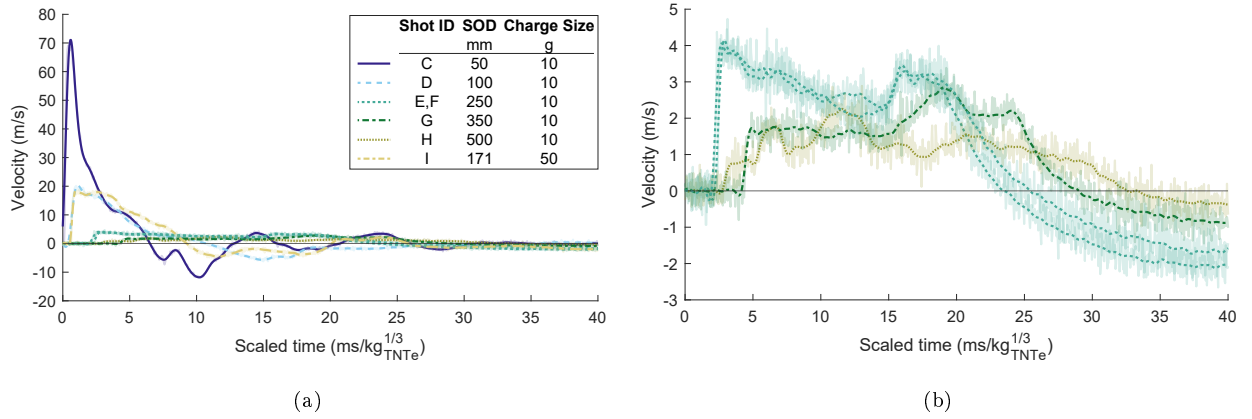


Figure 11: Midpoint velocity of plate against scaled time; (a) Full traces, (b) Zoomed-in view of lower magnitude traces

Figure 12 shows the full-plate peak velocity profiles recorded from each test. It can be seen that although the central peak velocity is approximately the same between Test D and Test I, the peak velocity at positions away from the centre is increased for Test I. This, again, is likely due to the decreased scaled radial distance from the charge to those same horizontal positions when a larger charge is used.

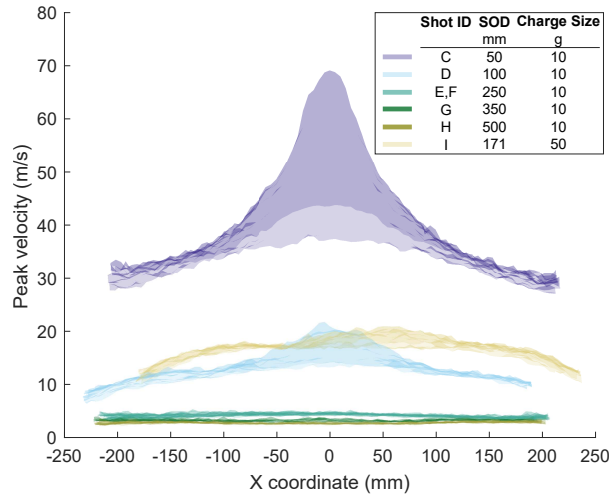


Figure 12: Peak plate velocity profiles from each test (surface plots over the whole DIC capture region viewed side-on)

2.4.5. Ground shock wave speed

Time of arrival, inferred from initial plate motion, for every recorded point on the plate, is plotted against scaled radial distance in Figure 13, wherein a linear trend can be seen representing a constant wave speed through the soil media. This parameter was automatically determined for each point by finding the first time at which the displacement of the point exceeded the baseline level plus a threshold of 1.5 times the maximum noise range in the pre-trigger period.

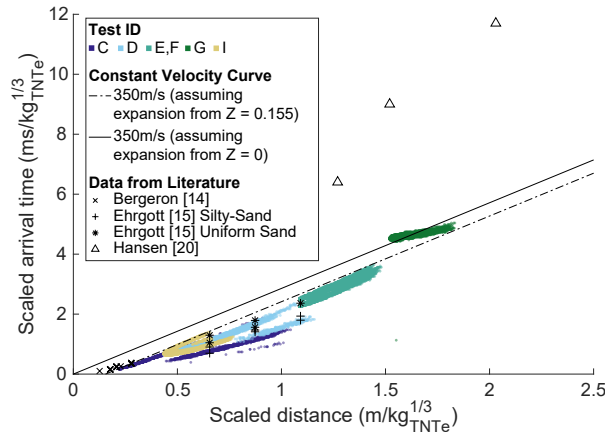


Figure 13: Scaled time of arrival of plate motion at each recorded point against scaled distance

A wave speed of 350 m/s agrees well with most of the data, and this fit is further improved by assuming an instantaneous wave progression to 0.155 m/kg^{1/3} (the typical range of high-pressure and high-velocity hydrodynamic camouflet formation as described by Drake et al. [24]). This wave speed matches the suggested wave speed for a “medium density dry soil” in [24], of density 1700 kg/m³. Data from test H has been omitted due to a triggering error with the camera system (causing the camera recording to begin too late), leading to an incorrect time of arrival. Additionally, data from various tests in the literature is plotted for comparison. It should be noted that the work by Hansen [20] was carried out under centrifuge conditions with very small-scale charges in dry soil, which may lead to the difference in arrival time from that seen in this work.

3. Numerical modelling

In order for the ground shock loading findings from this study to be extended from a simple soil-plate interaction to more complex scenarios, the results have been used for the validation of a numerical model produced in LS-DYNA. This model allows for the study of in-soil behaviour including pressure distribution, for direct comparison between soil in contact with a structure against the predictive formulae presented for free-field behaviour. Previous work by Waddoups et al. [73] explored preliminary ground shock modelling however this was limited in its ability to capture the temporal behaviour of the plate and soil system, particularly for larger SODs. This study rectifies this and presents an improved numerical model which better captures the experimentally measured behaviour.

For this purpose, a 2D axisymmetric Multi-Material Arbitrary Lagrange Eulerian (MMALE) model was used to simulate the fluid-type behaviour of the explosive detonation products, and the loading transfer through the soil. The deformable plate was modelled as a *MAT_SIMPLIFIED_JOHNSON_COOK Lagrangian shell with parameters as found by Chen et al. [74] (Table B.10 in Appendix B). The LB sand was modelled as a *MAT_PSEUDO_TENSOR with *EOS_TABULATED_COMPACTION according to the parameters found by Laine and Sandvik [75] for Sjöbo sand (Tables B.6 and B.7 in Appendix B) at an approximate bulk density of 1.674 g/cm^3 and moisture content of 6.57% (very similar to the nominal 1.65 g/cm^3 density and 5% m/c in this experiment). The PE10 explosive was modelled as a *MAT_HIGH_EXPLOSIVE_BURN with *EOS_JWL according to a calibration by Helte et al. [76], as in Tables B.8 & B.9 in Appendix B. The air was modelled using a *MAT_NULL with *EOS_LINEAR_POLYNOMIAL with the parameters outlined in Tables B.12 & B.13. Fluid-structure interaction was handled with the *ALE_STRUCTURED_FSI keyword, which linked the *ALE_STRUCTURED_MESH domain to the Lagrangian plate structure. Modelling air directly resulted in instabilities during model development, leading to the use of a vacuum in preliminary work instead [73]. Careful selection of ALE and FSI control parameters – including modelling two air multi-material groups for the initial full-domain fill and a separate group for the bottom of the

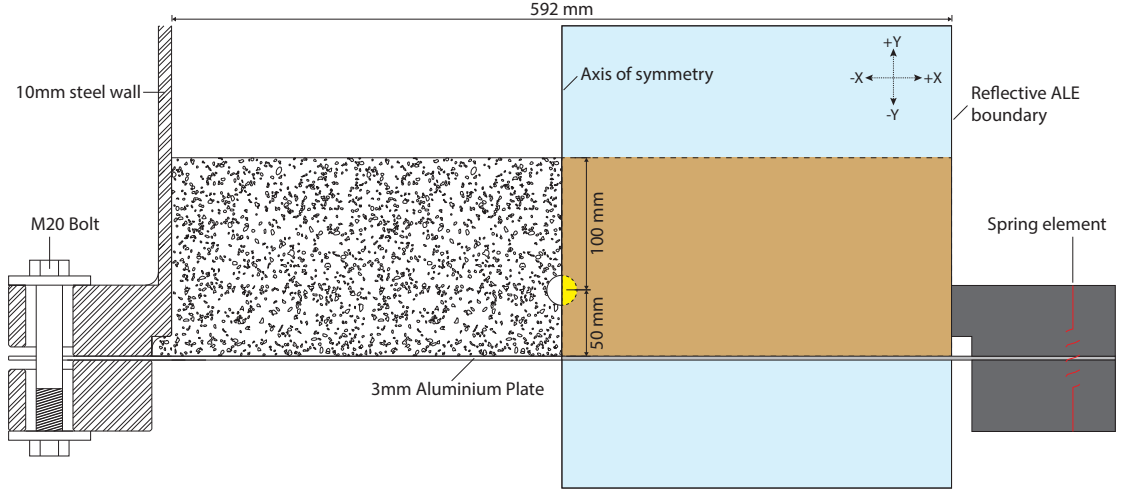


Figure 14: Illustrative model input geometry (right) vs experimental (left) with a 10 g charge at 50 mm SOD – blue: ALE domain filled with air, brown: soil, yellow: PE10 charge, light grey: aluminium plate, dark grey: clamps, red: spring element

domain, with only the region below the plate coupled via FSI – improved model stability when using air. These models exhibited responses more consistent with experimental observations. It was noted during this process that plate behaviour was often impacted by leakage of air through the plate, with tuning required to address this on inspection of the model results.

The modelled plate was clamped similarly to the arrangement in the experimental setup as per Figure 14. The clamps were modelled as *MAT_SIMPLIFIED_JOHNSON_COOK Lagrangian shells with parameters as found by Vedantam et al. [77] (Table B.11 in Appendix B). These were clamped to the plate through the use of a spring element in the place of the bolt, with a spring stiffness of 42.5 kN/mm and an initial offset of 0.5 mm. This was derived by spreading an individual bolt's spring stiffness over the area around the circumference which is bolted (16 M20 bolts) – for a total of 3920 mm² over a ring of area 42725 mm². An individual bolt spring stiffness (k) of 463.5 kN/mm was calculated from a Young's Modulus (E) of 210 GPa, with a tensile area (A) of an M20 bolt of 245 mm² [78], and a length (L) of 111 mm from the top to the bottom of the clamp according to the equation $k = EA/L$. Contact between the steel clamps and aluminium plate was mod-

elled using *CONTACT_2D_AUTOMATIC_SURFACE_TO_SURFACE with static and dynamic friction coefficients of 0.61 [79, 80] and 0.47 [79, 81] respectively. The plate was rotationally fixed at the axis of symmetry. Initially, fixing the node at the inner edge of the clamp was trialled, without modelling the clamp, although this resulted in a higher natural frequency due to a shorter, undamped, effective length of the plate; thus the clamps were modelled in detail. For simplicity, the plate was modelled with zero initial deflection; modelling the limited initial deflection under gravity seen experimentally was deemed unwarranted for this study. However, acceleration due to gravity was applied nodally in the numerical model using the LOAD_BODY_Y keyword.

A fully-reflective condition was used for the -X and +X boundaries of the ALE domain, with undefined (outflow) conditions on the -Y and +Y boundaries (-X being the axis of symmetry, +X the rigid pipe wall, -Y open air below the plate, and +Y the open air above the soil). An element size of 1 mm was used for the ALE domain and for the plate, with a 2 mm element size for the clamps. Dynamic relaxation with a termination time of 0.5 ms was used to remove the dynamic effects of the instantaneous clamp force application, thus eliminating an initial plate vibration which introduced a pressure wave into the soil column base if the relaxation phase was not carried out.

