

Comparative Study of Concrete Constitutive Models for Predicting Blast Response

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Synopsis: The robustness of concrete constitutive material models in explicit finite element codes is typically measured by their ability to match peak dynamic and permanent displacements of a reinforced concrete specimen. In a series of recent shock tube experiments, reinforced concrete slabs were subjected to simulated blast loads. Applied pressure histories were recorded in these tests, as were peak and residual displacements. This paper evaluates the Concrete Damage Model (Material 72, Release 3) and the Continuous Surface Cap Model (Material 159) within LS-DYNA (Version 971), as well as the Applied Engineering Cap model (AEC-3I) in DYNA3D. The results indicate some variation in predicted damage and failure modes between the three material models, but overall, all three models produced satisfactory comparisons to the test with regard to peak positive deflection (i.e., within a factor of 2 of the measured response). A surprising outcome is that the inclusion of rate-dependent material properties actually increased the error in the predicted response. In terms of predicting crack patterns, the AEC-3I model appears to be preferable, whereas MAT72R3 is preferred for predicting peak deflection. Overall, MAT159 was the most consistent predictor and the least sensitive to variations in the rate dependent properties of concrete.

Keywords: concrete, constitutive, model, blast, deflection, slab, response, shock, tube, test

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David Bogosian, P.E., has accumulated over 25 years of experience in design and analysis of structures subjected to dynamic and impulsive loading from blast and impact loads. He has been involved in numerous studies in which LS-DYNA was applied to computation of the response of reinforced concrete structures subjected to blast and fragment effects from conventional munitions. He is also a co-author of three chapters in the Design and Analysis of Hardened Structures for Conventional Weapons Effects manual (UFC 3-340-01).

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Johan Magnusson obtained an MSc degree in Structural Engineering in 1998. He worked as a Research Engineer for ten years at the Swedish Defense Research Agency conducting research on the effects of weapons on protective structures. Mr. Magnusson joined Baker Engineering and Risk Consultants where he worked for two years as a Senior Engineer. At the end of 2010, he returned to Sweden and commenced his current employment at the Swedish Fortifications Agency.

Alex Christiansen received a B.S. in aerospace engineering and a M.S. in mechanical engineering from the University of California, Los Angeles. His work for both commercial and government clients has focused on the analysis and design of new and existing structures subjected to dynamic and conventional loading. In this work, he has performed many evaluations of structures using high-fidelity finite element models and worked on the development of non-linear material models.

INTRODUCTION

Recent years have seen the increased and widespread use of explicit finite element (FE) models to represent the response of structures to dynamic blast and impact loads. In particular, reinforced concrete structures (typically wall and floor/roof slabs, but also beams, columns, domes, and other geometries) have been the focus of extensive research, using both analytical and experimental techniques. As a result, numerous nonlinear material models have been developed for use in finite element codes to represent concrete response all the way to the most severe damage levels. These models attempt to replicate the complex behaviors of the real material with varying degrees of fidelity, while making available to the analyst a practical means of using FE technology to either predict response or design structural members to resist blast and impact loads.

The purpose of the current paper is to evaluate the ability of three different constitutive concrete models to predict the response of reinforced concrete slabs subjected to blast loads. The models in question are the AEC-3I model (available in DYNA3D), Material 72–Release 3 (available in LS-DYNA), and Material 159 (also in LS-DYNA). The approach taken was to select a set of two well-controlled experiments (one at a moderate level response, the other at a higher level) on a simple one-way reinforced concrete slab and use those results to evaluate all three models. To the extent practicable, other aspects of the modeling process (boundary conditions, blast loads, mesh discretization, representation of rebar) were held constant across all the analyses. The final results are helpful in understanding the sensitivity of calculated response to one's choice of material model. They also indicate each model's ability to replicate various aspects of the experimentally observed response.

The context of the comparisons contained in this paper is that of a consulting engineering practice with a primary focus on computing the response of structures to blast loads from a variety of sources. As such, the material models selected for the comparison are those which are widely used within our company, and to our knowledge, within the broader community of consulting engineers. While perhaps not representative of the latest and technologically most advanced constitutive models (such as those representing crack formation and propagation), they are typical of models used for production calculations supporting building assessments and designs for various real-world applications.

PREVIOUS EXPERIMENTAL RESEARCH

A set of previously performed experiments^[1] was used as a reference for the numerical simulations. The experimental study focused on investigating the blast resistance of reinforced concrete slabs with mechanical reinforcement splices. The slabs were 8.5 ft tall by 8.0 ft wide and 5.5 in thick [2.59 m tall by 2.44 m wide by 140 mm thick]. The concrete was designed to have a nominal 28-day [0.93 month] compressive strength of 4,000 psi [27.6 MPa]. The reinforcement consisted of No. 5 [15.9 mm diameter] steel reinforcing bars spaced at 12 inches [305 mm] on center spanning in the vertical direction and at mid-depth, and with No. 4 bars [12.7 mm diameter] spaced at 12 inches [305 mm] on center spanning in the horizontal direction. The nominal steel yield strength was 60,000 psi [414 MPa]. The slab was supported at the top and bottom only to achieve vertical one-way flexural behavior, consistent with the primary reinforcing being in the vertical direction. Details of the supports are shown in Figure 1.

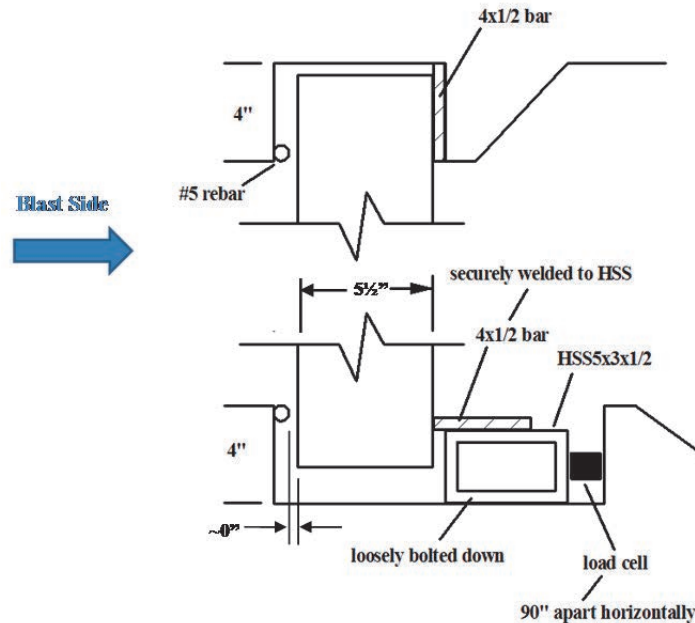


Figure 1 — Slab Support Conditions for Shock Tube Tests

The blast tests were conducted at the BakerRisk's large shock tube facility (Figure 2). The shock tube consists of two major sections, a driver section and an expansion section. Blast pressures are generated when a rupture diaphragm placed between the two sections fails at a specified pressure in the driver section. A shock wave then propagates along the expansion section and loads the test specimen at the end of this expansion section.



Figure 2 — BakerRisk Large Shock Tube

Within the series, two tests (numbered 1 and 6) were conducted on baseline slabs with no rebar couplers. These tests were chosen for use in this analytical study, since they eliminate the potential complications introduced by the use of mechanical couplers. The tests were conducted at different levels of loading, with test 1 registering a peak applied pressure of 7.7 psi [53 kPa] while test 6 reached 10.6 psi [73 kPa]. The impulse in test 6 was also significantly higher. The applied pressure, as measured from three pressure gauges mounted in the shock tube wall only 3 inches [76 mm] upstream from the face of the wall, is presented in Figure 3. The curve shown for each test represents the average of the three gauges (the variability among the individual gauges was relatively small). Aside from the change in loading, the two experiments were otherwise identical.

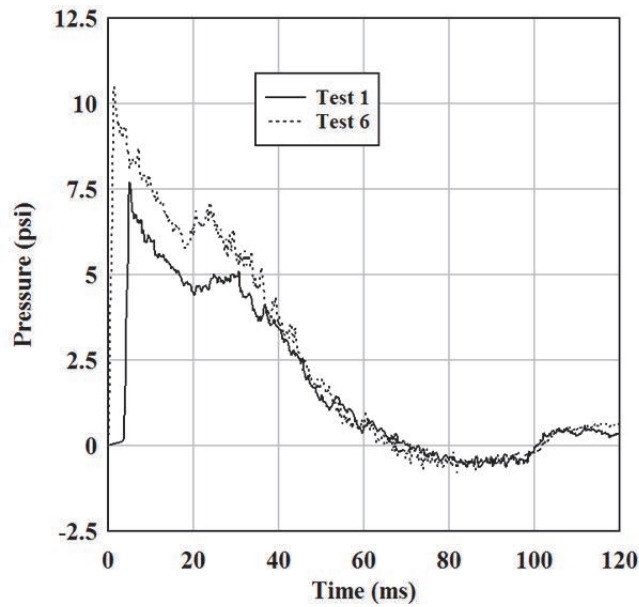


Figure 3 — Blast Wave Profiles from Shock Tube Tests 1 and 6

In addition to the pressure gauges, high speed cameras recorded the response of the slabs when subjected to the blast loads. By recording the position of the slab back face at various times over the duration of the video record, a displacement history of the slab centerline was obtained for each test, as plotted in Figure 4. As the figure indicates, the response in test 6 is nearly three times as large as that in test 1, even though the applied load was only incrementally larger. We also observe that the slab in test 1 experienced significant rebound even though the deflection went well into the plastic regime, with the late-time displacement being under half an inch [12 mm] (20% of peak), whereas test 6 produced a late-time residual displacement of approximately 2 inches [51 mm] (30% of peak). A possible factor here may be the presence of late-time negative phase loads produced by the shock tube.

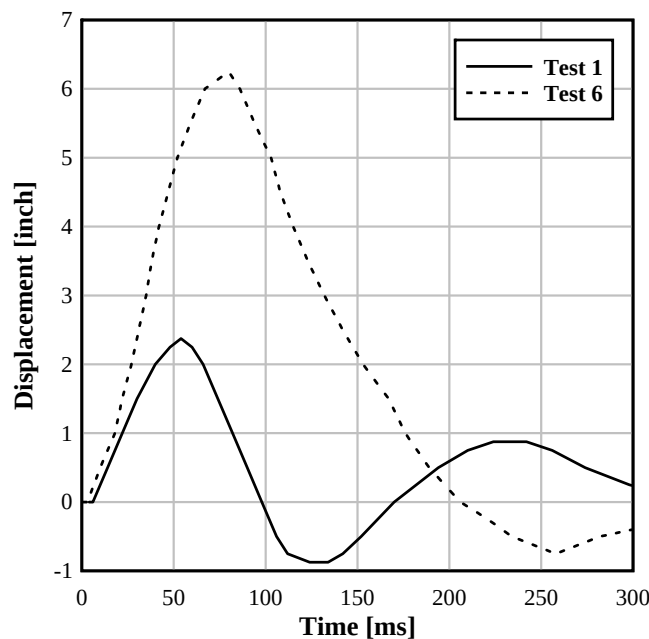


Figure 4 — Slab Centerline Displacement Histories

For convenience, Table 1 contains a summary of the key blast metrics and results from the two tests. Note that the peak support rotation (which is the typical way of characterizing damage) is around 3 degrees in test 1, while the test 6 result is around 7 degrees. Typical damage criteria for reinforced concrete slabs without shear reinforcing indicate a transition from medium to high damage at 2 degrees, and from high damage to failure at 5 degrees^[2]. Using these criteria, the test 1 slab would be labeled “high damage” while the test 6 slab would be at “failure.” Figure 5 shows photos of the slabs after each test, viewed from the downstream side of the slab (i.e., the non-loaded side). The slabs clearly responded in vertical one-way action, with consistent horizontal cracks across their entire width. The degree of cracking visible in test 1 is quite minor, and even though there is some small residual displacement, labeling this result as “high damage” is clearly conservative. Furthermore, while the test 6 slab shows some large visible cracks and significant permanent deformation, it appears to be in no danger of imminent failure. These observations reinforce the heavily conservative approach typically used to define damage criteria for design purposes.

Table 1 — Key Metrics from Slab Tests

Test No.	Applied Pressure (psi) [kPa]	Applied Impulse (psi·ms) [kPa·ms]	Peak Deflection (in) [mm]	Peak Support Rotation (°)
1	7.7 [53]	217 [1,500]	2.4 [61]	2.7
6	10.6 [73]	297 [2,050]	6.2 [157]	7.1

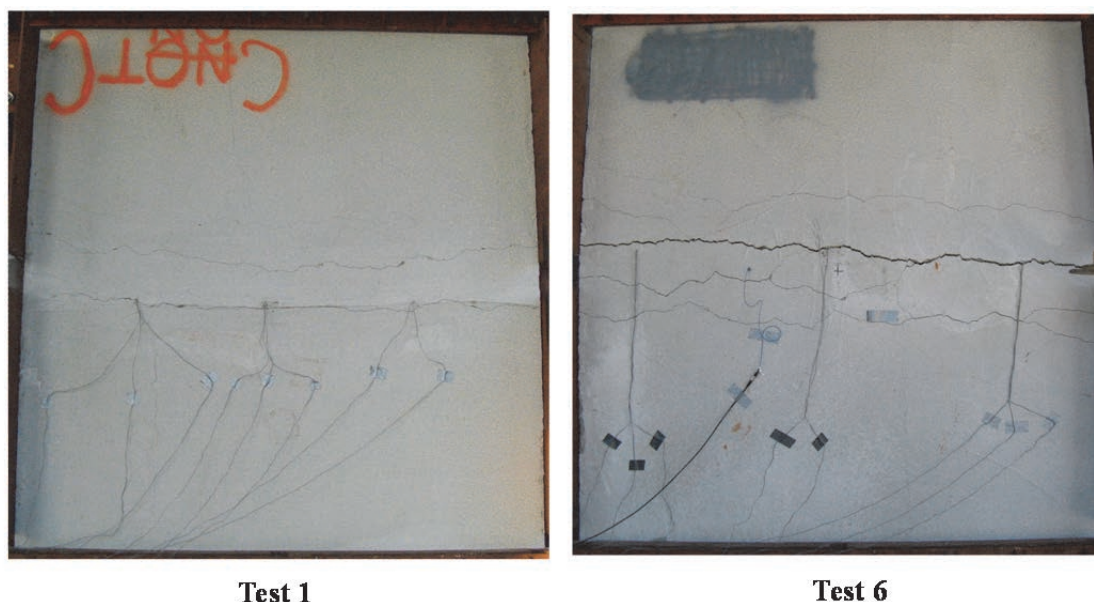


Figure 5 — Post Test Photos of Slab Surface (Non-Loaded Side).

Actual concrete strengths for the test articles were not experimentally obtained. The concrete mix was specified as having a nominal 28-day compressive strength of 4,000 psi [27.6 MPa]. The two slabs had somewhat different cure time at their respective test dates. A reasonable estimate of the test-day strength of the concrete is 3,900 psi [26.9 MPa] for Test 1 and 4,400 psi [30.3 MPa] for Test 6, using standard curves for concrete aging and assuming a 28-day static increase factor of 1.1 over the nominal design strength of the concrete mix.

FINITE ELEMENT MODEL

A 3D finite element model was defined to represent the test slab and its supports. For consistency, a single mesh was used in the analyses using all three material models. The model is shown schematically in Figure 6, rotated 90° for convenience. Hexahedral (solid brick) elements were used to represent the concrete, with six elements through the slab's thickness. As a result, the concrete element size is slightly less than 1 inch in each dimension (0.96×0.96×0.92 inch, to be precise).

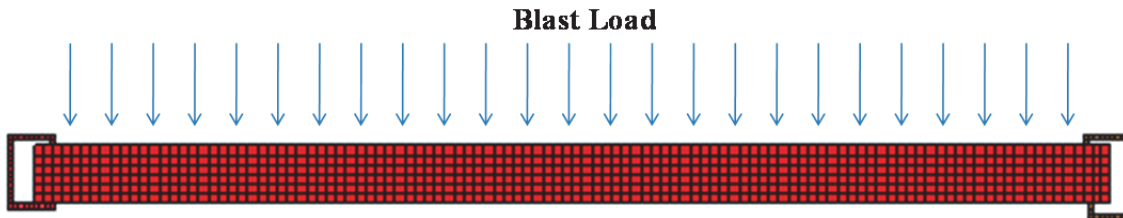


Figure 6 — Section View Through the Concrete Slab (Shown Rotated 90°)

A mesh convergence study would have been of additional interest, but may also have distracted attention from the primary purpose of this study, as each constitutive model may have reached a target convergence tolerance at varying levels of mesh refinement. Instead, since our goal was to exercise all models using the same mesh, a fixed mesh size was used for all three models. Experience with calculation of blast-loaded slabs has indicated that a mesh with six elements through the thickness is a reasonable lower bound for practical problems. More elements are certainly better, but for typical applications, this level of mesh refinement has historically produced satisfactory results.

Beam elements were used to represent the reinforcing. Both vertical and horizontal rebar were collocated at the mid-plane of the slab, using shared nodes between the beams and the solid elements (no provision for bond-slip between rebar and concrete). For a slab response in the light to moderate range of damage, inclusion of bond-slip is not typically required since the amount of slippage is negligible. The steel supports were modeled with an elastic material that was rigidly supported, with contact defined between the slab and the steel. The boundaries replicate the actual conditions in the test article, and are similar to simply-supported conditions. Note that the slab is unable to develop any arching or membrane behavior.

Because of the symmetry of the slab, a vertical half-segment was included in the model with a symmetry boundary applied along the vertical centerline. The blast load measured in each test (Figure 3) was applied as a uniform dynamic pressure over the entire slab surface.

The reinforcing steel used in the test articles was ASTM A 605 Grade 60 rebar, with a minimum yield strength of 60 ksi [414 MPa]. The steel was represented in the finite element model using an elastic-plastic model (MAT3 in the LS-DYNA and DYNA3D material libraries). For those runs which involved rate sensitivity in the concrete (see discussion below), the steel strength was increased from its nominal 60 ksi [414 MPa] with a static increase factor of 1.1 (to represent realistic vs. minimum strength) and a dynamic increase factor of 1.22. A tangent modulus of 230 ksi [1,590 MPa] was included to represent post-yield hardening behavior. Failure was not included in the steel model as the rebar did not reach significantly high strains, nor was failure observed in either of the experiments themselves.

CONCRETE MATERIAL MODELS

Three different concrete models were evaluated in this study; the primary features of each are briefly summarized below. The models were chosen since they are implemented within widely available explicit finite elements codes, are broadly used within the blast effects community, and are familiar to the authors.

AEC-3I model

The Applied Engineering Cap Model with Three Invariants (AEC-3I)^[3] was implemented into DYNA-3D under a license/collaboration agreement with Lawrence Livermore National Laboratory. The AEC-3I models material plasticity with a unified shear yield and cap surface that can harden from an initial yield position to ultimate strength, and then soften to residual surfaces under continued loading. The cap portion of the yield surface only hardens and does not soften. The AEC-3I incorporates standard predictions of elastic stresses until a yield surface is reached. The AEC-3I uses a non-associated flow rule in the form of normal stress correction to the yield surfaces. Dilation and shear compaction are controlled through relative strain hardening parameters of the shear yield surface and cap, respectively. The shear yield surface provides strength dependence upon mean normal stress, which typically represents the behavior of frictional materials such as concrete, rock, ceramics and soils. The yield surfaces are also a function of the intermediate principal stresses (third invariant) for modeling strength dependence on stress state (triaxial compression, extension, and torsion). Strain softening is dependent on mean normal stress for modeling brittle compressive failure at low pressures, transitioning to ductile behavior at high pressures.

A decohesion model for modeling tensile fracture and crack growth^[4] was married with the plasticity of the AEC-3I model in a program funded by the Defense Threat Reduction Agency (DTRA) in 2005 and 2006. The decohesion algorithms model the formation of crack planes with defined orientation, and controlled growth based on material fracture energy, specified as a user input. Crack growth is represented with decohesion strain components resulting in anisotropic material behavior. Finite elements cannot carry traction forces across a fully formed crack plane, and subsequent compressive forces are not engaged until the crack has closed. Crack formation does not modify the shear yield surfaces of the material. Any number of crack failure planes can form and grow within any finite element. Orientation of crack plane initiation is based on principal-stress Rankine criteria, characteristic of brittle-fracture materials. Use of these criteria results in efficient computational speeds similar to standard plasticity models.

The AEC-3I/decohesion model does not incorporate strain rate effects on material properties. Instead, dynamic strength enhancement is modeled by explicit (offline) application of an increase factor to the compressive and tensile strength values.

MAT72R3

Material 72 in the LS-DYNA material library (identified as MAT_CONCRETE_DAMAGE_REL3) was the second model used to represent the concrete. This material model consists of three shear failure surfaces, the initial yield surface, the maximum yield surface and the residual yield surface.^[5, 6] During initial loading, the deviatoric stresses are elastic until the initial yield surface is reached. The stresses can then increase further until these reach the maximum yield surface. Beyond this stage, the response can soften to the residual surface or be perfectly plastic. Shear dilatancy, which occurs as concrete approaches failure,^[9] is incorporated in the model. The user has the option to take shear dilatancy into consideration via the fractional dilatancy parameter α ; the parameter can range from 0.0 to 1.0, with a default value of 0.5. A softening model for concrete in tension is also included in the material model.

One noteworthy feature of the model is its ability to accept user input of two separate curves defining the dynamic increase factor as a function of strain rate, one curve for tension and another in compression. This allows the model to represent the significant differences that have been observed in physical experiments between rate enhancement in tension and compression.

MAT159

Material 159 from the LS-DYNA material library^[7] (identified as MAT_CSCM_CONCRETE) was also used to represent the concrete. This material model has a smooth intersection between the failure surface and the hardening cap and is often referred to as a Continuous Surface Cap Model (CSCM).^[7] The model parameters are fit to data for unconfined compression strengths between approximately 4,000 and 8,400 psi [27.6 and 57.9 MPa]. Shear dilatancy, which occurs as concrete in compression approaches failure,^[7, 9] is incorporated in the model. The

softening of concrete after the peak stresses have been reached is modeled via a damage formulation. This formulation models strain softening and modulus reduction for both tensile and compressive hydrostatic pressures. The concrete model also includes strain rate dependence for tensile and compressive states of stress, as well as for fracture energy. Default values of parameters controlling the rate sensitivity were used in these calculations.

CONCRETE MODEL FEATURES

Two model features, their representation of strain rate enhancement and their ability to represent stress states for material that is loaded cyclically, are discussed below.

Strain rate enhancement

The apparent enhancement of both compressive and tensile strength of concrete when loaded rapidly (i.e., at high rates of strain) has been well documented in numerous earlier studies. The three models considered in this paper treat concrete rate sensitivity in varying ways, as described above. In order to better evaluate the models on a consistent basis, the following finite element analyses were conducted.

First, as a baseline, the static concrete strength was used with no rate enhancement of any kind.

Second, a constant dynamic increase factor (DIF) was applied to the concrete strength as part of the input, without using actual rate-dependent features of the material models. In this study, a DIF of 1.19 was applied to the static compressive strength; since the tensile strength is computed internally by all three models using a square-root formulation, the effective DIF in tension would be 1.10. The value of 1.19 was obtained from guidance contained in standard published design manuals^[10] for reinforced concrete sections in bending.

Third, fully rate-dependent properties were used in models MAT72R3 and MAT159 only; the AEC-3I model does not have strain rate dependence as an option. In the case of MAT159, the rate dependence (DIF as a function of strain rate) is internally fixed via a hard-wired function with no user inputs. In the case of MAT72R3, the user is able to input DIF curves in both tension and compression. The curves used in this study were computed as a function of the static compressive strength using the approach derived by Malvar,^[11] which in turn is based on the recommendations of the European concrete design code.^[12]

Figure 7 plots the rate curves used by these two models in tension and compression along with the constant DIF values. Note the dissimilar vertical scales in the two plots: the rate enhancement in tension greatly exceeds that in compression as the rate goes beyond 1 s^{-1} . We also see that the constant DIF value is quite conservative for all but the very lowest strain rates, as would be expected in a design context.

Cyclic loading

One potentially important feature of a concrete constitutive model is its ability to represent the material's behavior when undergoing cyclic loading. In particular, if the concrete is first loaded in tension, cracks and fails, and then is reloaded in compression, the material's ability to model the crack formation followed by closing of the crack and eventual reloading in compression would be very relevant to its ability to represent slab response, particularly in the time domain following peak response and during the rebound phase.

To evaluate our three models, a simple single-element calculation was performed in which a 1-inch cube of concrete was subjected to a displacement constraint on one surface while being held on the other. The cube was allowed to expand or contract laterally (unconfined condition with uniaxial vertical stress). The applied displacement history can thus be converted to a vertical strain, as plotted in Figure 8. The element is first elongated to a tensile strain of 1%, after which the strain is reversed and the element is shortened back to its original length and beyond.