3.1. Model validation – comparison to experimental

In order to validate the performance of the numerical model, each modelled setup has been compared against the results of the corresponding experiment. Due to the high resolution of data from the DIC recordings of the experiments, the model outputs can be compared at many positions across the plate. In Figure 15 data from each of the experimental setups (with varied SOD and charge size) has been plotted as a scaled radial SOD (distance of that point on the plate from the charge centre) against the scaled peak displacement, with the numerical model results overlaid.

Overall, the mean absolute percentage error in peak displacement at the plate centre between each of the experimental recordings and the numerical simulations is 8%. In Figure 15 it can be seen that the peak displacement across the plate from the centre to the plate

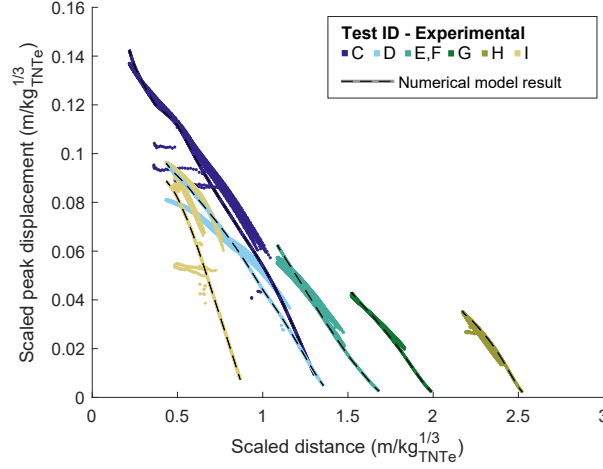


Figure 15: Scaled radial SOD vs scaled peak displacement for each of the tests, with numerical results overlaid. Outliers in experimental data (at lower values of peak displacement) result from points at the FOV edges where the correlation was lost before the time of true peak displacement.

edge decreases more sharply in the simulations (particularly for close-in charges) than in the experimental results, which is particularly evident in Test D, where the model suggests a significantly larger central displacement which then reduces to below the experimental magnitude at greater distance. Test I also exhibits a significant difference in peak displacement between the model and the experiment. Additionally, tests at a greater scaled distance from the charge to the plate generally resulted in better correlation between the model and experimental results for peak displacement.

The time of peak displacement has a greater discrepancy than the magnitude, with a mean absolute percentage error of 16% between the simulation and experimental results at the plate centre; reducing to 5% at a 200 mm radial position (ignoring timings of test H due to the inaccurate triggering of the system). To extract data at radial positions from the experimental data, the mean value was taken of all datapoints which fell within 1 mm of the required radius.

Figures 16, 17 and 18 show the difference in plate response between the experimental tests and numerical simulations for the 50 mm, 250 mm and 500 mm SOD tests with 10 g charges respectively. Shown in Figure 16 is the reduction in displacement at later times for the 50 mm SOD, which was seen to correlate with air leakage through the plate in the

numerical model. As the main transient plate behaviour had completed by this time, it was decided that this leakage need not be addressed herein. Full comparisons for each of the experimental tests with transient DIC data (Tests C–I) against their numerical counterparts can be found in Appendix C.

For the validity of the shock propagation through the soil to be determined, the time of initial plate movement was compared between experimental and numerical results. In the numerical model this was determined by finding the time at which the absolute vertical nodal motion first exceeded 0.01 mm, with this compared against the time of arrival in the experiments as found by the method described in Section 2.4.5. At the central point a mean absolute percentage error of 22% was found (again discounting test H), however, due to the time of arrival being so early, this equates to a mean absolute error of only 0.06 ms (equivalent to six frames from the HSV footage). The modelled propagation of the pressure wave through the soil in the pipe is displayed in Figure 19, which shows the 500 mm SOD, 10 g charge model. Additionally, Figure 20 shows the same pressure distribution, but with a minimum displayed pressure of 100 kPa and a maximum of 104 kPa, therefore demonstrating the limited magnitude of ground-shock-induced air shock at this depth of burial. It can be seen that depending on the SOD from the charge to the plate, the plate response is resultant from interaction with primary ground shock waves and reflections from the domain boundaries (i.e. the pipe walls and soil surface).

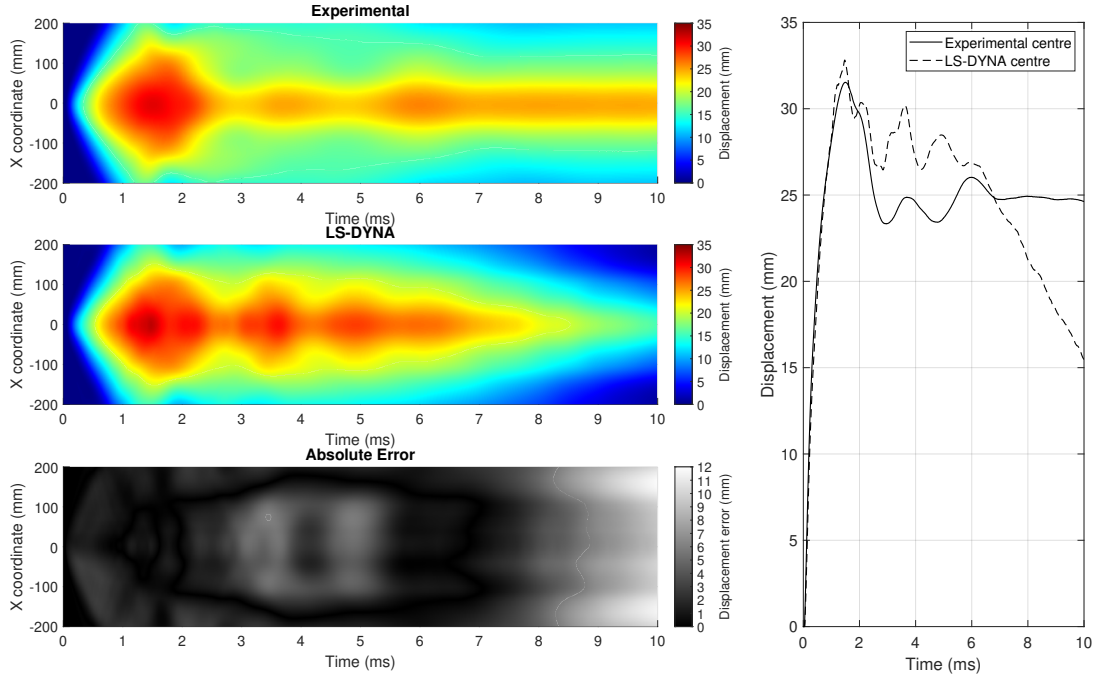


Figure 16: 50 mm SOD experimental (Test C) displacement–time history and the corresponding numerical modelling output (plate width and central point)

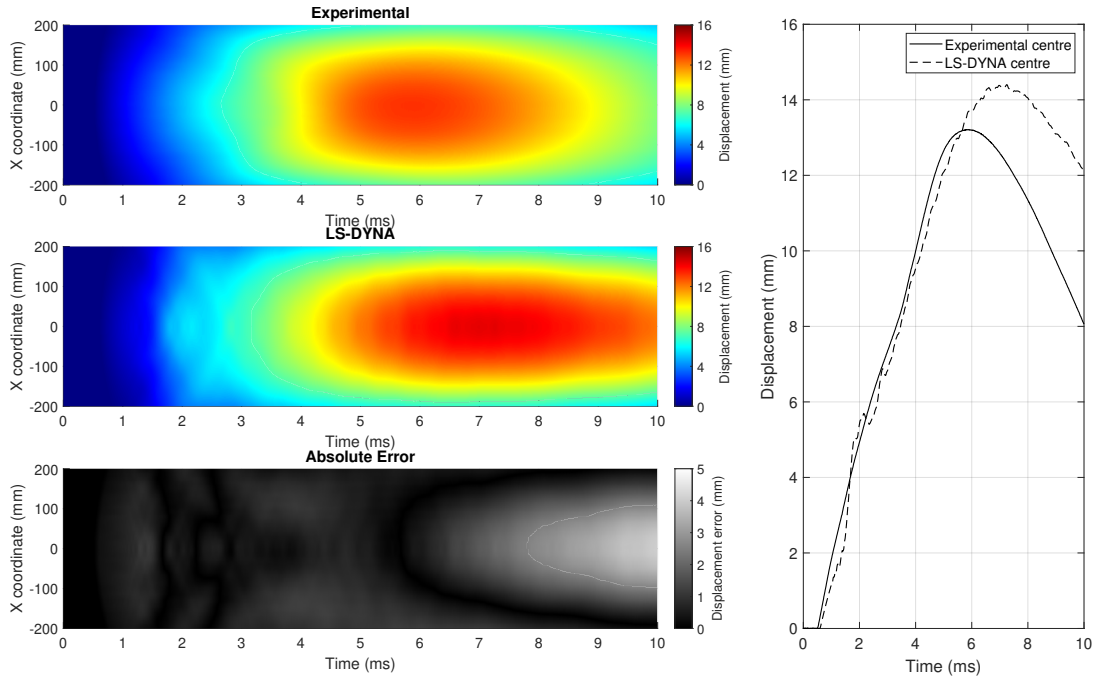


Figure 17: 250 mm SOD experimental (Test F) displacement–time history and the corresponding numerical modelling output

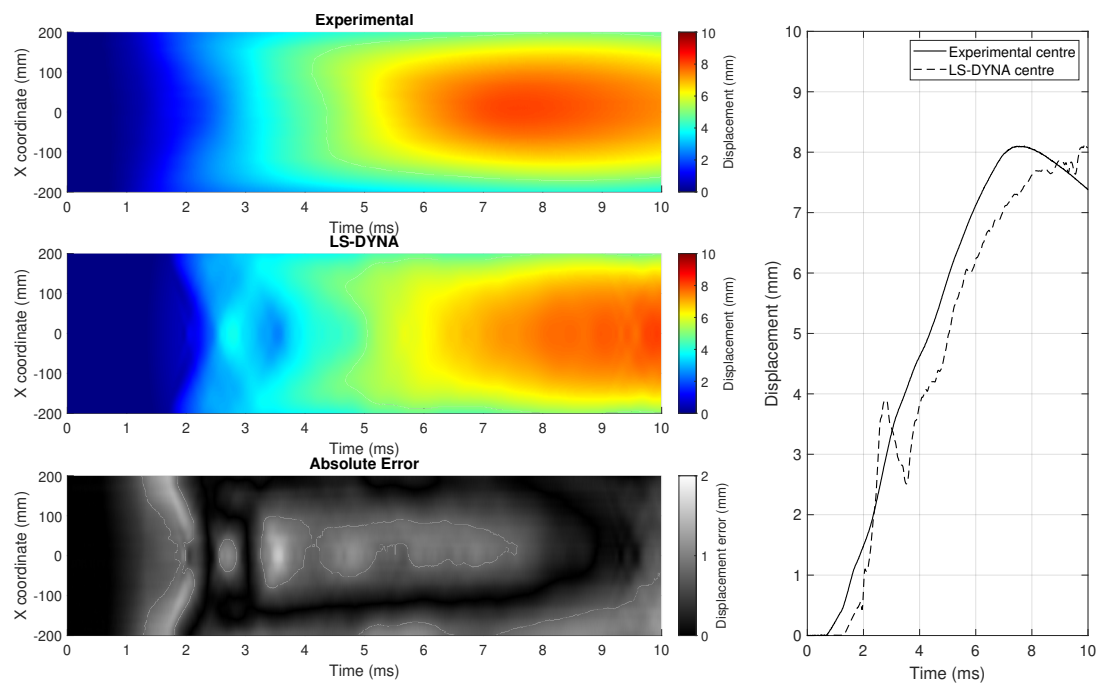


Figure 18: 500 mm SOD experimental (Test H) displacement–time history and the corresponding numerical modelling output

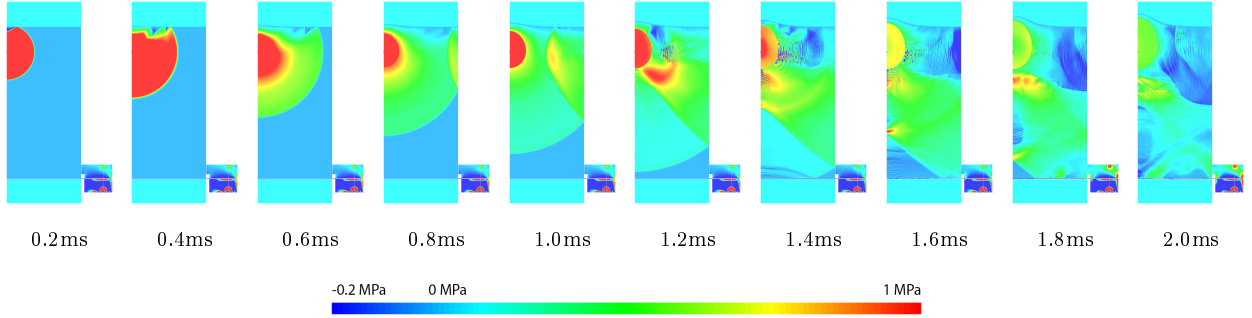


Figure 19: Axisymmetric pressure distribution within the pipe, taken from the 500 mm SOD, 10 g PE10 numerical model. Left edge is the pipe central axis of symmetry, right edge is the pipe wall, with the clamp restraining the edge of the aluminium plate.