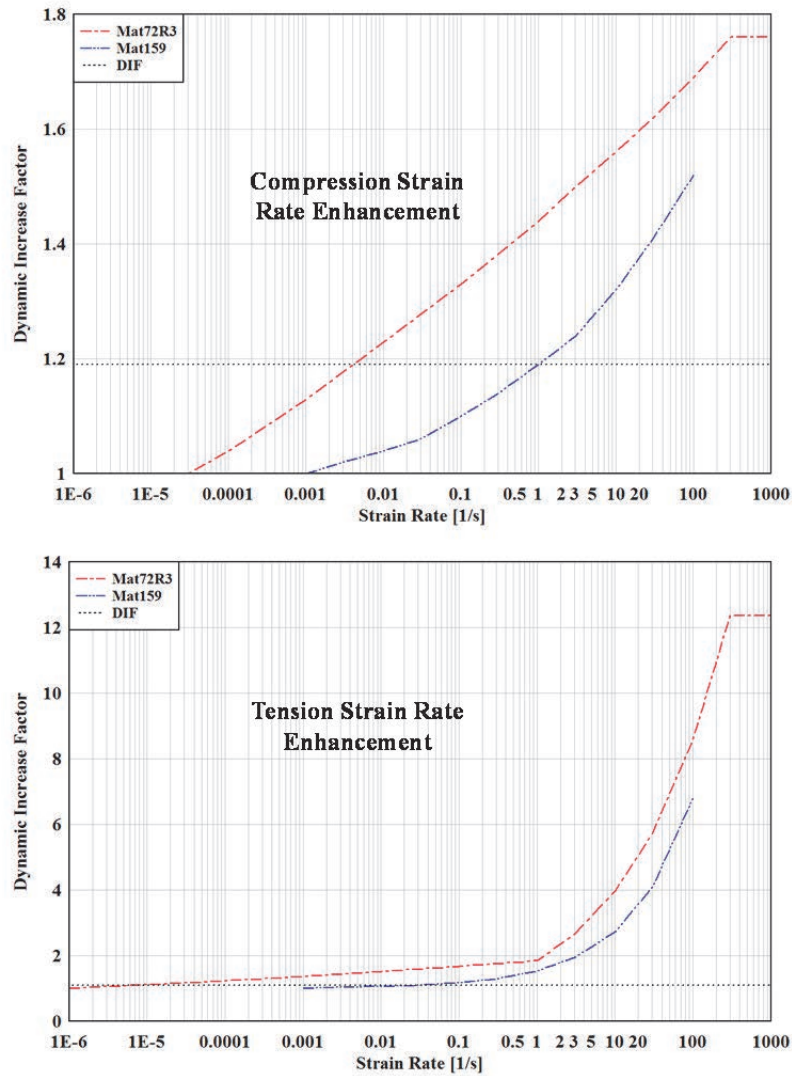


Figure 7 — Concrete Strain Rate Enhancement Curves Compared to DIF

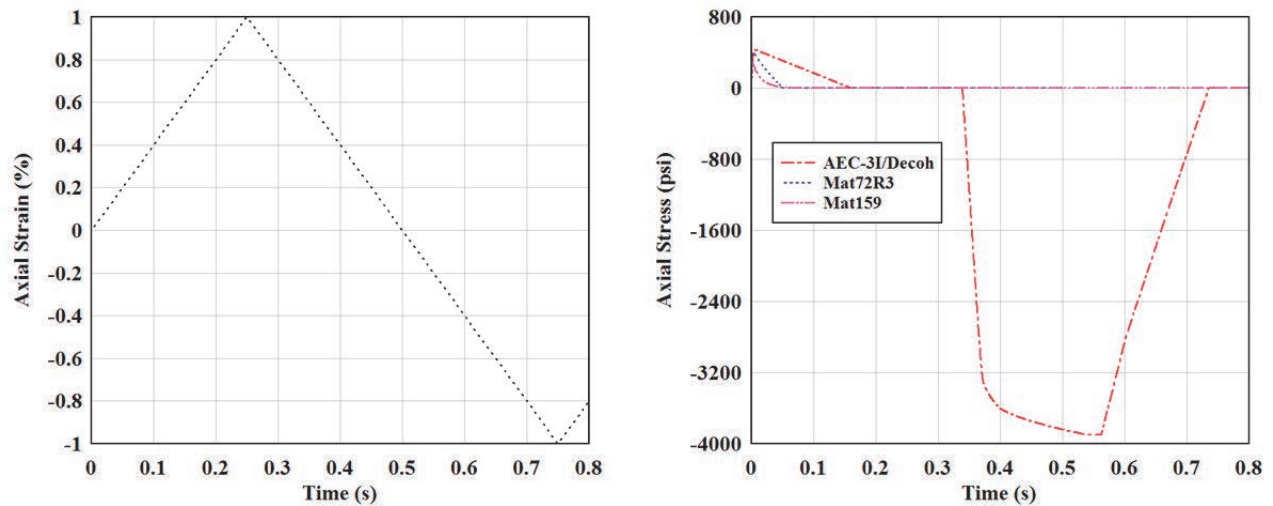


Figure 8 — Axial Strain and Stress from Single-Element Cyclic Loading Test

In a physical experiment of this sort, we would expect that the material will first carry tensile stress up to the point of failure, beyond which the stress should return to zero (with some softening) until the direction of loading is reversed. Upon reversal, the stress should remain zero until the extensional strain has been completely recovered, the crack has closed, and the element can begin to carry compressive stress up to the point of failure.

The results of this numerical experiment are plotted in the right-hand graph of Figure 8. The three material models all exhibit a comparable tensile strength, but their post-failure softening behavior is quite different. In approximate terms, the AEC-3I model exhibits roughly three times the failure energy (area under the stress-strain curve) as MAT72R3, which in turn exhibits roughly twice the energy of MAT159. The fracture energy in the AEC-3I decohesion model is a user-specified input, and has historically been defined to be five times higher than published fracture energies in order to match test results. MAT72R3 and MAT159 reach full crack formation (i.e., zero tensile stress) by 0.2% axial strain, while AEC-3I reaches that point at 0.6% axial strain.

Additional differences are observed upon cycling of the strain. Both MAT72R3 and MAT159 remain at zero stress, even as the compressive strain reaches and exceeds zero. This is due to the MAT72R3 and MAT159 models dropping the shear yield surfaces to zero strength once tensile fracture is experienced, so long as no confining pressure is applied (as in this test case). The MAT72R3 and MAT159 models are also isotropic with simultaneous tensile fracture occurring in all three directions. These models are thus unable to distinguish the orientation of the crack plane. AEC-3I, however, is able to not only pick up the compressive stress, but to do so only after closing of the gap formed during the extensional displacement. The AEC-3I model loads up to full unconfined compressive strength capacity, followed by loss of strength with continued forced displacement. It should also be noted that MAT159 includes an option that picks up compressive stresses when reversing the load direction from a tensile stress to a compressive stress; this option was not utilized in these calculations as it seemed to lead to other undesirable behaviors in the model.

SIMULATION RESULTS

The results from the simulations using the three concrete models are presented in the following sections. For each of the two tests, calculations were performed with AEC-3I using either static or DIF-enhanced concrete strength; using MAT72R3 and MAT159, calculations were performed with static, DIF-enhanced, and strain-rate dependent concrete strengths.

Centerline deflection histories

The primary response metric for a simply supported, one-way reinforced concrete slab is the displacement at the centerline, which is normally correlated to various damage states. The test result is derived from high-speed video footage of the slab's response, which has been sampled to produce the test records plotted.

The results for Test 1 are plotted in Figure 9, while those for Test 6 are in Figure 10. For each test, one graph is used to plot all the analysis results for a given material model (using various representations of the material's rate-dependent strength).

Considering the Test 1 results, we observe that the AEC-3I model predicts the response with excellent accuracy, not only with regard to the peak deflection (which is typically the primary interest) but also the post-peak rebound response. The frequency of the oscillatory response is also predicted very well. We note that the change in material strength (with and without the DIF) produces a negligible variation in response. MAT72R3, by comparison, exhibits a very large variability in response as a function of the type of rate enhancement. The highest response corresponds to the static properties, as we might expect; using the DIF produces an excellent match to the peak response; and using rate-dependent properties seems to make the model too stiff. MAT159 is much like AEC-3I in its relative insensitivity to the rate enhancement assumptions. The calculated responses are very similar from all three runs, and compare very well with the test in both initial peak and rebound phases.

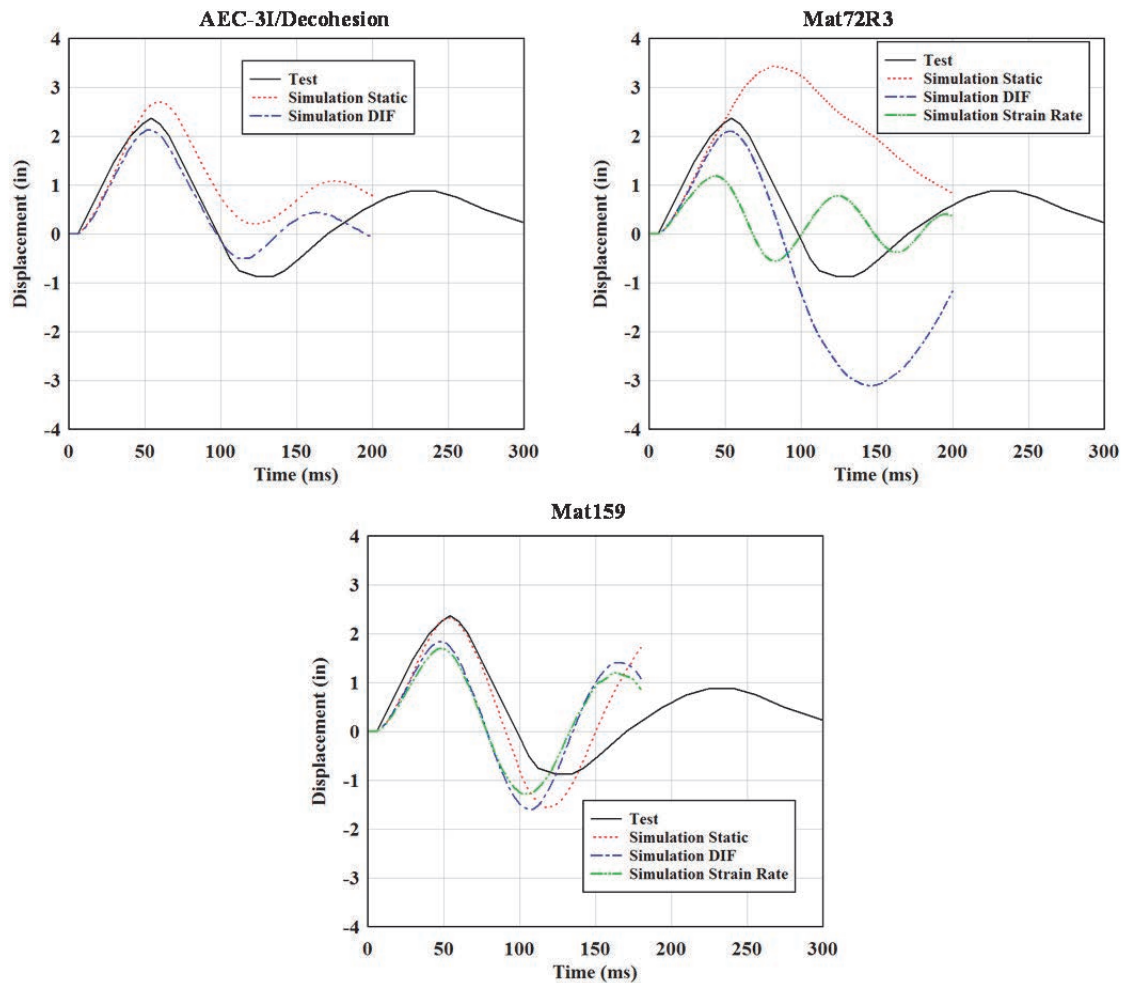


Figure 9 — Deflection Histories for Test 1

When considering Test 6, the same conclusions can be made with regard to the sensitivity of the response to various assumptions regarding rate enhancement. MAT72R3 shows the greatest sensitivity, while the others appear relatively insensitive. Once again, MAT72R3 with rate-dependent enhancement produces a response that is too low, with the slab appearing to be far too stiff.

In terms of the magnitude of deflection for Test 6, AEC-3I and MAT159 both underpredict the peak deflection; the latter model does so by about a factor of two. MAT72R3 overpredicts the response somewhat unless one is using rate-dependent properties, in which case it severely underpredicts. In terms of the waveform, MAT159 appears too elastic with post-peak oscillations being of nearly the same magnitude and frequency as the initial response. AEC-3I shows some decay post-peak, but the frequency is still high. MAT72R3 seems to produce the most realistic waveform shape in terms of the later time behavior, provided one is using the static or DIF properties (not the rate dependent).

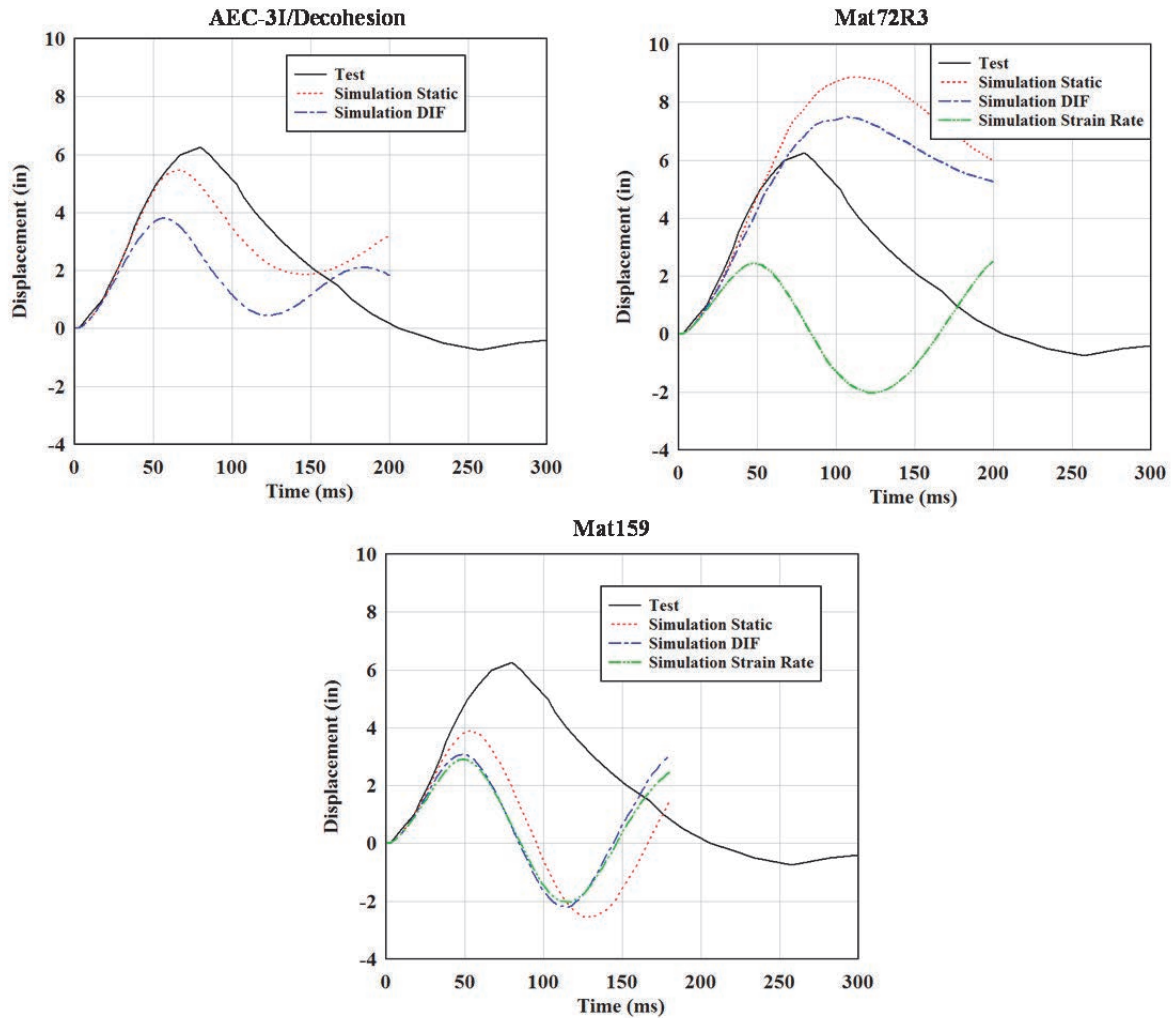


Figure 10 — Deflection Histories for Test 6

The frequency of the predicted response is similar to that observed in the measurements. Generally, those calculations that underpredict response also overpredict the natural frequency of post-peak oscillations, and vice versa. Those calculations that predict the peak response very well (e.g., AEC-3I for Test 1) also predict the frequency very well.

Response metrics

To facilitate more detailed quantitative comparisons among the models, it is helpful to use the relative error as defined by the following for the generic quantity x :

$$E = (x_{FEA}/x_{test}) - 1$$

Some quantities of interest include the peak positive deflection; the amplitude of the first post-peak oscillation (i.e., the peak positive minus the peak negative deflection); and the half-period (time of peak negative minus time of peak positive). These values were tabulated from the test record as well as the analyses, and converted to relative error metrics. We can then take the average of the absolute value of each error metric to obtain a single quantitative “score” for each of the material models. Using the absolute value is appropriate because it does not reward a model for erring high in some metrics and low in others. A summary of all these values is provided in Table 2 for Test 1

and Table 3 for Test 6. Note that for the case of MAT72R3 with static properties, the response was not calculated out to a point of peak negative displacement, thus the metrics for amplitude and half-period are listed as “N/A.” For the MAT72R3 case with DIF-enhanced properties, an approximate estimate of the peak negative deflection could be made from the available response history, and so those values are included in the tables.

Table 2 — Summary of Relative Errors for Material Models, Test 1

	AEC-3I, Static	AEC-3I, DIF	MAT72R3, Static	MAT72R3, DIF	MAT72R3, Rate dep.	MAT159, Static	MAT159, DIF	MAT159, Rate dep.
Peak pos. defl.	13%	-13%	42%	-13%	-50%	0%	-21%	-29%
Amplitude	-24%	-21%	N/A	58%	-45%	18%	3%	-12%
Half-period	-18%	-14%	N/A	23%	-52%	-14%	-22%	-26%
Average (abs. value)	18%	16%	N/A	31%	49%	11%	15%	22%

Table 3 — Summary of Relative Errors for Material Models, Test 6

	AEC-3I, Static	AEC-3I, DIF	MAT72R3, Static	MAT72R3, DIF	MAT72R3, Rate dep.	MAT159, Static	MAT159, DIF	MAT159, Rate dep.
Peak pos. defl.	-11%	-39%	44%	21%	-61%	-37%	-50%	-53%
Amplitude	-49%	-51%	N/A	-64%	-37%	-9%	-24%	-30%
Half-period	-55%	-64%	N/A	-20%	-58%	-58%	-66%	-65%
Average (abs. value)	38%	51%	N/A	35%	52%	34%	47%	49%

The average of the absolute value of the three error metrics for each test is plotted in Figure 11. Additionally, the relative error on peak positive deflection (the metric of greatest interest) from the DIF models only (the only material option common to all three models) is plotted separately in Figure 12.

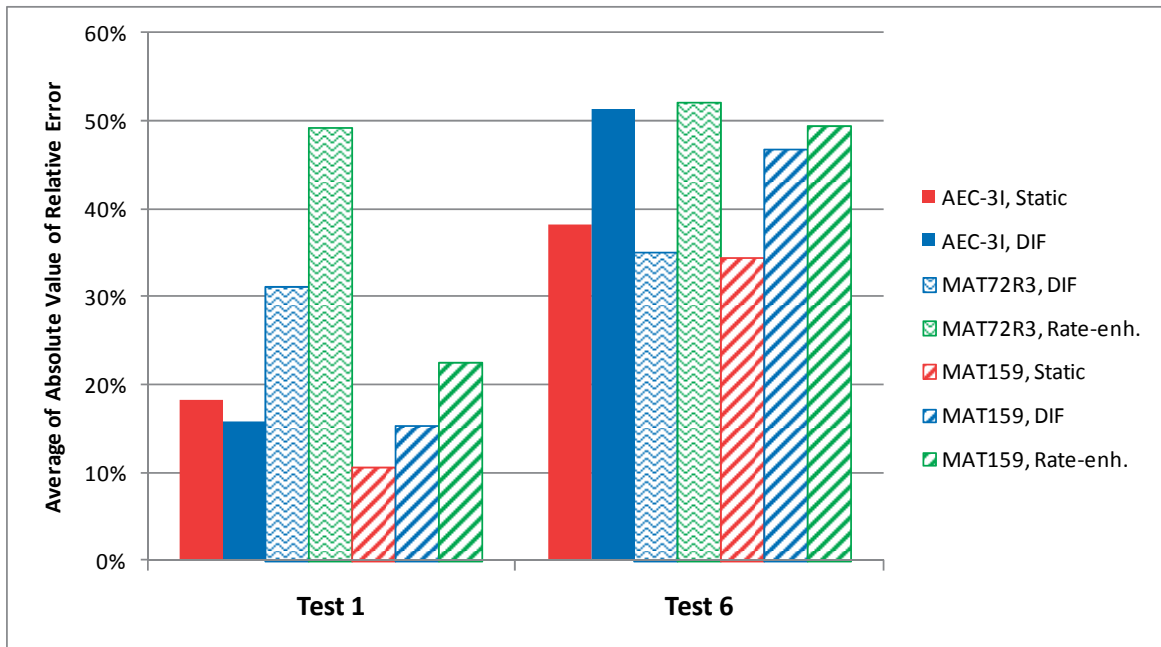


Figure 11 — Average Error Metrics for All Models and All Tests

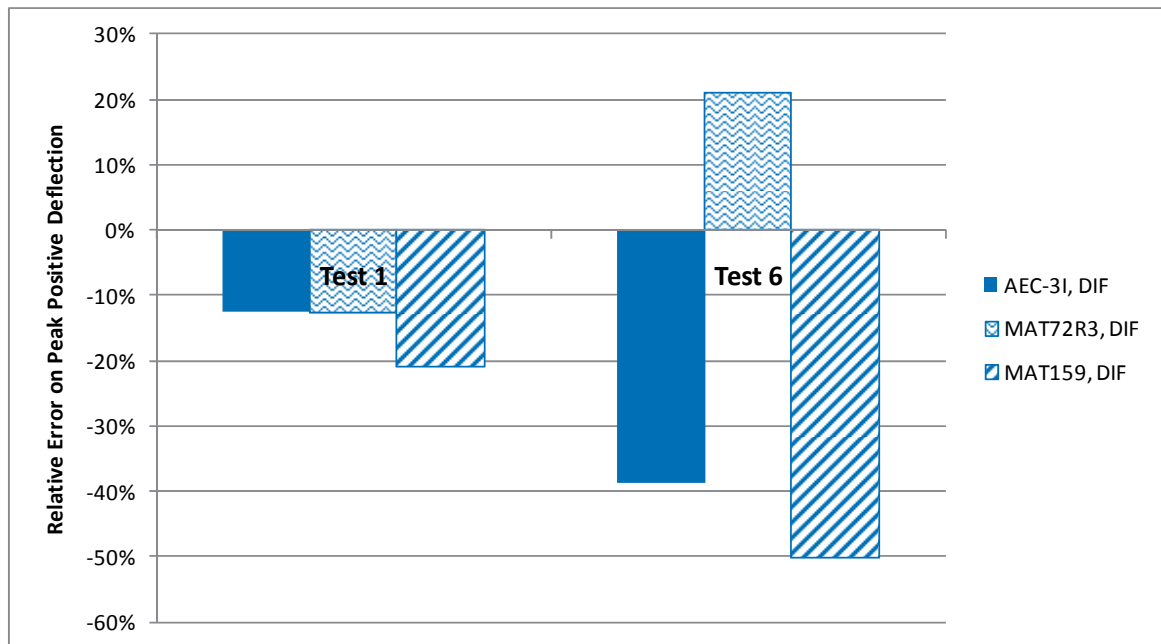


Figure 12 — Relative Error on Peak Positive Deflection from DIF Models Only

A number of key observations may be drawn from the data presented herein. First, generally speaking and looking across all the runs, the average error is greater in test 6 than in test 1. As we had observed earlier, the test 1 deflections compared much more favorably with the test results than those from test 6. The rebound response is also better represented in the test 1 calculations. Thus, the models are more reliable at predicting small magnitude response than at higher magnitudes.

Second, when comparing material models, there is no obviously preferable model if one is equally interested in all three metrics (as is assumed by taking the average of the absolute values). MAT72R3 is clearly the least accurate for Test 1, but it does as well as the others in Test 6. And AEC-3I is comparable to MAT159 for both tests.

Third, if one is only interested in peak positive deflection, and if one uses the DIF models, MAT72R3 appears to be the best predictor when including both tests. AEC-3I and MAT159 underpredict both tests, rather significantly in the case of test 6. MAT72R3 is a little low for test 1 and a little high for test 6.

Fourth, it is surprising to note that the addition of more sophisticated methods of addressing rate enhancement in the concrete does not result in greater predictive accuracy. When considering a given model for a given test, the ones using static properties produced the least average error (red bars in Figure 11) while the rate-dependent model produced the highest error (green bars in Figure 11). Rate sensitivity of concrete strength has been documented over hundreds of individual sample tests and must be accepted as being physical. Clearly, in these analyses, the rate sensitivity is masking other non-physical behaviors in the models. In other words, these results should not be used to support the omission of rate sensitivity from the concrete models.

Fifth, as had been seen previously, the AEC-3I and MAT159 models are the least sensitive to variations in how one represents the rate sensitivity of the concrete.

Damage and cracking

Post-test photos of the damage to the slabs from Test 1 and Test 6 were shown earlier in Figure 5 (non-loaded side). Since all three material models track the damage state of the material, and since the tensile damage is clearly visible as cracks, it is helpful to observe the damage fringe plots (at late time) as shown in Figure 13 and Figure 14. The plane of symmetry is visible on the right-hand side of each half-slab. The color fringes represent the magnitude of damage in the visible elements, ranging from blue for undamaged to red for the progressively higher levels of damage (cracking).

The fringe plots allow us to gain further insight into the relative merits of these three material models. In the case of AEC-3I, a few cracks occur near the centerline, which agrees quite well with the observations from the test. MAT72R3 and MAT159, however, are less consistent with the observations; the former shows large regions of uniformly severe damage, while the latter predicts a far larger number of cracks than were observed experimentally. In terms of its ability to forecast the damage and crack patterns of the slab, AEC-3I appears to be clearly preferable.

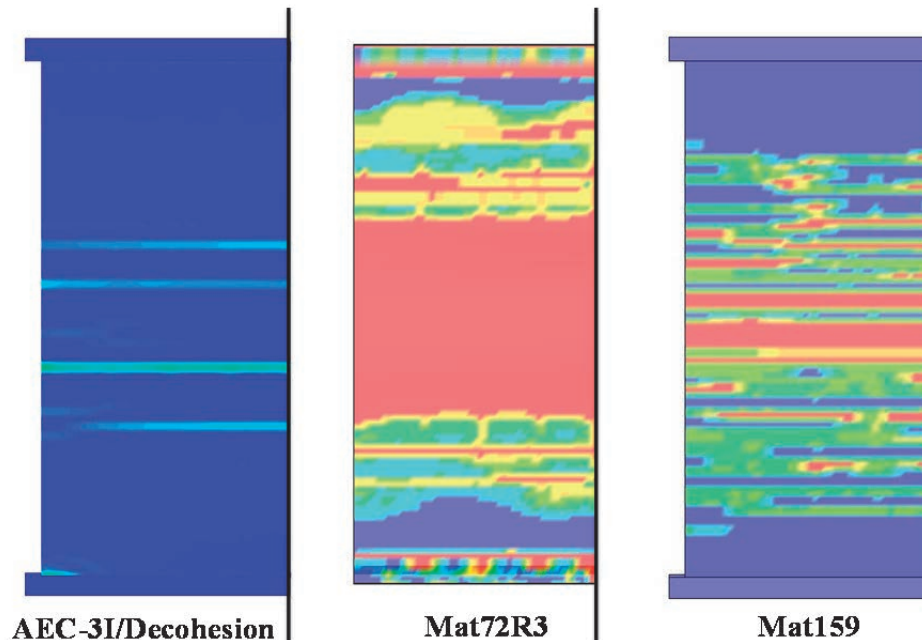


Figure 13 — Damage Fringe Plots from the DIF Simulations of Test 1

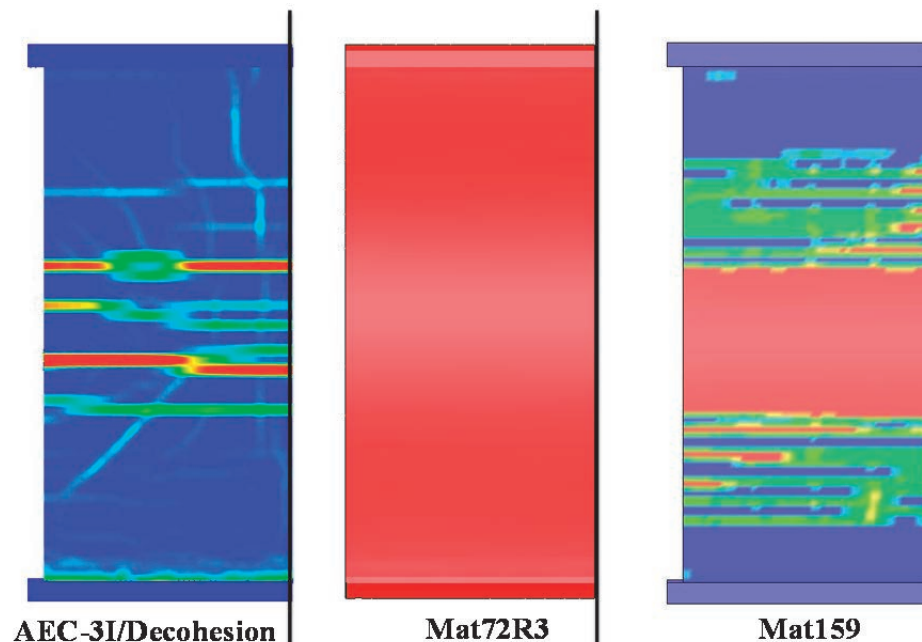


Figure 14 — Damage Fringe Plots from the DIF Simulations of Test 6

CONCLUSIONS

We may summarize the primary conclusions derived from this study as follows:

- Surprisingly, attempts to model the dynamic strength of concrete do not necessarily produce clear improvements in predictive ability. The most sophisticated such model, MAT72R3 with rate-dependent properties, proved to be overly stiff and strong and to underpredict response by a factor of about two.