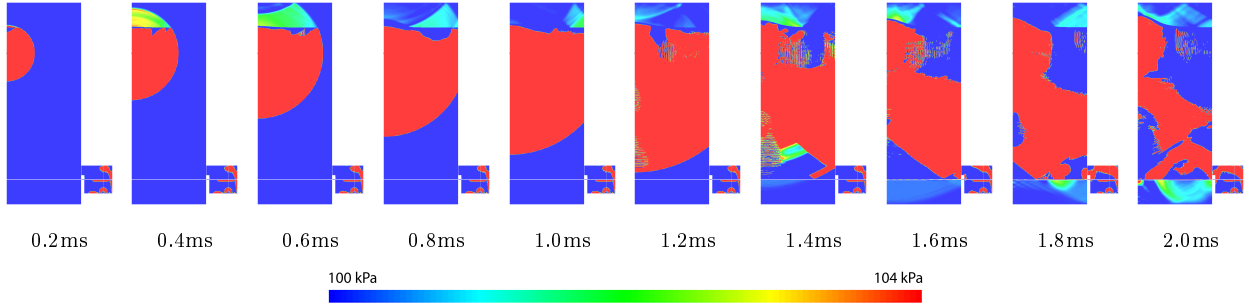


Figure 20: Same as Figure 19, but with reduced contour range to show ground-shock-induced air shock.

3.2. Numerical in-soil outputs

Figure 21a shows the temporal and spatial distribution of pressure with height from the plate in the 500 mm SOD, 10 g PE10 case, along the central vertical axis of the soil. A symmetrical distribution about the charge can be seen until the pressure wave reaches the soil surface, at which point the overburden is ejected forming a crater. As the overburden remains an intact bubble over this timescale, the pressure due to the expanding detonation products is contained.

Downwards, the pressure can be seen to be transmitted as two distinct waves towards the plate. The first, as described previously, consists of the primary shock, arriving at the plate at approximately 1.3 ms. The second wave, a reflection from the pipe wall, arrives at approximately 1.9 ms. This reflected wave is also evident in the particle velocity distribution

in Figure 21b, wherein it can be seen that the soil moves at a high downward velocity up to approximately 250 mm from the charge with the primary shock wave, with this then diminishing at greater SOD. The higher vertical velocity is then carried by the more distributed loading resulting from the reflected wave.

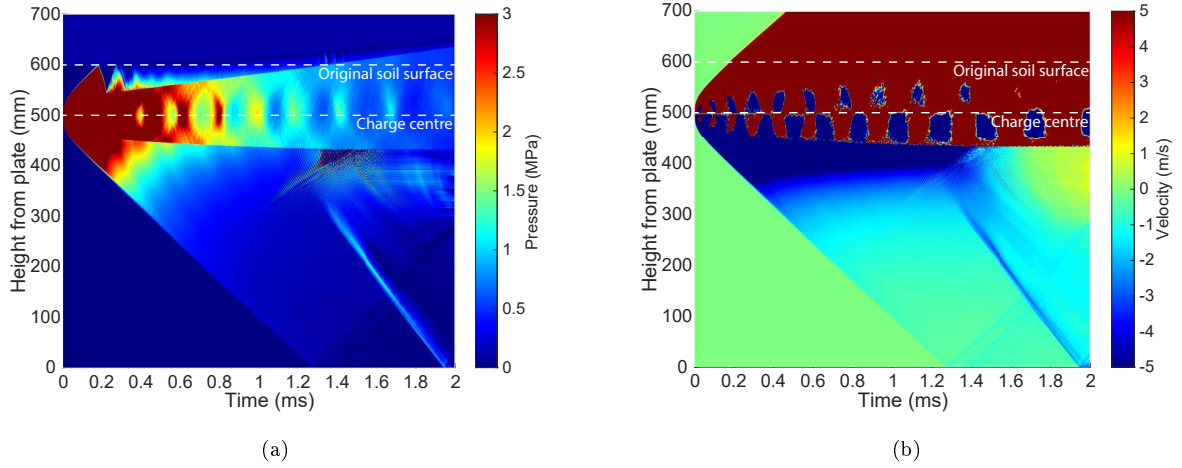


Figure 21: Distributions of quantities taken along the pipe central vertical axis for a 500 mm SOD model: a) Pressure distribution in time for 2ms, with the pressure representation capped at 3 MPa; b) In-soil velocity distributions in time for 2ms, with the velocity representation capped at ± 5 m/s

For each of the modelled SODs, a 0.1 ms window of the modelled pressure-time history at the arrival of the primary shock at the plate has been plotted in Figure 22. It can be seen that the pressure magnitude decreases with increasing SOD, and the profile of the pressure-time history changes from a discrete initial peak to a staged, more prolonged loading period as the plate moves further from the charge. The 100 mm, 10 g PE10 and the 171 mm, 50 g PE10 models display very similar pressure-time histories, showing that the magnitude of pressure can be seen to remain approximately constant at the same scaled distance, as expected according to the cube-root scaling theory [8]. However, the time duration of the pressure loading does not appear to scale, suggesting that this is a structure-dependent quantity, rather than purely load-dependent.

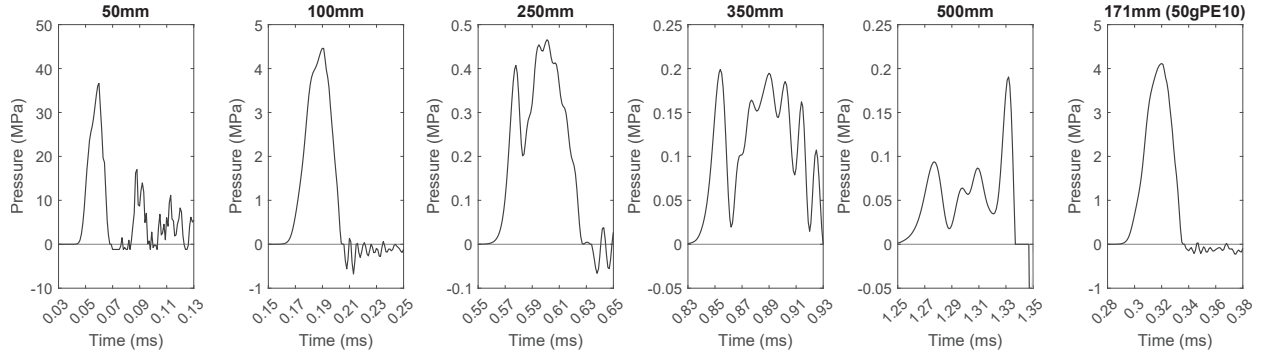


Figure 22: Pressure-time histories, from a position 1 element into the sand above the plate at the pipe centre, from each numerical model

4. Comparison to literature

The existing literature contains a large array of predictive formulae and experimental results from a range of charge scales and experimental setups, as described in Section 1.2, with the equations outlined in Appendix A. In order to establish the validity of these predictors, the in-soil primary wave (i.e the initial peak magnitude, rather than the overall peak which could be influenced by reflections from the pipe wall) vertical velocity and vertical acceleration (scaled by the cube root of TNT-equivalent charge mass) were extracted from each point along the central vertical axis of the 500 mm SOD, 10 g PE10 numerical model, with the curves from the literature overlaid, as shown in Figure 23. For reference, the charge radius extends to a scaled distance of approximately $0.05 \text{ m/kg}_{\text{TNTe}}^{1/3}$ from the charge centre – data from within this radius has been excluded from Figure 23.

Additionally, the validity of these curves for the prediction of structural motion resulting from ground shock has been explored against the measured plate movement recorded in the experimental testing, also shown in Figure 23. Experimental data of others (gathered from buried velocity sensors and accelerometers) has been included for comparison, scaled up to a fully coupled equivalent value where relevant by each scenario’s coupling factor (as derived from the work of Drake et al. [24]), then re-scaled by this study’s assumed coupling factor of 0.95. The plots display initial peak midpoint results from the plate rear surfaces (so as to capture the incident loading behaviour rather than the effect of reflections from the pipe

wall), displayed as scatter plots of scaled quantities against scaled distance, over the full range of the experimental series.

For the predictor curves, where relevant, the coupling between the explosive and soil has been plotted as a band of predicted quantities. A coupling factor of 0.95 for this study was attained from Figure 1 (for scaled DOB of $0.434 \text{ m/kg}_{\text{TNTe}}^{1/3}$). This coupling factor may be overly conservative, as Guan et al. [21] showed that for their testing, a DOB of $0.4 \text{ m/kg}_{\text{TNTe}}^{1/3}$ resulted in a best-fit coupling factor of 0.35, with full coupling achieved at a DOB between $0.4 - 0.8 \text{ m/kg}_{\text{TNTe}}^{1/3}$. As such, a band has been plotted between these two values of coupling factor. For curves without banding, the original source did not suggest a coupling factor relationship as they assumed full coupling; therefore the raw curve has been plotted herein. Also, $\rho = 1.65 \text{ g/cm}^3$ and $c / c_f = 350 \text{ m/s}$ (as found in Section 2.4.5) for all curves; $n = 2.5$ (taken from Table 1 of [7]) for the curves of Drake and Little [7] and Leong et al. [3]; $n = 2.3$ and $c_0 = 300 \text{ m/s}$ for the Drake et al. [24] curves (both taken from Table 1 of [24]). The TNT explosive energy was taken as 4.56 MJ/kg from Table 5-2 of [82]. The same value of c / c_f was used for all prediction curves as this represents a typical sound speed for this medium.