- Looking at all three material models utilizing the DIF on the strength, the best predictor of positive peak deflection is MAT72R3. However, when considering the uncertainty in response due to assumptions on how to represent the rate sensitivity, the most reliable and insensitive result is that from MAT159.
- All models tend to predict the displacement waveform with reasonable accuracy, though they tend to show the slab as being stiffer (higher frequency) than measured in the tests.
- Overall, it is encouraging to note that all the models can predict the peak positive deflection (as well as most other metrics) within a factor of 2. For a test in the low response regime, that factor would be considerably lower. Thus, use of finite element models to predict reinforced concrete element response continues to be a useful tool for designers everywhere, but within the uncertainties indicated by this and other similar studies.

ACKNOWLEDGMENTS

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Effects of High-Strength Materials on Blast Response of Reinforced Concrete Panels

Stephen D. Robert, Carol F. Johnson, and Stanley C. Woodson

Synopsis: Concrete structures designed to meet blast criteria often require substantial increases in structural system size, weight, and cost when using conventional materials, but high-strength materials may offer a way to mitigate these increases while achieving desired performance levels. The U.S. Army Engineer Research and Development Center (ERDC), Vicksburg, Mississippi, investigated the effects of using High Strength Low Alloy vanadium microalloyed steel (HSLA-V steel) reinforcing bar coupled with high-strength concrete as a structural system. The combination of high-strength portland cement concrete (HSPCC) and HSLA-V steel can improve a component's ability to satisfy a given blast resistance criteria while allowing a more efficient structural design than can be achieved with conventional materials. Vanadium is widely used as an alloying element in steel production. Micro-alloying vanadium with steel reinforcing bar contributes to higher yield strengths than can be achieved with traditional Grade 60 rebar, without compromising ductility or formability. The investigators performed dynamic testing of one-third scale reinforced concrete panels using the ERDC Blast Load Simulator (BLS). The panels consisted of double-mat conventional Grade-60 rebar or HSLA-V steel rebar in combination with 4-ksi (27.6 Mpa) or 15-ksi (103 Mpa) concrete. Tests were performed using blast loads to determine the performance of simply-supported concrete panels constructed from different combinations of reinforcing steel and concrete. Measured properties included center-span deflection, average blast pressure, and average impulse. The 4-ksi (27.6 Mpa) concrete in combination with HSLA-V steel rebar provided the best alternative to the use of only high-strength materials, but provided a lower level of blast protection. Permission to publish this paper was granted by the Director of the Geotechnical and Structures Laboratory and the U.S. Army Research Laboratory.

Keywords: high strength concrete; HSLA-V steel; reinforced concrete wall panels; dynamic blast testing

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INTRODUCTION

The U.S. Army and other government agencies are evaluating wall systems that are more cost efficient and provide higher levels of protection than current wall systems to be incorporated into new construction. Higher strength materials are being investigated to determine their effectiveness in improving the performance of wall systems subjected to explosive effects. The ERDC-Vicksburg Blast Load Simulator (BLS) can replicate the pressure and impulse loading resulting from detonation of an explosive charge. Ten subscale reinforced Portland cement concrete panels were constructed using different combinations of either high-strength or conventional concrete with either high-strength low-alloy Vanadium (HSLA-V) or conventional Grade 60 reinforcing bars. The panels were then load tested dynamically in the BLS to investigate the contribution made by each material to the blast resistance of the reinforced concrete panels. The pressure and impulse values were also increased for some of the load tests to explore the reserve capacity of the tested reinforced concrete panels.

BACKGROUND

These HSLA-V steel reinforcement experiments are part of the Vanadium Technology Program funded by the Army Research Laboratory (ARL) to investigate the potential benefits of utilizing HSLA-V steel in structural applications for the U.S. Army and, if proven beneficial, eventually in commercial use. Subscale reinforced Portland cement concrete panels were tested in ERDC-Vicksburg's quasi-static water chamber. The quasi-static experiments evaluated panels with the same material combinations, panel dimensions, and relative support conditions as those tested in the dynamic loading BLS experiments. The quasi-static experiments were conducted to determine the load vs. deflection curves for each material combination. These curves are referred to as resistance functions, and are presented in a previous publication [Robert, 2010]. The U.S. government-owned computer program, Wall Analysis Code (WAC) [Slawson, 1995] was the SDOF software used in this study. The goal was to determine the standoff and charge weights required to induce significant damage to the reinforced concrete panels. The quasi-static experiments also assisted in determining the material combinations and reinforcement details (either single or double mat reinforced panels) to be used in the dynamic experiments. It was determined that the single mat reinforced panels did not perform as well as doubly-reinforced panels in producing a symmetrical response. The desired response was for the maximum deflection to occur at the midspan of the panel [Robert, 2010].

The selection of materials for both the quasi-static and dynamic experiments was based on results of a previous case study that investigated the effect of concrete strength in conjunction with strengths of the conventional and HSLA-V steel rebar [DiPaolo, et al., 2005]. The quasi-static and dynamic experiments were conducted at one-third scale to accommodate ERDC-Vicksburg's testing facilities. A blast resistant reinforced concrete design of 4-ksi (27.6 MPa) concrete with Grade-60 reinforcing steel was established as the basis of comparison for determining the effectiveness of using higher strength materials. Two concrete mix designs were included in both the quasi-static and dynamic experiments. Concrete with a 28-day design compressive strength of 4,000 psi (27.6 MPa) and concrete with a 56-day design compressive strength of 15,000 psi (103MPa) were used in constructing the reinforced

concrete panels. The 15,000 psi (103 MPa) concrete is referred to as High-Strength Portland Cement (HSPC) concrete. It was developed in the late 1980s as a result of a research program on development of high-strength concrete mixtures [Neeley, et al., 1991]. The concrete mixes for all of the reinforced concrete panels were prepared and placed at the ERDC Geotechnical and Structures Laboratory (GSL) using Type I Portland cement. The principal flexural reinforcing steels used in the experiments were No. 3 A615 Grade 60 steel reinforcement and No. 3 HSLA-V steel reinforcement with minimum yield strengths of 60,000 psi and 75,000 psi, respectively. Vanadium is an optional alloying element in Grades 60, 75, and 80 ASTM specifications rebar that provides an increase in strength without compromising the ductility and bendability [HSLA-V, 2009]. The high-strength HSLA-V rebar used in these experiments contained a higher percentage of vanadium than the conventional Grade 60 rebar. A comparison of the stress v. strain properties of conventional Grade-60 and HSLA-V rebar is shown in Figure 1.

EXPERIMENTAL SETUP

Experimental Matrix

The components that formed the experimental matrix for this series of dynamic reinforced concrete experiments were conventional Grade-60 70-ksi (483 MPa) steel rebar (RS), conventional 4-ksi (27.6 MPa) Portland cement concrete (RC), HSLA-V 83-ksi (572 MPa) steel rebar (HS), and high strength 15.4 ksi (106 MPa) concrete (HC). Three panels for each combination that involved HSLA-V steel were tested because the effectiveness of this steel in combination with various concrete strengths was of primary interest. Two panels were fabricated for each of the other material combinations because of the limit on the number of panels to be tested. All panels consisted of double mat rebar cages with #3 reinforcing steel. The experimental matrix is shown in Table 1.

Specimen Details

The one-third scale, simply-supported, one-way reinforced concrete panels were 4 in. (0.102 m) thick to replicate a 12-in. (0.305 m) thick wall. The principal reinforcing steel was placed at 4-in. (0.102 m) on-center and the transverse steel was placed at 12 in. (0.305 m). Each panel was 64 in. by 33.75 in. (1.63 m by 0.857 m) and had a clear span of 52 in. (1.32 m) in the test frame, which replicates a simply supported 13-ft (3.96 m) tall wall. Concrete cover on the front and back face was $\frac{1}{2}$ in. (12.7 mm) and the edge cover was $\frac{3}{4}$ in. (19.1 mm). To limit the concrete damage at the panel edges, a principal steel rebar was added at a 2-in. (50.8 mm) spacing near the two edges as shown in Figure 2. Figure 2 depicts a plan view of the panel construction details, while Figure 3 provides a side view of reinforced concrete panels.

Blast Load Simulator

The Blast Load Simulator (BLS) (Figure 4) was used for dynamic load testing of the reinforced Portland cement concrete panels. The BLS is designed to simulate blast waveforms for explosive yields equivalent to up to 20,000 lbs (9070 kg) of TNT at a peak reflected pressure of up to 80 psi (0.552 MPa) and a peak reflected impulse of up to 1,100 psi-msec (7.58 MPa-msec). The BLS can accommodate a target 71 in. (1.80 m) tall by 53 in. (1.35 m) wide. The control room allows spectators to witness the experiment first hand with immediate feedback from high speed videos and a state of the art data acquisition system.

The BLS facility consists of a cylindrical driver system, a transition system, and a target fixture all housed in an underground enclosure to contain the blast pressures. The driver system consists of a cylindrical pressure vessel, a push-pop device, and a mechanical striker/diaphragm system. The driver contains the high-pressure source that is released into the transition system by rupturing a series of diaphragms. The high pressure source is created by pumping air or an air/helium mixture into the pressure vessel. The gases are confined in the vessel by a predetermined number of steel or aluminum diaphragms. When the desired pressure is reached, a mechanical striker is activated, forcing the diaphragms to rupture and releasing the compressed gases into the transition system, forming the shock pulse. As the shock propagates downstream through the transition system consisting of the transition cone, expansion rings, and cascade, it expands and is shaped into the desired waveform. The transition

cone consists of three consecutively larger rings attached to a sled that allows the shock pulse to freely expand at each transition. One negative phenomenon created by most typical shock tubes, referred to as shock reverberations, is multiple shock pulses that are created at the target when the primary shock reflects off the target, travels the length of the tube, reflects off the driver, travels back down the tube, and reloads the target. The gaps in the transition cone created by the three expanding rings of ERDC's BLS serve as vents which allow this rebound load to vent out of the BLS device into the underground enclosure, therefore, reducing, delaying, or in some cases removing this rebound load completely. The shock pulse travels from the transition cone into the expansion rings and cascades to the target face. The targets are mounted in the front face of the target vessel which contains all the debris that could be generated in the experiment.

A specialized frame was fabricated and bolted into the BLS target vessel to allow for simple support conditions of the reinforced concrete panels (Figure 5). The exterior frame consisted of 6-in. (0.152 m) by 8-in. (0.203 m) tube steel on the top and bottom of the frame, which acted as the bearing surface for the back face of the panel, the panel face opposite of the pressure wave impact. On the front side of the panel, an interior frame with 3-in. by 3-in. (76.2 mm by 76.2 mm) tubing near the top and bottom of the frame acted as the front side support, which prevented the panel from falling out of the target vessel during testing. Thin 1/8-in. (3.18 mm) rubber was placed on the front and back bearing surfaces of the frame. The rubber strips eliminated any slack that might have existed between the panel and the frame and also provided some cushion to prevent crushing at the supports. When the exterior and interior frames were bolted together with high strength bolts, a panel clear span of 52 in. (1.32 m) existed. All areas of potential small gaps around the frame were sealed off to prevent the pressure wave from leaking into the target vessel, which could have affected the wall response.

The BLS allows for the recording of air pressure, deflection, and high speed video. Nine pressure gages were used, with one gage inside the target vessel, two external pressure gages mounted in the target vessel, and six external gages mount in the left and right sides of the target frame at the mid-span and quarter-points (Figure 6). The internal gage measured the air pressure that entered the target vessel. The other eight gages provided a measurement of the average air pressure to which the specimen was subjected. The mid-span deflection of the reinforced concrete panels was measured with a laser and an accelerometer (Figure 7). Four high speed videos, two side views and two rear views, captured the response of the wall. The high speed videos allowed for close investigation of the wall response at different time intervals.

EXPERIMENTAL RESULTS

The results from the dynamic reinforced concrete experiments are shown in Table 2. The peak pressure and impulse values in Table 2 are the averages of the data recorded by the eight pressure gages that were mounted to the front of the target vessel and frame. The intended applied pressure level was increased with corresponding increases in strength. For example, Panels 7 and 9 were tested with approximately the same loading, while Panels 6 and 8 were tested at higher loads. Panel 9 utilized conventional materials and served as the baseline for the experiments. Panel 3 was tested at the highest loading, but incurred significantly less damage than panel 9. The highest loading was required on Panel 3 in order to induce similar deflections as occurred for Panels 4 through 8.

The side of the reinforced concrete panel facing the blast wave will be referred to as the front face of the panel. Back-face and front-face scabbing, along with significant cracking occurred during these experiments. Higher levels of scabbing were observed at larger deflections. In the case of large deflections, rebar was exposed and larger cracks formed. The high strength concrete panels tended to have a large amount of front face scabbing but held together relatively well on the back face. Figures 8, 10, 12 and 14 show the pressure and impulse loadings applied to Panels 3, 9, 5 and 6. Post-test photos of the back face of Panels 3, 9, 5 and 6 are in Figures 9, 11, 13 and 15, respectively. Figures 8 and 9 and Figures 10 and 11 present the blast loading time-history and post-test damage of Panels 3 and 9 respectively, which compare the performance of the high-strength material panel (Panel 3) to the baseline, conventional material panel (Panel 9).

ANALYSIS

Post-test Evaluation and Analysis

Initially the panels were in a one-way simply-supported state, but the ends became somewhat restrained as the panel deflected. The panels were able to slide minimally until the support rotation became too great, at which point a partially-restrained support condition existed. The pressure and impulse data show that with small changes in pressure and/or impulse, the panel response can change significantly. These panels withstood very large loads that were comparable to 30,000 lbs (13600 kg) of TNT equivalent at standoff of 225 ft (68.6 m). The pressure and impulses were converted into explosive loadings using ConWep [Hyde, 2003].

The data from these experiments demonstrate the potential role of higher strength materials utilized in reinforced concrete design. The panels designed with high-strength concrete and HSLA-V steel deformed less and withstood larger loads than any of the other panels. The data also show that the use of high-strength steel alone can play an important role in improving the blast resistance of a structure. Panels 6, 7, and 8 appeared to perform well even though 4-ksi (27.6 MPa) concrete was used. Panels 1 and 2, which were high strength concrete panels reinforced with HSLA-V steel rebar experienced 20 percent less deflection than Panels 4 and 5, which were high-strength concrete reinforced panels with Grade-60 rebar. A 45 percent reduction in deflection was observed in Panels 6, 7, and 8, which were conventional strength concrete with HSLA-V steel rebar when compared to Panels 9 and 10, which were conventional strength concrete with Grade-60 rebar. High strength concrete panels with HSLA-V steel rebar sustained the least damage and survived the greatest loading in these experiments. However, it is not known from these experiments what combination of concrete strength and high-strength steel will provide the ultimate amount of protection and efficiency. Additionally, the use of shear stirrups can add stability to a panel, but were not included in this study.

Post-test Single-Degree-of-Freedom (SDOF) Analysis

A post-test single-degree-of-freedom (SDOF) analysis was conducted for each of the ten reinforced concrete panels. The experimentally-generated pressure-impulse curve for each experiment that best matched the average pressure and impulse were used in the SDOF analyses, along with the material properties and construction details for the corresponding reinforced concrete panel. The one-way fixed support condition provided mid-span results between 0.1 and 0.2 in. (2.54 and 5.05 mm). These results are well below the results shown in Table 2. One-way simple end supports were also modeled in the SDOF analysis, and Table 3 provides the mid-span deflection for that analysis as well as the mid-span deflections recorded by the laser and accelerometer during the testing of Panels 1 through 10. Although some restraint may have developed at the supports, it appears from the data in Table 3 that the dynamically tested reinforced concrete panels behaved primarily as simply-supported one-way panels. Mid-span deflections predicted by the SDOF analyses were either under or over (depending on the modeled end support conditions) the measured deflections, but there was also some variance in the accuracy of the laser recording the mid-span deflections due to effects of the shock wave. From the analyses, it can be concluded that the SDOF method is a useful tool in predicting the response of reinforced concrete panel designs that include higher strength materials. Material strain-rate effects can have a significant effect on the dynamic response of reinforced concrete components. Strain-rate effects were not included in this study, but may have contributed to the difference in the data and the SDOF results.

CONCLUSIONS

Reinforced concrete walls designed with HSLA-V steel rebar and high-strength concrete can provide a high level of protection for predefined threats. An increased blast resistance, when a prescribed level of protection or support rotation is required, can be gained by constructing reinforced concrete walls with either HSLA-V steel or high-strength concrete. For a predetermined standoff, reinforced concrete walls designed using high-performance materials can withstand a larger charge weight than conventional materials. If a predetermined threat and level of protection has been established, using high-performance materials may allow a designer to decrease the

reinforcement ratio and/or the wall thickness when compared to conventional materials. However, designers must give consideration to shear effects and the potential need for stirrups when optimizing a structural design.

RECOMMENDATIONS

In the design, execution, and analysis of the quasi-static and dynamic BLS experiments, the effect of the material properties of high strength materials on the blast response of reinforced concrete panels was investigated. The experimental results show that the compressive strength of high-strength concrete is needed to balance the tensile strength of high-strength reinforcement. Determining the optimum range of concrete strength for use in such designs was not a part of this study, but experimental and/or numerical research in this area is recommended to provide input to design manuals and building codes. ASTM standards for Grade 80 rebar currently exist, and Grades 75 and 80 rebar are being considered in reinforced concrete design. Studies need to be conducted on the use of Grades 75 and 80 rebar in reinforced concrete design to ensure that the current design manuals and building codes capture the mechanics and material interactions that will occur, and to validate the design specifications and code requirements for higher strength reinforcement.

ACKNOWLEDGEMENTS

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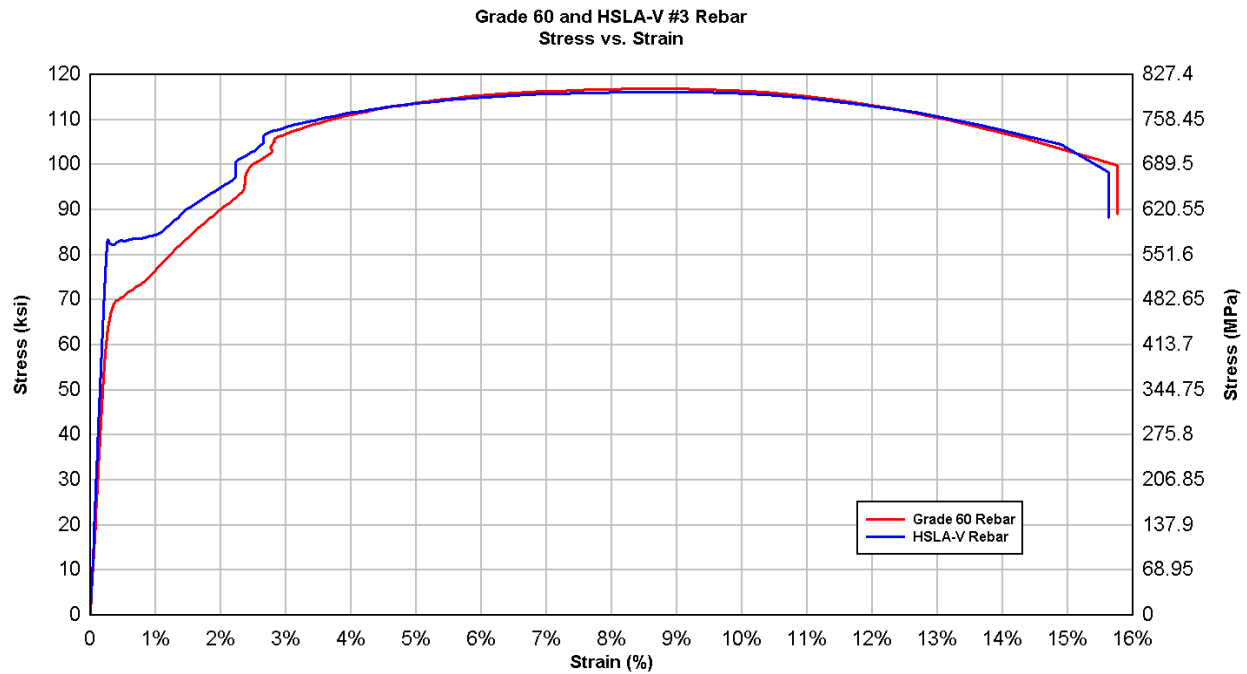


Figure 1 – Engineering Stress vs. Strain of Vanadium and Conventional rebar

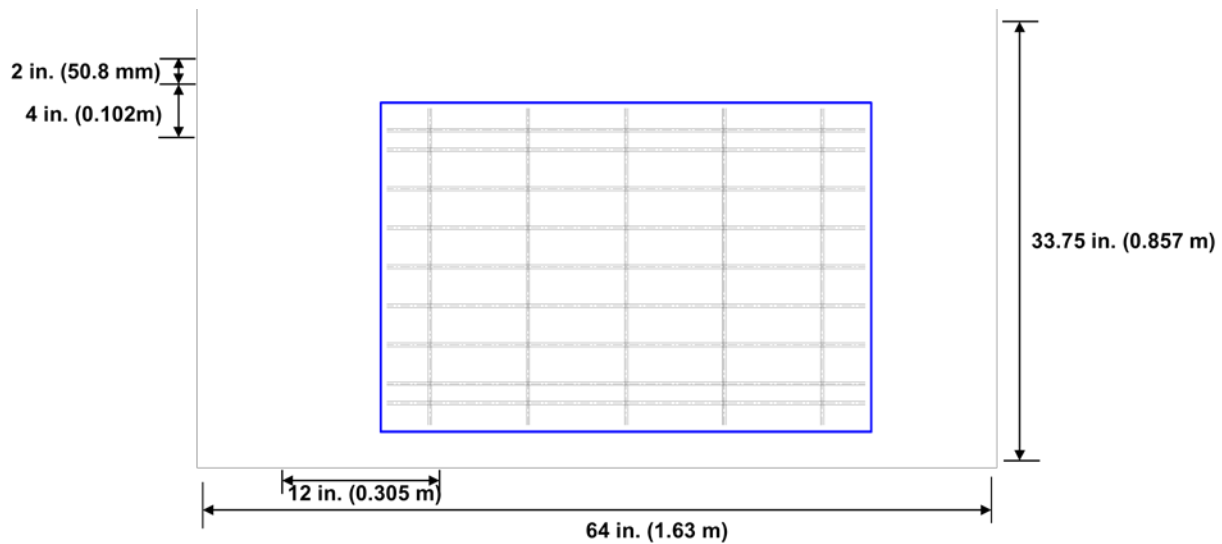


Figure 2 – Panel Construction Details (Plan View)