4.1. Acceleration

The formula of Drake and Little [7] (Eqn. A.4) suggests that in-soil acceleration should scale with the cube root of charge mass, consistent with the principle that it is controlled by the rise time to peak velocity, and as such should scale with the scaling of time. Drake et al. [24] (Eqn. A.5) makes use of a rise time term which depends on actual distance, so scales with the scaling of distance and time. However in this work it was noted that plate acceleration appears to be constant for a particular scaled SOD (similarly to the curve presented by Hansen [20], Eqn. A.6), resulting in different scaled accelerations for Tests D and I, at the same scaled SOD but with different charge masses, as seen in Figure 23a. For a plate subjected to an explosion in water (similar in principle to this scenario, but discounting compaction effects etc.), Taylor [83] described that “the motion of the plate is determined by its mass per unit area, m , by the pressure, p , and by external constraints such

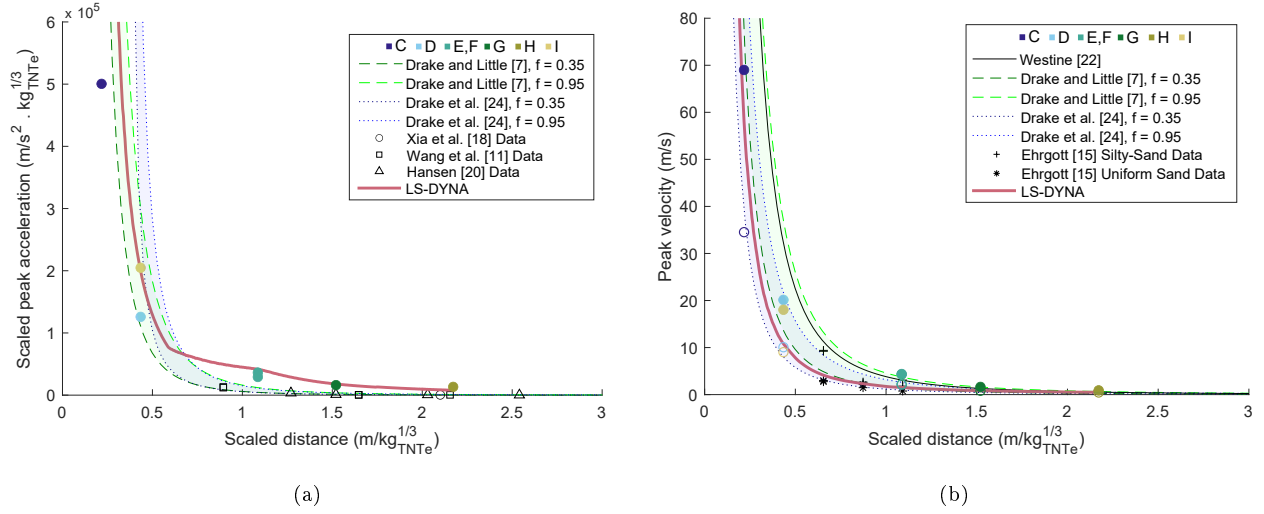


Figure 23: (a) Scaled SOD vs scaled initial peak acceleration and (b) Initial peak velocity from the experimental DIC and numerical modelling results, with historic prediction curves plotted alongside data from the literature. For velocity, filled circles represent plate measurements, with unfilled circles inferred in-soil quantities.

as the supporting framework". At the same scaled distance the pressure to which the plate is subjected should not change, so if plate mass per unit area is not changed in accordance with the scaling of the scenario (i.e. by scaling the thickness, which was not done in this work), the same acceleration should be experienced by the plate. Therefore, the measured plate acceleration is likely to be a feature of the experimental setup, and cannot be used to directly infer in-soil acceleration, which is expected to scale with charge mass.

As such, it is evident that the DIC-determined plate acceleration (initial peak values of which are plotted in Figure 23a) is an inadequate indicator of the in-soil acceleration; the material and form of the structure influences the transfer of ground shock acceleration to it. This is broadly in agreement with the findings of Wang et al. [11] but extends the range of data to significantly more near-field scenarios than this earlier study. Further work is required to accurately ascertain the relationship between in-soil and structural accelerations via the DIC technique, with real-structure materials required to capture the true transfer mechanisms at play.

4.2. Velocity

Under elastic conditions, it is expected that the initial peak plate velocity will be approximately twice the incident wave velocity (as the plate is effectively at a free surface, i.e. the acoustic impedance of the air on the opposite face of the plate is negligible compared to that of the soil), following the “so-called velocity doubling rule in an unloading wave” [84]. In order to establish an approximation of the incident wave properties, assuming that the wave is of sufficiently large wavelength, and in an elastic regime, the plate velocity can be used to infer the incident in-soil velocity, shown by the unfilled circles in Figure 23b. These represent an indicative lower bound on the incident wave velocity, as the decaying pressure of the wave after the peak (as opposed to a constant square wave) should result in the plate velocity not fully reaching the simplified theoretical value. Also, it should be noted that in soils under shock loading, Braithwaite et al. [85] explained that this assumption breaks down due to “irreversible compaction in a granular system”. It was observed in the present study, by inspection of the numerical model, that the plate initially moves with approximately the incident velocity in the soil at the nearest stand-off distance, which is hypothesised to result from the plastic deformation of the soil in this region, and the resulting inelastic conditions, in agreement with the findings of Braithwaite et al. [85]. Additionally, as high frequency components of the motion are expected to be attenuated by the plate’s response behaviour, the greater proportion of higher frequency components of the wave anticipated at reduced distances [86, 87, 88, 89] may also contribute to near-field reductions. The numerically modelled initial plate velocity increased to around 1.4 times the in-soil incident velocity at further SODs (with the values subject to variability as a result of sample time fidelity, in this case 0.01 ms). This is in keeping with the findings of Laine et al. [31], wherein it was found via numerical modelling that a buried compressible piston with a low stiffness and mass allowed for normalised particle velocity to increase by up to around 70% as it flowed into the piston. The intricacies of the complex phenomena associated with the soil-structure interaction during transmission of ground shock from the soil into the plate, and the effects of this on the plate motion, will be the subject of further study by the authors.

In Figure 23b it can be seen that a somewhat higher velocity was recorded experimentally

at large SODs than the curves predict (although these are still of low magnitude). This effect is attributed to the larger significance of the noise on the signal at these greater SODs. The two tests conducted at the same scaled SOD to the plate (D and I), but with varied charge sizes, resulted in approximately the same peak velocity, thus validating the use of scaling SOD by the cube-root of charge mass – discussed also in Section 2.4.4.

4.3. Pressure

Having confirmed the validity of the numerical model, it was used to explore the incident peak pressure (determined as the magnitude of the initial peak in the pressure-time history) within the soil body, as shown in Figure 24.

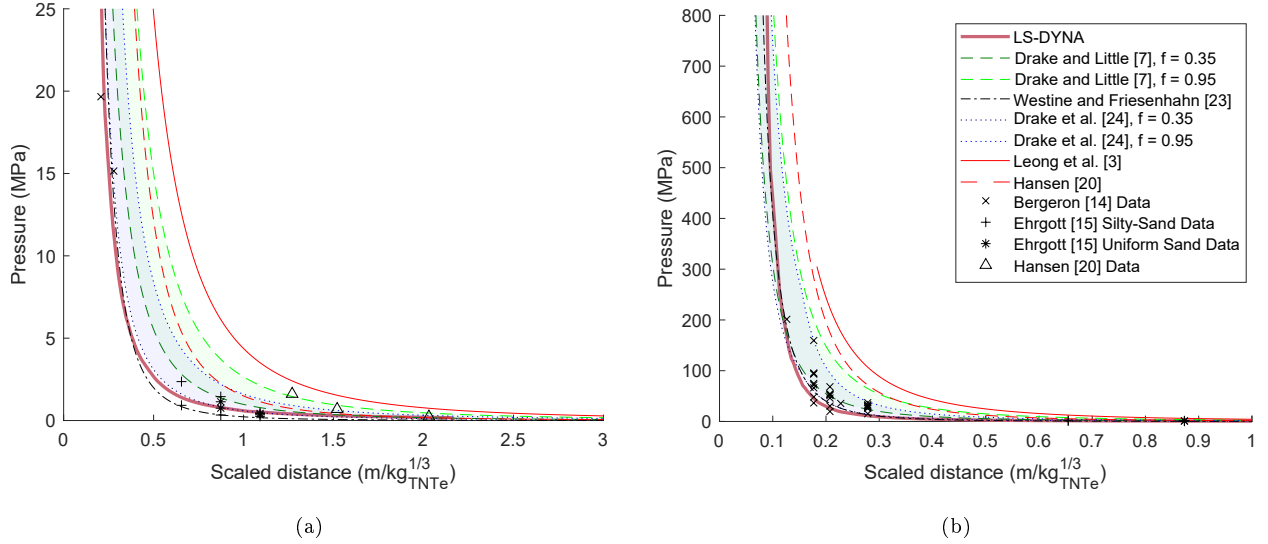


Figure 24: Initial peak pressure extracted from the LS-DYNA model, overlaid by historic prediction curves and literature data: (a) Over the full investigated scaled distance range, with reduced maximum plotted pressure for ease of comprehension, (b) Over a reduced scaled distance with increased maximum pressure plotted

4.4. Observations

As can be seen in Figures 23 & 24, the Drake and Little [7] curves (as presented within CONWEP and TM5-855-1 [6], and often taken as the default for structural modelling), are either accurate or conservative across each of the parameters when the near-unity coupling factor is employed, for most scaled distances. For velocity, in Figure 23b, the Drake et al.

[24] curve fits reasonably well, offering a more conservative prediction when the coupling factor is set to 0.95. The Drake and Little [7] curves fit the velocity data reasonably well at the lower coupling factor, but overestimate considerably at low SODs when $f = 0.95$.

However, the Drake and Little [7] curve (with f between 0.35 and 0.95) displays the best representation of scaled acceleration (which is often the main parameter used in the design of structures), particularly at small stand-off distances, but the modelled in-soil and experimentally measured plate accelerations deviate from the predictors at greater distances. The Drake et al. [24] acceleration curve was seen to represent the modelled soil acceleration poorly over a scaled distance up to around $0.4 \text{ m/kg}_{\text{TNTe}}^{1/3}$ (extremely close to the charge), where it predicts far higher values and appears to asymptote with these inputs at a scaled distance of approximately $0.37 \text{ m/kg}_{\text{TNTe}}^{1/3}$, although it behaves similarly to the other curve at greater scaled distances (as in Figure 23a). The Hansen [20] acceleration curve (Eqn. A.6) results in a value of peak (unscaled) acceleration which does not change for varying charge mass at the same scaled distance, so has been omitted from this plot of scaled acceleration. The Drake et al. [24] curve (Eqn. A.5) makes use of a scaled rise-time (t_r) in this implementation by inputting scaled distance, allowing the use of this curve in scaled-space.

The pressure attenuation in Figure 24 is better represented by the Westine and Friesenhahn [23] curve (assuming full coupling), although this does underestimate at larger SODs. The fit to the Drake and Little [7] and Drake et al. [24] curves was less strong, with a sharper attenuation at lower SOD seen in the model results (particularly clear in Figure 24b), leading to the best-fit coupling factor for these curves varying across the range.

These findings lend greater credence to the notion that the depth of burial used in this study results in a near-fully-coupled ground shock scenario, as the best fits are often to curves with no allowance for reduction in coupling, or when a near-unity coupling factor is used. However, it is clear that a constant value of coupling factor can only be determined if the in-soil effects are measured across a large range, in order to determine the efficacy of the curve fits or the reduction in measured quantities directly.

5. Conclusion

A new experimental technique has been developed, which, via the quantification of plate displacement as a result of ground shock loading from explosive charges, enabled detailed validation of numerical modelling of ground shock propagation. Digital image correlation from stereo high-speed video cameras allowed for high spatial and temporal resolution of data capture, from which plate displacement, velocity and acceleration could be derived, as well as ground shock wave speed from the time of arrival of plate motion.

Numerical modelling correlates well with the experimental results across the tested stand-off distances, although is prone to instabilities due to leakage across the thin plate. It is evident that care must be taken when modelling soil-structure interactions under ground shock loading in order to notice and correct non-physical, unstable behaviours.

Peak plate velocity derived from the DIC experiments was seen to be a useful parameter for the determination of ground shock loading, as the plate properties in this study appear to have limited influence on this, whereas the determined plate acceleration is expected to vary based on the mass per unit area of the plate. Existing predictive equations from the literature have been compared against results extracted from the numerical model, as well as from the experiments, with varying degrees of correlation. It was found that the Drake and Little [7] curves (as presented within CONWEP and TM5-855-1 [6]), are generally either accurate or conservative for in-soil quantities across each of the parameters investigated, with a strong sensitivity to the coupling of the explosion with the ground, although others may be more appropriate depending on the parameter of interest.

This study is believed to be the first time that such a detailed assessment of the mechanisms of ground shock propagation and transfer to buried structures has been possible, giving enhanced confidence in the ability of numerical models to predict the effect of ground shock loading on buried infrastructure, at least in the more damaging near-field scenarios.

Author Contributions

Ross Waddoups: Data curation, Formal analysis, Investigation, Methodology, Visualisation, Writing – original draft, **Sam Clarke:** Conceptualisation, Funding acquisition, Project administration, Resources, Supervision, Writing – review and editing, **Richard Curry:** Conceptualisation, Methodology, Supervision, Writing – review and editing, **Adam Dennis:** Data curation, Investigation, Writing – review and editing, **Andy Tyas:** Conceptualisation, Funding acquisition, Supervision, Writing – review and editing, **Dain Farri-mond:** Investigation, Visualisation, Writing – review and editing, **Tommy Lodge:** Project administration, Resources, Supervision, Writing – review and editing, **Andrew Barr:** Visualisation, Writing – review and editing, **Jay Karlsen:** Formal analysis, Writing – review and editing, **Joshua Mullins:** Funding acquisition, Investigation, Resources, Writing – review and editing, **Joseph Cordell:** Funding acquisition, Investigation, Resources, Writing – review and editing

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References

- [1] S. Cooke, Unexploded ordnance, *Tunnels & Tunneling International* (2015) 43–46.
- [2] K. Stone, A. Murray, S. Cooke, J. Foran, L. Gooderham, Unexploded ordnance (UXO). A guide for the construction industry, CIRIA, 2009.
- [3] E. C. Leong, S. Anand, H. K. Cheong, C. H. Lim, Re-examination of peak stress and scaled distance due to ground shock, *International Journal of Impact Engineering* 34 (2007) 1487–1499. doi:10.1016/j.ijimpeng.2006.10.009.
- [4] M. Grujicic, B. Pandurangan, A. Hariharan, Comparative discrete-particle versus continuum-based computational investigation of soil response to impulse loading, *Journal of Materials Engineering and Performance* 20 (2011) 1520–1535. doi:10.1007/s11665-011-9844-0.
- [5] R. D. Oldham, On the propagation of earthquake motion to great distances, *Philosophical Transactions of the Royal Society A* 194 (1900) 135–174. doi:10.1098/rsta.1900.0015.
- [6] US Army Engineers Waterways Experimental Station, TM5-855-1 Fundamentals of protective design for conventional weapons, Technical Report, US Army, Navy and Air Force, US Government Printing Office, Washington, DC, 1986.
- [7] J. L. Drake, C. D. Little, Ground Shock From Penetrating Conventional Weapons - Report No. ADP001706, Technical Report, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, 1983. From ‘The Interaction of Non-Nuclear Munitions with Structures: Symposium Proceedings Held at U.S. Air Force Academy, Colorado on May 10-13, 1983. Part 1,’ AD-A132 115, p1-6.
- [8] C. W. Lampson, Final report on effects of underground explosions - NDRC Report No. A-479, Technical Report, National Defense Research Committee of the Office of Scientific Research and Development, Washington, D.C., USA, 1946.
- [9] L. F. Ingram, Instrumentation for Earth Stresses and Motions Produced by Explosions - Miscellaneous Paper No. I-885, Technical Report, U.S. Army Engineer Waterways Experiment Station Corps of Engineers, Vicksburg, Mississippi, 1967.
- [10] P. van Dongen, J. Weerheijm, Interaction of ground shock with soil pressure transducers, in: P. Bulson (Ed.), *Proceedings of Structures Under Shock and Impact II*, Portsmouth, UK, 1992, pp. 625–635.
- [11] X. Wang, Y. Li, H. Zhou, H. Dai, L. B. Jayasinghe, Accurate measurement of ground shock with cellular solid, *International Journal of Impact Engineering* 145 (2020). doi:10.1016/j.ijimpeng.2020.103675.
- [12] L. F. Ingram, Ground Motions from High-Explosive Experiments - Miscellaneous Paper No. N-72-10, Technical Report, U.S. Army Engineer Waterways Experiment Station Weapons Effects Laboratory, Vicksburg, Mississippi, 1972.
- [13] J. K. Ingram, J. L. Drake, L. F. Ingram, Influence of Burst Position on Airblast, Ground Shock and Cratering in Sandstone - Miscellaneous Paper No. N-75-3, Technical Report, U.S. Army Engineer

- Waterways Experiment Station Weapons Effects Laboratory, Vicksburg, Mississippi, 1975.
- [14] D. Bergeron, R. Walker, C. Coffey, Detonation of 100-gram Anti-personnel mine Surrogate Charges in Sand - Suffield Report No. 668, Technical Report, Defence Research Establishment Suffield, 1998.
- [15] J. Q. Ehr Gott, Tactical Wheeled Vehicle Survivability: Results of Experiments to Quantify Aboveground Impulse - Report No. ERDC/GSL TR-10-7, Technical Report, US Army Corps of Engineers Engineer Research and Development Center, 2010.
- [16] H. J. Melosh, Shock viscosity and rise time of explosion waves in geologic media, *Journal of Applied Physics* 94 (2003) 4320–4325. doi:10.1063/1.1602566.
- [17] C. Wu, Y. Lu, H. Hao, W. K. Lim, Y. Zhou, C. C. Seah, Characterisation of underground blast-induced ground motions from large-scale field tests, *Shock Waves* 13 (2003) 237–252. doi:10.1007/s00193-003-0212-3.
- [18] C. Xia, L. Chen, R. Xu, M. Cao, D. Chen, Q. Fang, Experimental and Numerical Studies on Ground Shock Generated by Large Equivalent Surface Explosions, *Applied Sciences* 12 (2022). doi:10.3390/app12167987.
- [19] W. A. Charlie, N. A. Dowden, E. J. Villano, G. E. Veyera, D. O. Doebling, Blast-induced stress wave propagation and attenuation: Centrifuge model versus prototype tests, *Geotechnical Testing Journal* 28 (2005) 207–216.
- [20] C. B. Hansen, Buried Explosive-Induced Blast Characterization By Geotechnical Centrifuge Modeling, Ph.D. thesis, University of Colorado, 2016.
- [21] L. Guan, J. Zhang, J. Li, Y. Ding, Y. Wang, Q. Lu, Ground deformation and blast wave propagation in dry sand subjected to buried explosion: a centrifuge modelling study, *International Journal of Impact Engineering* (2024) 104935. doi:10.1016/j.ijimpeng.2024.104935.
- [22] P. S. Westine, Ground Shock From the Detonation of Buried Explosives, *Journal of Terramechanics* 15 (1978) 69–79.
- [23] P. S. Westine, G. J. Friesenhahn, Free-Field Ground Shock Pressures From Buried Detonations In Saturated and Unsaturated Soils - Report No. ADP001708, Technical Report, Southwest Research Institute, San Antonio, Texas, 1983.
- [24] J. L. Drake, E. B. Smith, S. E. Blouin, Enhancements of the prediction of ground shock from penetrating weapons, in: *Proceedings of the Fourth International Symposium on the Interaction of Non-nuclear Munitions with Structures (volume 2)*, Panama City Beach, Florida, USA, 1989, pp. 7–12.
- [25] Y. Lu, Underground blast induced ground shock and its modelling using artificial neural network, *Computers and Geotechnics* 32 (2005) 164–178. doi:10.1016/j.compgeo.2005.01.007.
- [26] L. Laine, O. P. Larsen, Numerical study of how the ground shock coupling factor is influenced by soil properties, in: *Proceedings of the 78th Shock & Vibration Symposium*, Philadelphia, PA, USA, 2007.

- 972 [27] L. Laine, Study of planar ground shock in different soils and its propagation around a rigid block, in:
973 Proceedings of the 77th Shock and Vibration Symposium, 2006.
- 974 [28] L. Laine, M. Johansson, O. P. Larsen, Simulation of experiments which show that reflection pressure
975 time history from ground shock depends on the reflected structure's stiffness and mass, in: Proceedings
976 of the 86th Shock and Vibration Symposium, Shock and Vibration Exchange, Orlando, Florida, USA,
977 2015.
- 978 [29] S. Hultgren, Explosion of buried model structures to buried TNT explosions in sand, Technical Report,
979 Fortifikationsförvaltningen Fortf, Report nr C 183, Eskilstuna, Sweden, 1979.
- 980 [30] L. Laine, M. Johansson, O. P. Larsen, 3D FE and 2DOF simulations of ground shock experiments-
981 Reflection pressure time history dependency due to the structure's stiffness and mass, in: Proceedings
982 of the 87th Shock and Vibration Symposium, Shock and Vibration Exchange, New Orleans, Louisiana,
983 USA, 2016.
- 984 [31] L. Laine, M. Johansson, O. P. Larsen, 2D FE and 2DOF Simulations of Ground Shock Experiments –
985 Total Structure's Spring Energy Displacement Dependency to the Charge's and Structure's Properties,
986 in: Proceedings of the 89th Shock and Vibration Symposium, Shock and Vibration Exchange, Dallas,
987 Texas, USA, 2018.
- 988 [32] T. W. Shelton, J. Q. Ehrgott, R. J. Moral, M. Barbato, Experimental and Numerical Investigation
989 of the Ground Shock Coupling Factor for Near-Surface Detonations, Shock and Vibration (2014).
990 doi:10.1155/2014/789202.
- 991 [33] J. E. Windham, Ground shock prediction for conventional weapons effects, US Army Engineer Water-
992 ways Experiment Station, 1996.
- 993 [34] L. Laine, O. P. Larsen, J. Leppänen, Proposal for modifying the empirical equations of soil displacement
994 for ground shock from buried explosions, in: WIT Transactions on The Built Environment, Vol 198,
995 volume 198, WITPress, 2020, pp. 3–13. doi:10.2495/SUSI200011.
- 996 [35] C. Wu, H. Hao, Modeling of simultaneous ground shock and airblast pressure on nearby struc-
997 tures from surface explosions, International Journal of Impact Engineering 31 (2005) 699–717.
998 doi:10.1016/j.ijimpeng.2004.03.002.
- 999 [36] C. Wu, H. Hao, Numerical simulation of structural response and damage to simultaneous
1000 ground shock and airblast loads, International Journal of Impact Engineering 34 (2007) 556–572.
1001 doi:10.1016/j.ijimpeng.2005.11.003.
- 1002 [37] S. Tolani, S. D. Bharti, M. K. Shrimali, T. K. Datta, Estimation of the effect of surface blast on
1003 buildings, in: Proceedings of the Institution of Civil Engineers: Structures and Buildings, volume 174,
1004 ICE Publishing, 2021, pp. 202–214. doi:10.1680/jstbu.19.00055.
- 1005 [38] Y. Lu, H. Hao, G. Ma, Experimental investigation of structural response to generalized ground shock