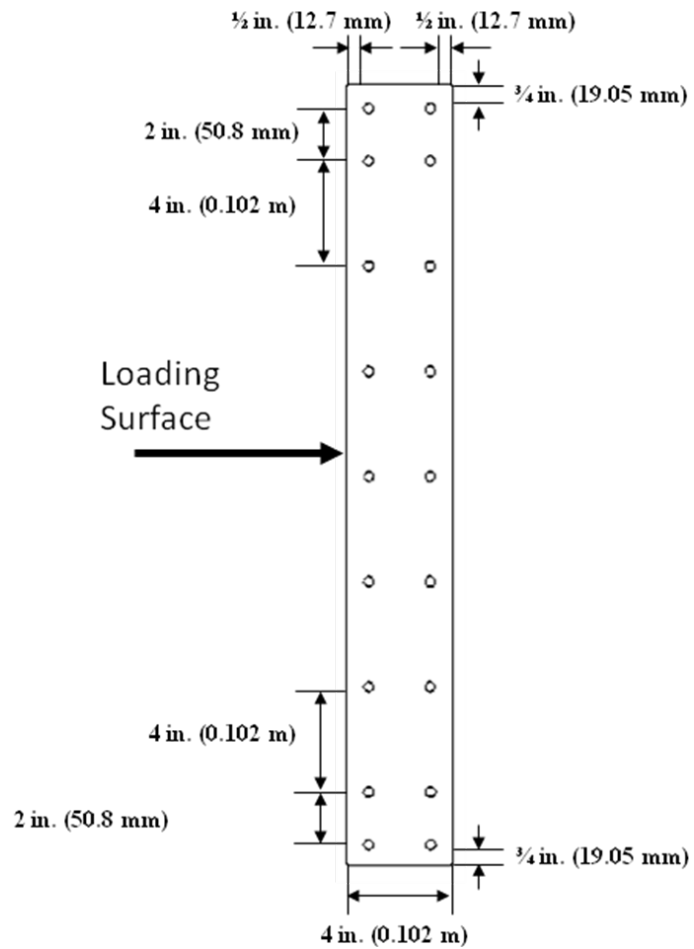


Figure 3 – Side View of Reinforced Concrete Panel

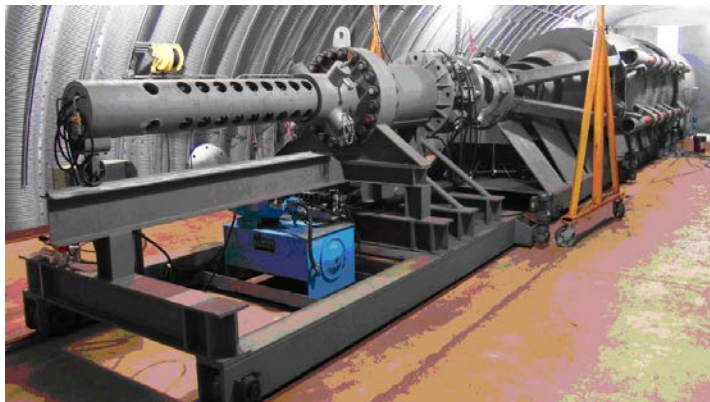


Figure 4 – ERDC-Vicksburg's Blast Load Simulator



Figure 5 – Target Frame

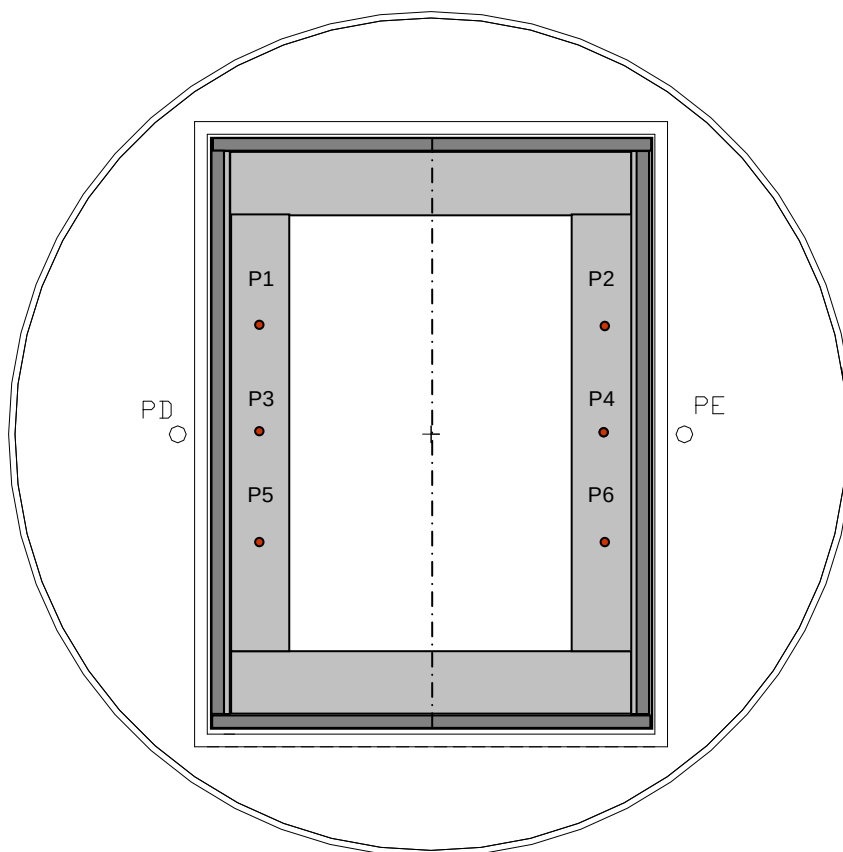


Figure 6 – Instrumentation Layout for the Pressure Gages in the BLS Experiments

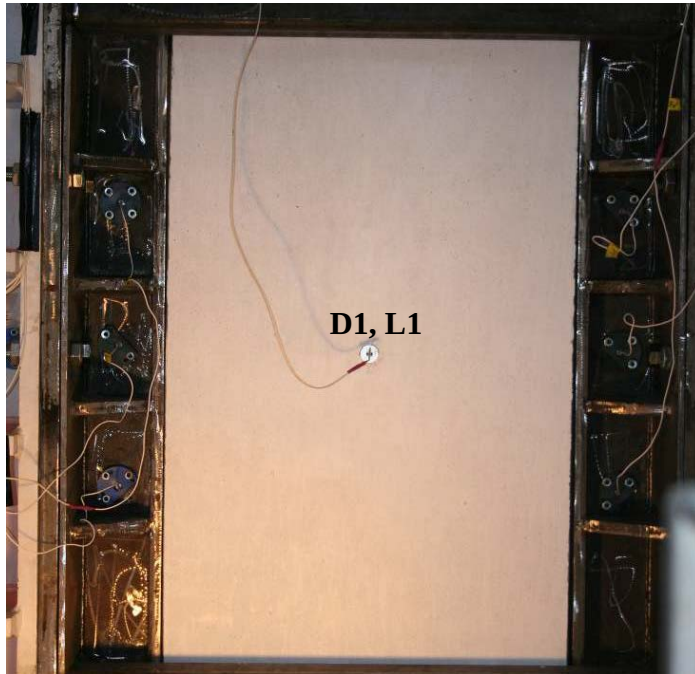


Figure 7 – Instrumentation Layout to Measure Deflection in the BLS Experiments

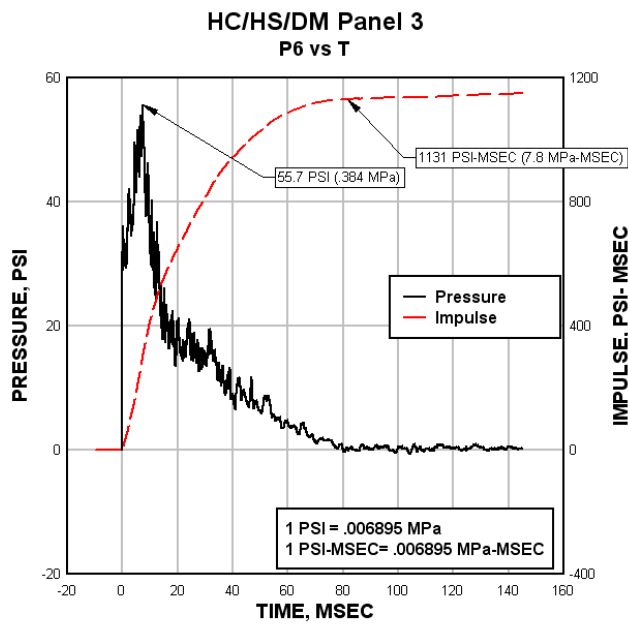


Figure 8 – Panel 3 Pressure and Impulse

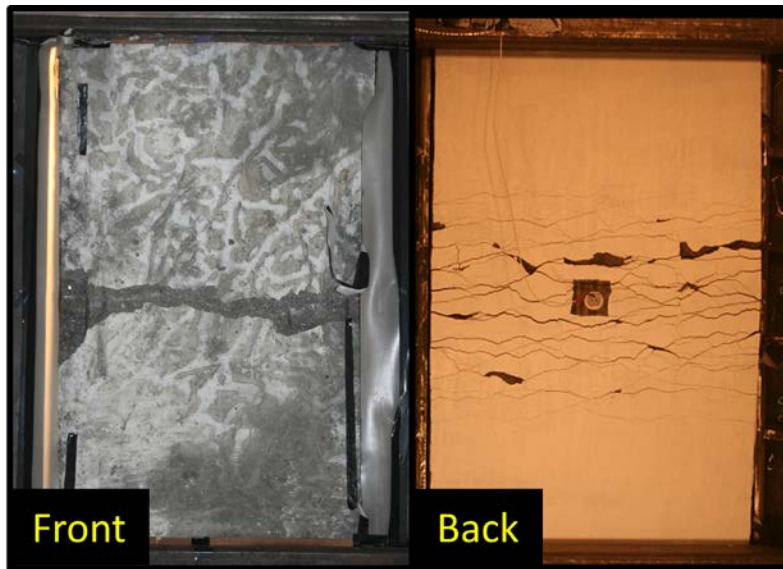


Figure 9 – Front and Back Surface of Panel 3

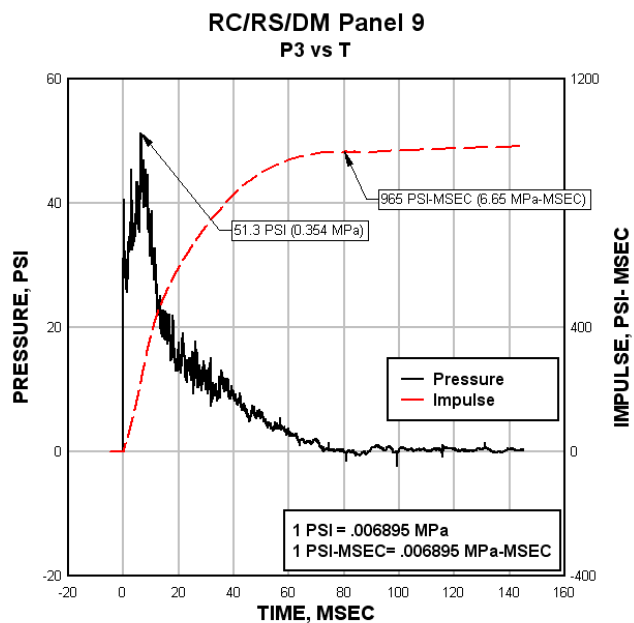


Figure 10 – Panel 9 Pressure and Impulse

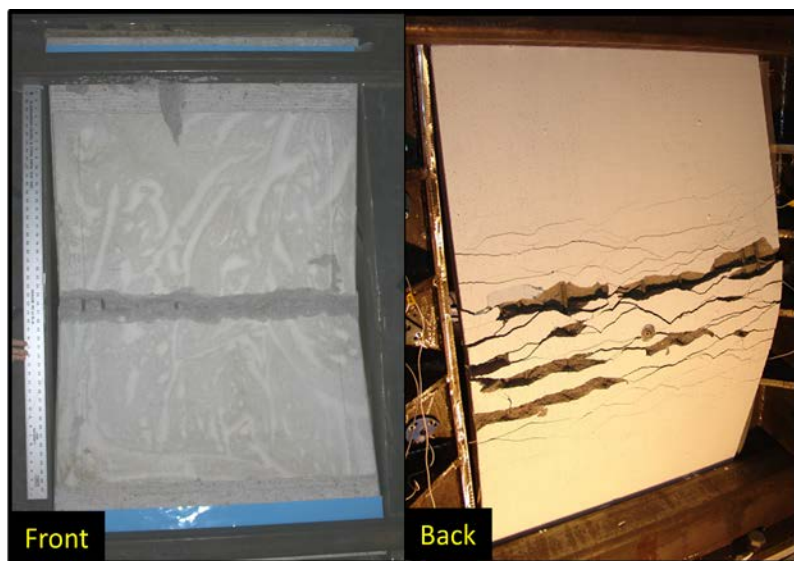


Figure 11 –Front and Back Surface of Panel 9

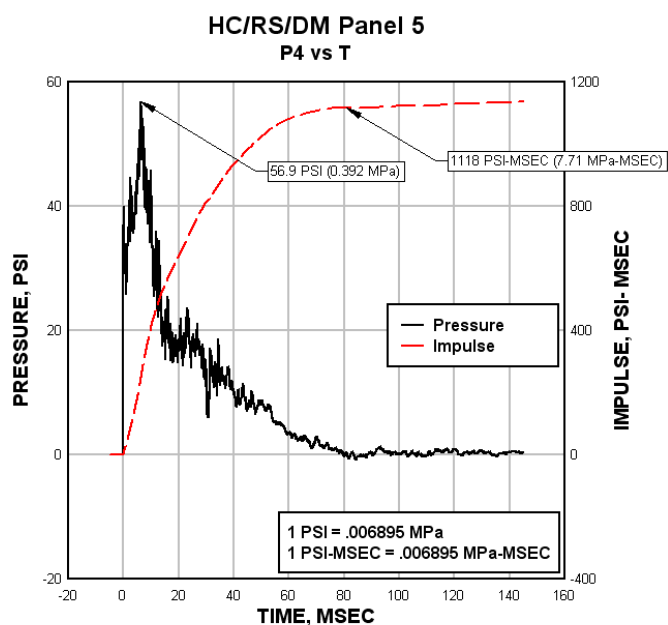


Figure 12 – Panel 5 Pressure and Impulse



Figure 13 – Front and Back Surface of Panel 5

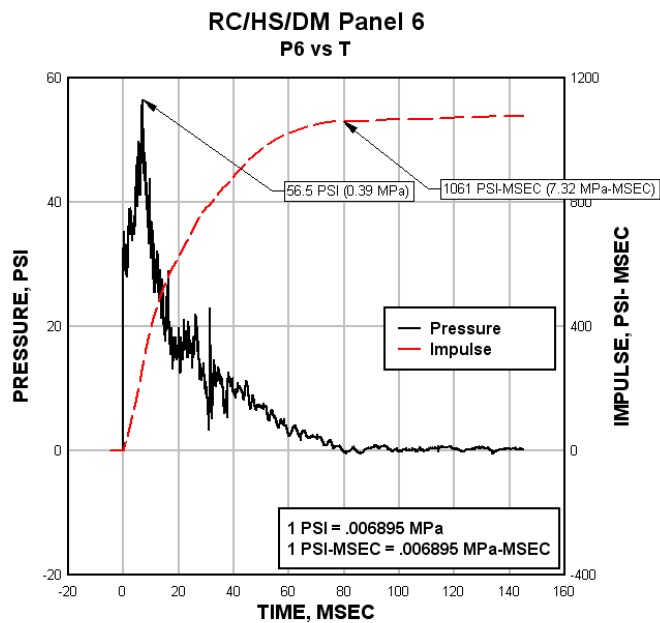


Figure 14 – Panel 6 Pressure and Impulse



Figure 15 – Front and Back Surface of Panel 6

Table 1 – Experimental Matrix

Panel	HC	RC	HS	RS
1	×		×	
2	×		×	
3	×		×	
4	×			×
5	×			×
6		×	×	
7		×	×	
8		×	×	
9		×		×
10		×		×