- excitations, *Experimental Mechanics* 42 (2002) 261–271. doi:<https://doi.org/10.1007/BF02410981>.
- [39] H. Zhou, M. Beppu, G. Ma, Z. Zhao, In-structure shock of underground structures: A revisit with experimental investigation, *Engineering Structures* 56 (2013) 1620–1630. doi:[10.1016/j.engstruct.2013.07.037](https://doi.org/10.1016/j.engstruct.2013.07.037).
- [40] H. Zhou, P. Cong, X. Wang, Y. Wang, H. Dai, S. Yu, J. Du, Global response of underground structures subjected to ground shock with consideration of rise time, *Soil Dynamics and Earthquake Engineering* 143 (2021). doi:[10.1016/j.soildyn.2021.106624](https://doi.org/10.1016/j.soildyn.2021.106624).
- [41] K. Hacıfendioğlu, E. Alpaslan, Stochastically simulated blast-induced ground motion effects on nonlinear response of an industrial masonry chimney, *Stochastic Environmental Research and Risk Assessment* 28 (2014) 415–427. doi:[10.1007/s00477-013-0761-7](https://doi.org/10.1007/s00477-013-0761-7).
- [42] Y. E.-H. Ibrahim, M. Nabil, Risk of surface blast load on pile foundations, *Magazine of Civil Engineering* 90 (2019) 47–61. doi:[10.18720/MCE.90.5](https://doi.org/10.18720/MCE.90.5).
- [43] G. McKenzie, B. Samali, C. Zhang, E. Ancich, Additional structural damage from ground shock as a result of a bombing, *International Journal of GEOMATE* 16 (2019) 190–203. doi:[10.21660/2019.53.68703](https://doi.org/10.21660/2019.53.68703).
- [44] A. Jahami, J. Halawi, Y. Tamsah, L. Jaber, Assessment of Soil–Structure Interaction Effects on the Beirut Port Silos Due to the 4 August 2020 Explosion: A Coupled Eulerian–Lagrangian Approach, *Infrastructures* 8 (2023) 147. doi:[10.3390/infrastructures8100147](https://doi.org/10.3390/infrastructures8100147).
- [45] L. X. Le, L. S. Chin, Energy equivalent impulse method for blast induced ground shock loads, in: *Proceedings of the 26th International Symposium on Military Aspects of Blast and Shock (MABS 26)*, Wollongong, Australia, 2023.
- [46] J. D. McCaughey, Damage from explosions - real or imaginary?, *The Structural Engineer* 62 (1984).
- [47] T. C. Chu, W. F. Ranson, M. A. Sutton, Applications of digital-image-correlation techniques to experimental mechanics, *Experimental Mechanics* 25 (1985) 232–244. doi:[10.1007/BF02325092](https://doi.org/10.1007/BF02325092).
- [48] P. L. Reu, T. J. Miller, The application of high-speed digital image correlation, *Journal of Strain Analysis for Engineering Design* 43 (2008) 673–688. doi:[10.1243/03093247JSA414](https://doi.org/10.1243/03093247JSA414).
- [49] R. J. Curry, G. S. Langdon, Transient response of steel plates subjected to close proximity explosive detonations in air, *International Journal of Impact Engineering* 102 (2017) 102–116. doi:[10.1016/j.ijimpeng.2016.12.004](https://doi.org/10.1016/j.ijimpeng.2016.12.004).
- [50] S. E. Rigby, A. Tyas, R. J. Curry, G. S. Langdon, Experimental Measurement of Specific Impulse Distribution and Transient Deformation of Plates Subjected to Near-Field Explosive Blasts, *Experimental Mechanics* 59 (2019) 163–178. doi:[10.1007/s11340-018-00438-3](https://doi.org/10.1007/s11340-018-00438-3).
- [51] R. Curry, G. Langdon, A. Tyas, Steel Plates Subjected to Localised Blast Loading: Transient Measurements Using Ultra-High-Speed Camera Technology, *Strain* 61 (2025). doi:[10.1111/str.70019](https://doi.org/10.1111/str.70019).
- [52] V. Tiwari, M. A. Sutton, S. R. McNeill, S. Xu, X. Deng, W. L. Fournery, D. Bretall, Application of 3D image correlation for full-field transient plate deformation measurements during blast loading,

- International Journal of Impact Engineering 36 (2009) 862–874. doi:10.1016/j.ijimpeng.2008.09.010.
- [53] R. Curry, Response of plates subjected to air-blast and buried explosions., Ph.D. thesis, University of Cape Town, 2017.
- [54] I. D. Elgy, S. D. Clarke, B. J. Fuller, A. D. Barr, D. W. Armstrong, M. T. Gant, J. J. Keirl, G. C. Porter, I. D. Softly, A. Tyas, Deformation of armox 440t plates subject to buried explosive charge detonations: A benchmark for appliqué systems, International Journal of Impact Engineering 150 (2021). doi:10.1016/j.ijimpeng.2021.103819.
- [55] S. Seo, Y. Ko, M. Chung, Evaluation of field applicability of high-speed 3d digital image correlation for shock vibration measurement in underground mining, Remote Sensing 14 (2022). doi:10.3390/rs14133133.
- [56] S. D. Clarke, J. Warren, A. Tyas, The influence of soil density and moisture content on the impulse from shallow buried explosive charges, in: Proceedings of the 14th International Symposium on Interaction of the Effects of Munitions with Structures (ISIEMS), Seattle, USA, 2011.
- [57] S. D. Clarke, J. Warren, S. D. Fay, S. Rigby, The role of geotechnical parameters on the impulse generated by buried charges, in: Proceedings of the 22nd International Symposium on Military Aspects of Blast and Shock (MABS 22), Bourges, France, 2012.
- [58] S. D. Clarke, S. D. Fay, A. Tyas, J. Warren, S. Rigby, I. Elgy, R. Livesey, Repeatability of buried charge testing, in: Proceedings of the 23rd International Symposium on Military Aspects of Blast and Shock (MABS 23), Oxford, UK, 2014.
- [59] P. S. Westine, B. L. Morris, P. A. Cox, E. Polch, "Development of Computer Program for Floor Plate Response from Landmine Explosions" - Report no. 13045, Technical Report, Southwest Research Institute, 1985.
- [60] NATO, AEP-55, Volume 2 (Edition 2). Procedures for Evaluating the Protection Level of Armoured Vehicles - Mine Threat, Technical Report, NATO, 2011.
- [61] S. D. Clarke, S. D. Fay, J. A. Warren, A. Tyas, S. E. Rigby, J. J. Reay, R. Livesey, I. Elgy, Predicting the role of geotechnical parameters on the output from shallow buried explosives, International Journal of Impact Engineering 102 (2017) 117–128. doi:10.1016/j.ijimpeng.2016.12.006.
- [62] S. E. Rigby, S. D. Fay, A. Tyas, S. D. Clarke, J. J. Reay, J. A. Warren, M. Gant, I. Elgy, Influence of particle size distribution on the blast pressure profile from explosives buried in saturated soils, Shock Waves 28 (2018) 613–626. doi:10.1007/s00193-017-0727-7.
- [63] S. Clarke, S. Rigby, S. Fay, A. Barr, A. Tyas, M. Gant, I. Elgy, Characterisation of buried blast loading, Proceedings of the Royal Society A: Mathematical, Physical and Engineering Sciences 476 (2020). doi:10.1098/rspa.2019.0791.
- [64] R. Waddoups, S. Clarke, A. Tyas, S. Rigby, M. Gant, I. Elgy, An Approach to Quantifying the Influence

- of Particle Size Distribution on Buried Blast Loading, *Eng* 4 (2023) 319–340. doi:10.3390/eng4010020.
- [65] P. Reu, All about speckles: Speckle size measurement, *Experimental Techniques* 38 (2014) 1–2. doi:https://doi.org/10.1111/ext.12110.
- [66] D. G. Farrimond, S. Woolford, A. Tyas, S. E. Rigby, S. D. Clarke, A. Barr, M. Whittaker, D. J. Pope, Far-field positive phase blast parameter characterisation of RDX and PETN based explosives, *International Journal of Protective Structures* 15 (2024) 141–165. doi:10.1177/20414196221149752.
- [67] J. Shin, A. S. Whittaker, D. Cormie, TNT Equivalency for Overpressure and Impulse for Detonations of Spherical Charges of High Explosives, *International Journal of Protective Structures* 6 (2015) 567–579.
- [68] P. Reu, Stereo-rig design: Stereo-angle selection - Part 4, 2013. doi:10.1111/ext.12006.
- [69] Correlated Solutions, Inc., VIC-3D, 2023.
- [70] S. Donner, A. Steinberg, J. Lehr, C. Pilger, P. Hupe, P. Gaebler, J. O. Ross, E. P. S. Eibl, S. Heimann, D. Rebscher, T. Plenefisch, L. Ceranna, The January 2022 Hunga Volcano explosive eruption from the multi-technological perspective of CTBT monitoring, *Geophysical Journal International* 235 (2023) 48–73. doi:10.1093/gji/ggad204.
- [71] H. Carrington, The Frequencies of Vibration of Flat Circular Plates fixed at the Circumference, *The London, Edinburgh, and Dublin Philosophical Magazine and Journal of Science* 50 (1925) 1261–1264. doi:https://doi.org/10.1080/14786442508634850.
- [72] J. Prescott, *Applied Elasticity*, Dover Publications, 1946.
- [73] R. Waddoups, S. Clarke, R. Curry, A. Tyas, D. Farrimond, A. Dennis, T. Lodge, J. Mullins, J. Cordell, Validation of numerical modelling of explosive ground shock propagation in dry sand with digital image correlation experiments, in: *Proceedings of the 19th International Symposium on Interaction of the Effects of Munitions with Structures*, 2024.
- [74] X. Chen, Y. Peng, S. Peng, S. Yao, C. Chen, P. Xu, Flow and fracture behavior of aluminum alloy 6082-T6 at different tensile strain rates and triaxialities, *PLoS ONE* 12 (2017). doi:10.1371/journal.pone.0181983.
- [75] L. Laine, A. Sandvik, Derivation of mechanical properties for sand, in: *Proceedings of the 4th Asia-Pacific Conference on Shock and Impact Loads on Structures*, CI-Premier PTE LTD, Singapore, 2001, pp. 361–368.
- [76] A. Helte, J. Lundgren, H. Örnhed, M. Norrefeldt, Evaluation of performance of m/46, Report No. FOI-R-2051-SE, Technical Report, FOI – Swedish Defense Research Agency, 2006.
- [77] K. Vedantam, D. Bajaj, N. S. Brar, S. Hill, Johnson-Cook strength models for mild and DP 590 steels, in: *Proceedings of CP845, Shock Compression of Condensed Matter*, volume 845 I, American Institute of Physics, 2006, pp. 775–778. doi:10.1063/1.2263437.
- [78] BS EN ISO 898-1:2013, Mechanical properties of fasteners made of carbon steel and alloy steel. Bolts,

- screws and studs with specified property classes - Coarse thread and fine pitch thread, Technical Report, The British Standards Institution, 2013.
- [79] D. D. Fuller, American Institute of Physics Handbook - Section 2d. Coefficients of Friction, American Institute of Physics, 1972.
- [80] G. Tomlinson, CVI. A molecular theory of friction, The London, Edinburgh, and Dublin Philosophical Magazine and Journal of Science 7 (1929) 905–939. doi:10.1080/14786440608564819.
- [81] W. G. Beare, F. P. Bowden, Physical Properties of Surfaces I - Kinetic Friction, Philosophical Transactions of the Royal Society of London - Series A 234 (1935) 329–354.
- [82] B. M. Dobratz, P. C. Crawford, LLNL Explosives Handbook Properties of Chemical Explosives and Explosive Simulants, Lawrence Livermore National Laboratory, 1985.
- [83] G. Taylor, The pressure and impulse of submarine explosion waves on plates, in: G. Batchelor (Ed.), The scientific papers of Sir Geoffrey Ingram Taylor, volume 3, Cambridge University Press, 1963, pp. 287–303.
- [84] Y. B. Zel'dovich, Y. P. Raizer, Physics of shock waves and high-temperature hydrodynamic phenomena, Dover Publications, 2002.
- [85] C. H. Braithwaite, J. I. Perry, N. E. Taylor, A. P. Jardine, Behaviour of sand during release from a shocked state, Applied Physics Letters 103 (2013). doi:10.1063/1.4824764.
- [86] M. S. Zhao, X. G. Wang, E. A. Chi, Q. Kang, Influence of distance from blast center on time-frequency characteristics of blast vibration signals, Advanced Materials Research 1033-1034 (2014) 444–448. doi:10.4028/www.scientific.net/AMR.1033-1034.444.
- [87] D. Liu, W. Lu, Y. Liu, M. Chen, P. Yan, P. Sun, Analysis of the main factors influencing the dominant frequency of blast vibration, Shock and Vibration 2019 (2019). doi:10.1155/2019/8480905.
- [88] C. Yu, Z. Wang, W. Han, A prediction model for frequency spectrum of blast-induced seismic wave in viscoelastic medium, Geophysical Prospecting 66 (2018) 87–98. doi:10.1111/1365-2478.12601.
- [89] S. Hultgren, Response of buried model structures to buried tnt explosions in sand, in: 6th International Symposium on Military Aspects of Blast and Shock, 1979.

Appendix A. Predictive formulae from literature

Eqn. No.	Source	Velocity Formula	Terms
A.1	Westine [22] 1978 (dimensionless)	$\frac{U}{c} \left(\frac{p_0}{\rho c^2} \right)^{1/2} = \frac{6.169 \times 10^{-3} \left(\frac{W}{\rho c^2 R^3} \right)^{0.8521}}{\tanh \left[26.03 \left(\frac{W}{\rho c^2 R^3} \right)^{0.30} \right]} \quad (\text{A.1})$	U = peak radial soil particle velocity c = speed of sound in soil p_0 = atmospheric pressure ρ = mass density of the soil W = explosive energy release R = standoff distance
A.2	Drake and Little [7] 1983	$V_0 = f \cdot 160 \cdot \left(\frac{R}{W^{1/3}} \right)^{-n} \quad (\text{A.2})$	V_0 = peak radial soil particle velocity (ft/s) f = coupling factor for near surface detonations R = standoff distance (ft) W = charge mass (lb) n = attenuation exponent
A.3	Drake et al. [24] 1989	$v_p = \begin{cases} f^* \cdot \frac{606.2}{\sqrt{\rho_0}} \left(\frac{r}{W^{1/3}} \right)^{-3/2} & r \leq .155 W^{1/3} \\ f^* \cdot \frac{9906}{\sqrt{\rho_0}} \left(\frac{r}{r_c} \right)^{-n} & r > .155 W^{1/3} \end{cases} \quad (\text{A.3})$	v_p = peak radial soil particle velocity (m/s) f = coupling factor for near surface detonations ρ_0 = mass density of the soil (kg/m^3) r = standoff distance (m) W = charge mass (kg) $r_c = .155 W^{1/3}$ n = attenuation exponent *use fully-coupled v_p for subsequent calculation of acceleration and pressure

Table A.3: Historic predictive formulae for soil particle velocity

Eqn. No.	Source	Acceleration Formula	Terms
A.4	Drake and Little [7] 1983	$a_0 \cdot W^{1/3} = f \cdot 50 \cdot c \cdot \left(\frac{R}{W^{1/3}} \right)^{-n-1}$ (A.4)	a_0 = peak particle acceleration ($g's$) W = charge mass (lb) f = coupling factor for near surface detonations c = speed of sound in soil (ft/s) R = standoff distance (ft) n = attenuation exponent
A.5	Drake et al. [24] 1989	$a_p = f \cdot \frac{2v_p}{gt_r}$ (A.5)	a_p = peak particle acceleration ($g's$) f = coupling factor for near surface detonations v_p = peak radial soil particle velocity (m/s) g = gravitational acceleration (m/s^2) t_r = rise time (s) $t_r = \left(\frac{c_f}{c_L} - 1 \right) \frac{r}{c_f}$ $c_f > c_L$ c_f = speed of sound in soil (m/s) $c_L = c_0 + Sv_p(m/s)$ c_0 = initial loading velocity of the soil (m/s) S = equation of state factor: 1.5 for geologic media
A.6	Hansen [20] 2016 (10% sat.)	$a_{peak} = 2661.1 \cdot \left(\frac{R}{W^{1/3}} \right)^{-2.98}$ (A.6)	a_{peak} = peak particle acceleration ($g's$) R = standoff distance (m) W = charge mass (kg)

Table A.4: Historic predictive formulae for soil particle acceleration

Eqn. No.	Source	Pressure Formula	Terms
A.7	Westine and Friesenhahn [23] 1983 (dimensionless)	$\frac{\left(\frac{P}{\rho c^2}\right)}{(4.35 + \frac{Y}{d}) \left[0.25 + 0.75 \tanh\left(0.48 \frac{\rho^{1/3} c^{2/3} d}{W^{1/3}}\right)\right]} = 0.0175 \left(\frac{W^{1/3}}{\rho^{1/3} c^{2/3} R_{eff}}\right)^{3.42} \quad (A.7)$	P = peak pressure ρ = mass density of the soil c = speed of sound in soil Y = depth of point below C.G. of ordnance d = depth of ordnance buried to its C.G. W = explosive energy release R_{eff} = effective slant range which accounts for ordnance geometry and orientation $\frac{R_{eff}}{l} = \left[\frac{M - N^2 - \frac{1}{4}}{\frac{N+0.5}{(M+N)^{1/2}} - \frac{N-0.5}{(M-N)^{1/2}}} \right]^{1/3}$ $M = (Z/l)^2 + (X/l)^2 + (Y/l)^2 + 1/4$ $N = (Y/l) \cos \theta + (Z/l) \sin \theta$ Z = horizontal distance of location in vertical plane through the bomb X = transverse distance of location l = length of explosive source θ = bomb orientation ($\theta = 0$ degrees is a vertical bomb)
A.8	Drake and Little [7] 1983 (psi, ft, sec, g's, lb)	$P_0 = f \cdot (\rho c) \cdot 160 \cdot \left(\frac{R}{W^{1/3}}\right)^{-n} \quad (A.8)$	P_0 = peak pressure (<i>psi</i>) f = coupling factor for near surface detonations ρc = acoustic impedance (<i>psi/ft/s</i>) R = standoff distance (<i>ft</i>) W = charge mass (<i>lb</i>) n = attenuation exponent
A.9	Drake et al. [24] 1989 (Pa, m, sec, g's, kg)	$\sigma_p = f \cdot \rho_0 c_L v_p \quad (A.9)$	σ_p = peak pressure (<i>Pa</i>) f = coupling factor for near surface detonations ρ_0 = mass density of the soil (<i>kg/m³</i>) $c_L = c_0 + S v_p$ (<i>m/s</i>) c_0 = initial loading velocity of the soil (<i>m/s</i>) S = equation of state factor: 1.5 for geologic media v_p = peak radial soil particle velocity (<i>m/s</i>)
A.10	Leong et al. [3] 2007 (Pa, m, sec, kg)	$\left(\frac{P_0}{\rho c^2}\right) = 10^{f(n)} \left[R \left(\frac{\rho c^2}{W}\right)^{1/3} \right]^{-n}$ for $50 \leq R \left(\frac{\rho c^2}{W}\right)^{1/3} \leq 50000$ where $f(n) = 1.24n + 1.33 \leq 4.07$ $(A.10)$	P_0 = peak pressure (<i>Pa</i>) ρ = mass density of the soil (<i>kg/m³</i>) c = speed of sound in soil (<i>m/s</i>) R = standoff distance (<i>m</i>) W = charge weight (<i>N</i>) n = attenuation exponent
A.11	Hansen [20] 2016 (kPa, m, sec, kg, g's) (10% <i>sat.</i>)	$\sigma_{peak} = 1490.2 \cdot \left(\frac{R}{W^{1/3}}\right)^{-3.03} \quad (A.11)$	σ_{peak} = peak pressure (<i>kPa</i>) R = standoff distance (<i>m</i>) W = charge mass (<i>kg</i>)

Table A.5: Historic predictive formulae for soil shock pressure

MAT_PSEUDO_TENSOR Parameters		Pressure Dependent Yield Surface Parameters	
		p [MPa]	f_2 [MPa]
		0.000	0.000
ρ	1674 [kg/m ³]	3.401	4.235
G	77 [MPa]	34.898	44.695
ν	0.3	101.324	124.035
		184.650	226.000
		500.000	226.000

Table B.6: Sand Material parameters (values taken from [75])

EOS_TABULATED_COMPACTON Parameters		Plastic Compaction Curve Parameters		
		ε_v	$C(P)$ [GPa]	K [GPa]
		0	0	0.118
		-0.07374	0.009285	1.79
		-0.14251	0.02052	3.38
γ	0.0	-0.20671	0.0392	5.61
E_0	0.0	-0.26709	0.07254	8.54
V_0	1.0	-0.32411	0.1326	12.3
		-0.37801	0.2393	16.9
		-0.42912	0.4236	22.4
		-0.51419	3.402	60.1

Table B.7: Sand Equation of State parameters (adapted for LS-DYNA use from [75])

MAT_HIGH_EXPLOSIVE_BURN Parameters	
ρ	1500 [kg/m ³]
D	7680 [m/s]
P_{CJ}	21.15 [GPa]
β	0.0
K	0.0
G	0.0
σ_y	0.0

Table B.8: PE10 Material parameters (values taken from [76])

EOS_JWL Parameters	
A	759.9 [GPa]
B	12.56 [GPa]
R_1	5.1
R_2	1.5
ω	0.29
E_0	7.05 [kJ/cm ³]
V_0	1.0

Table B.9: PE10 Equation of State parameters (values taken from [76])

MAT_SIMPLIFIED_JOHNSON_COOK Parameters	
ρ	2710 [kg/m ³]
E	73 [GPa]
ν	0.33
VP	0
A	0.277 [GPa]
B	0.308 [GPa]
N	0.69
C	0.0032
ε_{PFAIL}	0.18
σ_{MAX}	1e28 [GPa]
σ_{SAT}	1e28 [GPa]
$EPS0$	1

Table B.10: Al-6082-T6 Material parameters (values taken from [74])

MAT_SIMPLIFIED_JOHNSON_COOK Parameters	
ρ	7800 [kg/m ³]
E	210 [GPa]
ν	0.3
VP	0
A	0.217 [GPa]
B	0.2337 [GPa]
N	0.6428
C	0.0756
ε_{PFAIL}	1e17
σ_{MAX}	1e28 [GPa]
σ_{SAT}	1e28 [GPa]
$EPS0$	1

Table B.11: Mild Steel Material parameters (values taken from [77])

MAT_NULL Parameters	
ρ	1.223 [kg/m ³]
PC	0.0 [Pa]
μ	0.0
T_{EROD}	0.0
C_{EROD}	0.0
YM	0.0
ν	0.0

Table B.12: Air Material parameters

EOS_LINEAR_POLYNOMIAL Parameters	
C_0	0.0 [Pa]
C_1	0.0 [Pa]
C_2	0.0 [Pa]
C_3	0.0 [Pa]
C_4	0.4
C_5	0.4
C_6	0.0
E_0	2.5e-4 [kJ/cm ³]
V_0	1.0

Table B.13: Air Equation of State parameters

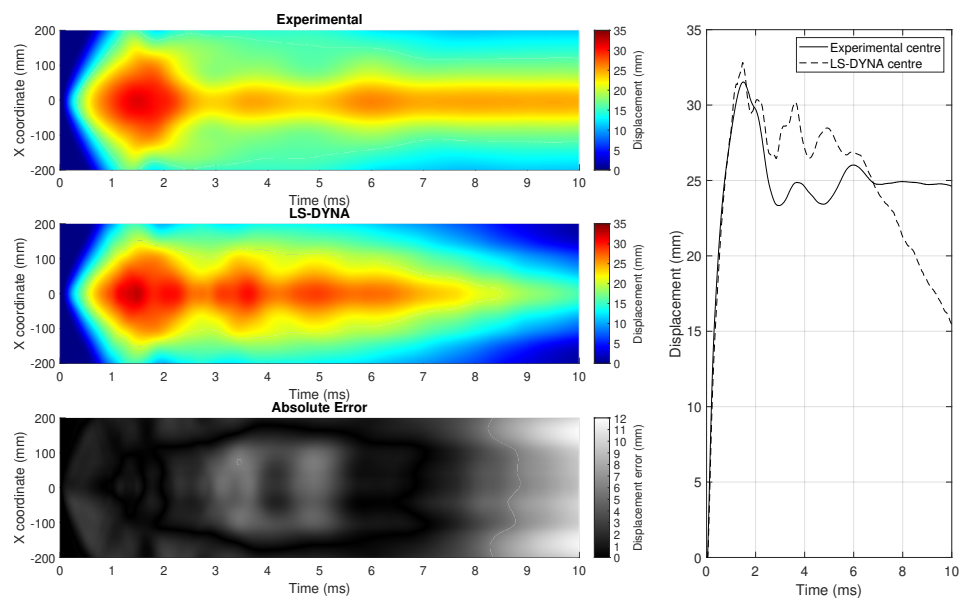


Figure C.25: 50 mm SOD, 10 g PE10, experimental (Test C) displacement–time history and the corresponding numerical modelling output

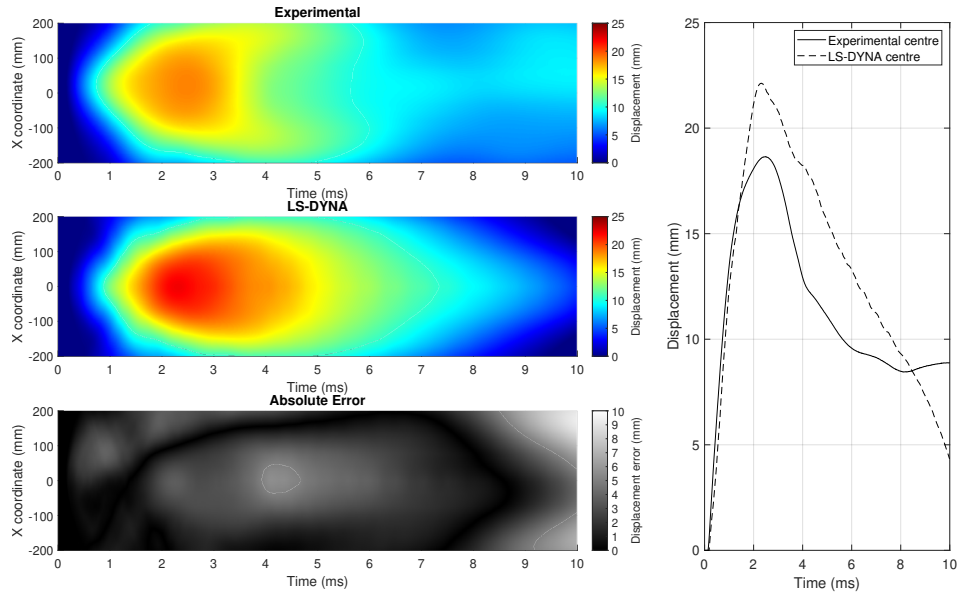


Figure C.26: 100 mm SOD, 10 g PE10, experimental (Test D) displacement-time history and the corresponding numerical modelling output

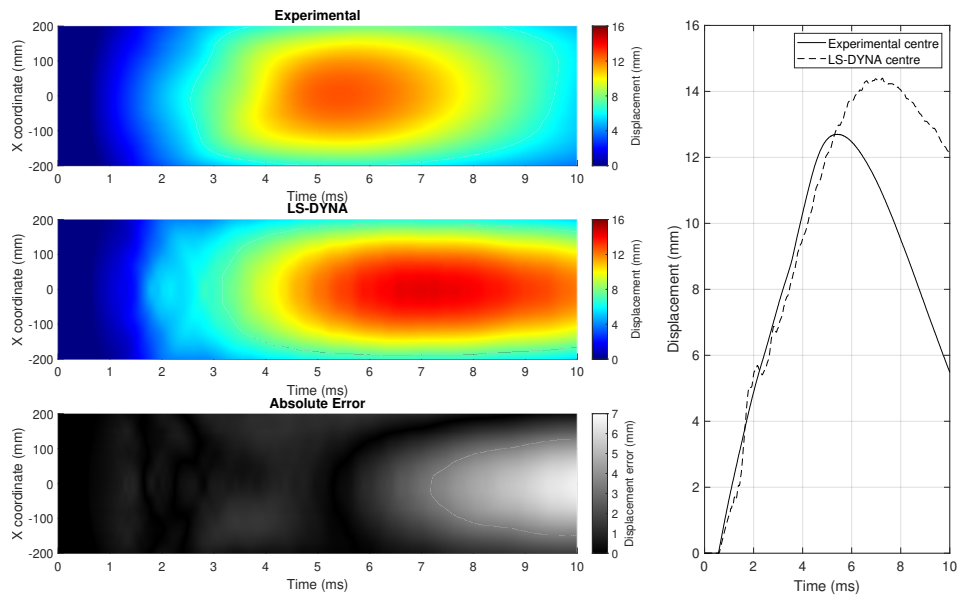


Figure C.27: 250 mm SOD, 10 g PE10, experimental (Test E) displacement-time history and the corresponding numerical modelling output

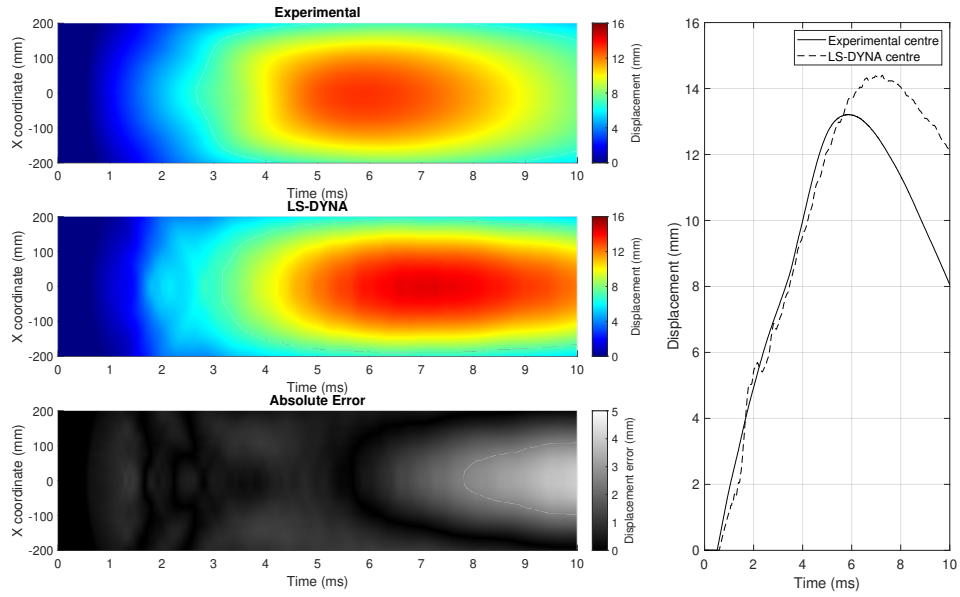


Figure C.28: 250 mm SOD, 10 g PE10, experimental (Test F) displacement-time history and the corresponding numerical modelling output

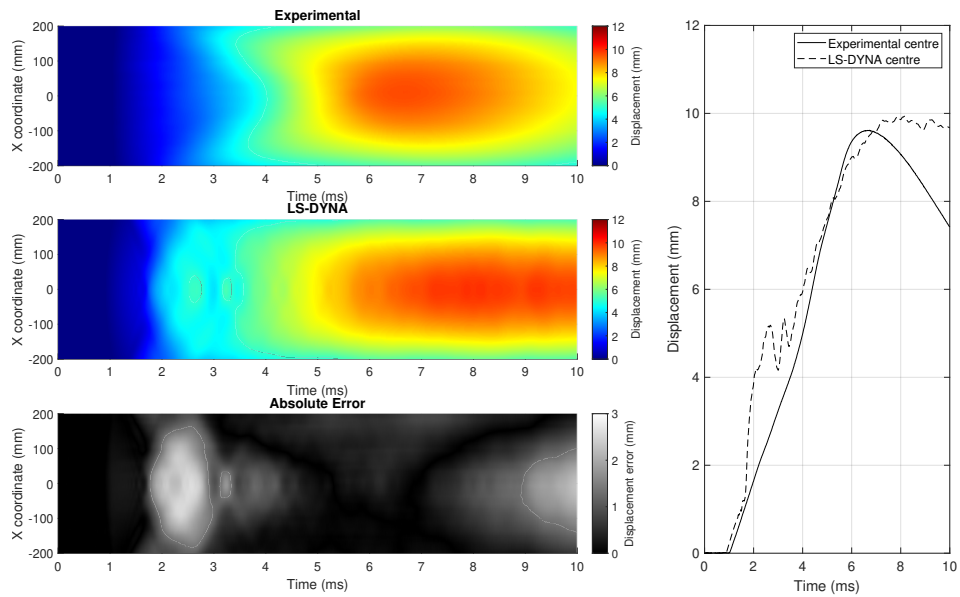


Figure C.29: 350 mm SOD, 10 g PE10, experimental (Test G) displacement-time history and the corresponding numerical modelling output

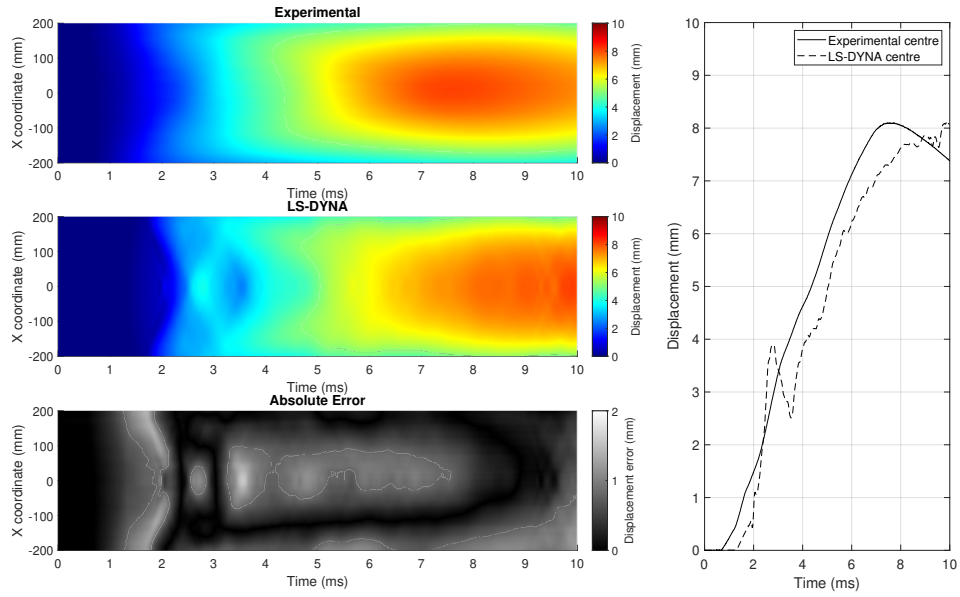


Figure C.30: 500 mm SOD, 10 g PE10, experimental (Test H) displacement-time history and the corresponding numerical modelling output

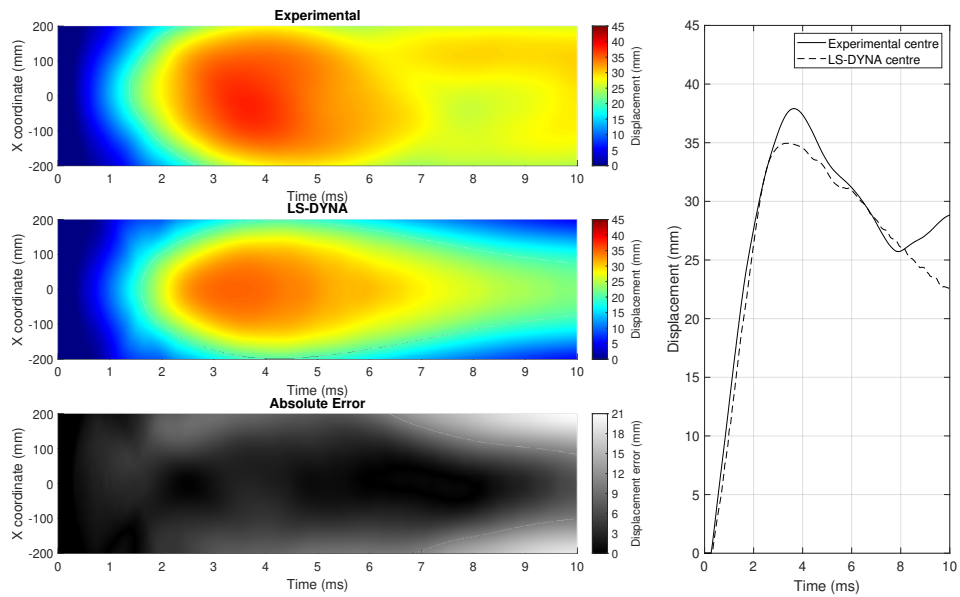


Figure C.31: 171 mm SOD, 50 g PE10, experimental (Test I) displacement-time history and the corresponding numerical modelling output