HC – High-Strength Concrete

RC – Conventional Strength Concrete

HS – HSLA-V Steel Rebar

RS – Conventional Steel Rebar (Grade 60)

Table 2 – Experimental Results

	Peak Reflected Pressure and Impulse Averages				Deflection			
Panel ID	Pressure (psi)	Pressure (MPa)	Impulse (psi-msec)	Impulse (MPa-msec)	D-Laser (in)	D-Laser (m)	D-Acc (in)	D-Acc (m)
HC/HS 1	55.8	0.385	1120	7.73	4.80	0.122	3.92	0.100
2	56.2	0.387	10900	7.50	5.20	0.132	4.80	0.122
3	57.2	0.395	1130	7.78	6.00	0.152	5.50	0.140
HC/RS 4	53.7	0.371	840	5.79	6.00	0.152	5.00	0.127
5	56.68	0.391	1080	7.42	6.00	0.152	5.45	0.138
RC/HS 6	56.7	0.391	1060	7.35	5.36	0.136	5.10	0.130
7	51.9	0.358	1010	6.96	4.70	0.119	4.70	0.119
8	55.6	0.383	1080	7.47	5.80	0.147	6.00	0.152
RC/RS 9	51.0	0.352	967	6.67	8.70	0.221	6.70	0.170
10	44.8	0.309	825	5.69	4.00	0.102	4.10	0.104

D – Deflection

Acc - Accelerometer

Table 3 – Comparison of Deflection Results from SDOF to Experimental Results

	Analysis		% Difference			% Error	
Panel ID	D-SDOF (in)	D-SDOF (m)	SDOF to Laser	SDOF to Acc	Laser to Acc	SDOF to Laser	SDOF to Acc
HC/HS 1	4.81	0.122	0.00	19.8	4.95	0.21	18.5
2	4.70	0.119	10.4	2.49	1.97	10.6	2.13
3	4.49	0.114	28.6	20.5	2.05	33.6	22.5
HC/RS 4	4.41	0.112	30.3	12.5	4.48	36.0	13.4
5	5.74	0.146	4.03	5.63	2.41	4.53	5.05
RC/HS 6	6.74	0.171	22.8	27.2	1.13	20.5	24.3
7	5.69	0.145	19.7	19.7	0.00	17.4	17.4
8	7.61	0.193	27.1	23.8	0.84	23.8	21.2
RC/RS 9	6.82	0.173	24.4	1.75	6.52	27.6	1.76
10	4.20	0.107	4.78	2.84	0.49	4.76	2.38

Capabilities of a Shock Tube to Simulate Blast Loading on Structures

by A. Lloyd, E. Jacques, M. Saatcioglu, D. Palermo, I. Nistor and T. Tikka

Synopsis: Using a shock tube to subject structures to shock wave loading is a safe, economical and reliable alternative to live explosive testing. The University of Ottawa shock tube testing facility is capable of simulating shock wave induced loading of structures subjected to high explosive blast. The shock tube can accurately generate shock waves with similar properties as those produced by the actual detonation of high explosives. The parameters of the shock waves generated by the shock tube are found to be a function of driver length and driver pressure. Pressure-time histories are found to be in good agreement with similar free-air detonations. Furthermore, shock waves are found to be planar at the testing frame. Blast induced testing of structural elements, such as columns, slabs and masonry walls have been subjected to shock wave induced loading. This paper outlines the parameters of shock tube induced shock waves and explores the benefits of shock tube testing over live high explosive testing.

Keywords: blast; explosion simulation; reflected pressure; shock tube; shock wave; shock wave properties

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INTRODUCTION

The past two decades has seen a significant increase in the number of terrorist attacks on embassies, commercial centers, government structures, industrial facilities, and residential buildings. From a structural standpoint, these attacks have highlighted the vulnerability of existing civilian infrastructure to the dynamic effects of high pressure, short duration blast loading. Civilian, government and military organizations have been addressing these vulnerabilities by developing new blast resistant design guidelines and retrofit procedures to mitigate blast hazards. The current state of blast resistant design methods are based largely on empirical observations of actual explosive testing. However, due to the dangerous, expensive and uncertain nature of experimental blast research, the body of experimental blast data is very limited and many aspects of blast response of structures remain unknown.

With a focus on enhancement of life safety and structural integrity of both new and existing infrastructure, the University of Ottawa has established a new Blast Research Group within the Department of Civil Engineering. Structural components, such as columns, walls and slabs, are subjected to simulated blast loading using a pressure-driven shock tube. The shock waves generated with the shock tube closely simulate shock waves formed by the free-air detonation of a high explosive device. The shock tube facility itself is fully equipped with state-of-the-art data acquisition equipment, instrumentation and high speed video cameras to capture all aspects of shock wave-induced specimen response. Discussions on the shock tube generated shock wave parameters and the effective use of the University of Ottawa's shock tube to simulate the interaction between high explosive free-air blast pressures and structural elements are presented in this paper.

Description of Shock Tube Testing Facilities

The University of Ottawa shock tube can simulate a wide range of explosions with varying pressures and impulses. The shock tube, shown in Figure 1, is comprised of four main components: a variable length driver section, a double diaphragm firing control section, an expansion section and a rigid end test frame. The driver section is the main pressure vessel responsible for generating simulated blast pressures. It is comprised of a series of 597 mm (23.5 in.) inside diameter pipes with 19 mm (0.75 in.) wall thickness, each bolted together with twenty

31.75 mm (1.25 in.) bolts. The length of the driver section is variable between 305 mm (12 in.) and 5185 mm (204 in.) in 305 mm (12 in.) increments. The driver length is selected based on desired shock wave parameters, specifically peak reflected pressure and total reflected impulse. The 90mm (3.5 in.) spool section contains the double diaphragm firing system. The flanges on both sides of the spool section are designed to mate with the flanges on the driver and expansion sections (Figure 2). When bolted together with 20 threaded rods 31.75 mm (1.25 in.) in diameter, the mated flanges of the driver-spool-expansion section clamp the diaphragms in place. Two diaphragms are employed in the operation of the shock tube. Each diaphragm is comprised of up to three sheets of 813 mm by 813 mm (32 in.) grade 1100 aluminum foil of varying thickness. Diaphragm rupture strength is controlled by the number and thickness of aluminum foils.

An expansion section connects the driver and spool sections to the rigid test frame. Over a length of 6096 mm (240 in.) the cross section of the expansion section increases from 597 mm (23.5 in.) diameter circular to a 2032 mm by 2032 mm (80 in.) square opening at the rigid test frame. The geometry of the expansion section ensures the formation of a planar shock front. The entire length of the square expansion section is comprised of 12.7 mm (0.5 in.) thick stiffened steel plates. At the end of the expansion section is the testing frame, where specimens are placed for simulated blast testing. The testing frame is square with a 2032 mm by 2032 mm (80 in.) opening and a 203.2 mm (8 in.) wide, 12.7 mm (0.5 in.) thick stiffened steel plate surrounding the opening. There are a total of twenty 19 mm (0.75 in.) holes located in the testing frame for connecting samples to the shock tube. Twelve sliding pressure relief vents are located at the end of the shock tube expansion section. These vents, initially closed during the charging and firing of the shock tube, slide open when the shock wave reaches the end frame. The opening of the vents allows for the creation of a negative phase in the pressure-time history as the compressed air exits the shock tube.

Initiation of Shock Tube Firing – The double diaphragm firing mechanism allows for precise control over the operation and firing of the shock tube. Two static pressure sensors display the pressure in the driver and spool sections on a control panel during shock tube charging. Remotely controlled electro-mechanical valves allow the driver section and the spool section to be charged independently. During charging of the shock tube, the driver section is brought to the desired firing pressure and the spool section is brought to a pressure level which is approximately half the driver section's pressure. The pressure differential between the driver/spool sections and the spool/expansion sections is kept below the diaphragm rupture strength. Once the driver section reaches the desired triggering pressure, an electro-mechanical valve in the spool section is activated, venting the pressurized air in the spool section outside the shock tube. The pressure relief within the spool section brings the pressure differential between the driver section and the spool section above the rupture pressure of the first diaphragm causing it to rupture, followed immediately by the rupturing of the second diaphragm. This allows for the rapid expansion of pressurized air into the expansion section, causing the formation of a shock front which propagates down the length of the expansion section.

Instrumentation – A sixteen channel high-speed digital oscilloscope is used to monitor and record shock wave propagation and structural response to blast loading. Instrumentation includes piezoelectric dynamic pressure sensors, linear variable displacement transducers, piezoelectric accelerometers, strain gauges and dynamic load cells. Pressures, displacements, and loads are measured on 16-bit, 300 kHz channels at up to 1000 kilo-samples per second (kS/s). Strains are measured on 16-bit, 20 kHz channels at up to 100 kS/s using 350 ohm electrical resistance strain gauges. A high speed video camera is used to capture color images of specimen response at up to 1000 frames per second. The data acquisition system and high speed camera are triggered when the shock wave reaches one of the reflected pressure sensors located near the end of the expansion section.

Shock Wave Properties

An atmospheric detonation of a typical high explosive (TNT, C4, ANFO etc.) will have many effects including: a compression shock wave radiating outward from the source of the explosion, a fire ball, cratering of the ground, and projectile debris. Of these phenomena, the shock wave-structure interaction is generally the greatest concern for structural engineers (Naito 2006). A shock wave produced by high explosive detonation has some distinctive properties that are apparent when the pressure-time history is plotted for a stationary point in space as shown in Figure 3 (Baker 1973). These properties include a near instantaneous rise time from atmospheric pressure to maximum over pressure, followed by a decay in pressure over a period of time known as the positive phase up to when it reaches atmospheric pressure again. After this period, the pressure will drop below ambient atmospheric

pressure during the negative phase. Eventually, the pressure will return to normal ambient atmospheric pressures once the shock wave and corresponding rarefaction waves pass over the stationary point.

The compression shock wave in air of an explosion is driven by the rapid release of explosive gasses at the point of detonation. This results in a large pressure differential at the boundary of the explosion and the ambient atmosphere causing a shock wave to propagate outwards. The pressure differential that drives the shock wave is recreated using the University of Ottawa shock tube. The shock waves generated with the shock tube closely resemble explosive driven shock waves.

Range of Operation – The University of Ottawa shock tube can simulate blast induced shock wave loading within a well defined operational range. This operating range is defined in terms of peak reflected pressure, reflected impulse and positive phase duration. The magnitude of shock wave reflected pressure is controlled by selecting an appropriate driver pressure and the positive phase duration is controlled by driver length. By selecting both driver length and driver pressure, the impulse imparted to test specimens may be controlled.

Figure 4 shows a scatter plot of maximum recorded reflected pressures versus driver pressure for different driver lengths. As expected, larger driver pressures result in larger reflected pressures. The maximum reflected pressure achievable with the shock tube is approximately 100 kPa (14.5 psi). Furthermore, driver length has only a minor influence on reflected pressure for all but the smallest driver lengths. For example, at the smallest driver length of 305 mm (12 in.), reflected pressures are significantly lower than the trend from all other driver lengths. A best fit curve has been added to the scatter plot for the 305 mm (12 in.) driver length values to help differentiate the values from those of the other driver lengths. This deviation is primarily due to the energy loss of a relatively small volume of compressed air for a 305 mm (12 in.) driver expanding through the 90 mm (3.75 in.) spool section into the large expansion section when compared with the larger volumes associated with longer driver lengths.

Figure 5 shows the reflected impulse over the positive phase plotted against driver pressure for various driver lengths. Longer driver lengths produce shock waves with a longer positive phase duration, since there is a greater volume of compressed air driving the creation of the shock front. The maximum reflected impulse for the positive phase that can be achieved in the shock tube is approximately 2200 kPa-ms (319 psi-ms). Tests at maximum pressure capacity were not performed for all driver lengths.

Maximum values for reflected pressures and corresponding impulses for five shock tube driver lengths are provided in Table 1. For comparison, the equivalent masses and standoff distances for hemispherical explosions of TNT have also been listed in Table 1 for the specified pressure and impulse combinations (Hyde 1992).

Positive Phase Duration and Tertiary Shock Waves – Figure 6 shows positive phase duration versus driver pressure for varying driver lengths. Best fit lines have been added to differentiate between the different driver lengths. For driver lengths below 3355 mm (132 in.), positive phase duration is relatively independent of driver pressure. However, for longer driver lengths, both the driver pressure and driver length have a significant influence on the positive phase duration. The significant scatter evident in the plot of positive phase duration versus driver pressure is largely due to two factors: inaccuracy due to signal noise and the shallow slope of the gradual change in pressure as the positive phase duration time is approached.

Figure 7 shows an example shock wave pressure-time and impulse-time history generated using a 2745 mm (108 in) long driver with a driver pressure of 497 kPa (72 psi). This figure shows that the exact positive phase duration may be difficult to determine due to the amount of signal noise and the shallow slope of the pressure versus time relationship at the end of the positive phase. In the integration of the pressure-time relationship to determine impulse, unbiased signal noise (which is equally distributed above and below the real pressure values) is eliminated. To overcome the difficulties in directly determining the positive phase duration from the pressure time history, the positive phase duration is taken at the local maximum of the impulse-time relationship.

Tertiary Shock Waves – Tertiary reflections are the internal reflections within the shock tube as the wave interacts with the end of the driver section. These tertiary shocks continue with decreasing amplitude and duration until they are eventually damped out of the system. These reflections occur after the end of the first positive phase, as shown in the pressure-time and impulse-time histories in Figure 8 for a shock wave generated using a driver

length of 2745 mm (108 in) and a driver pressure of 69.0 kPa (10 psi). Since the tertiary reflections occur as shock fronts rather than more gradual ascending waves, they have no influence on positive phase duration.

Figure 9 shows the maximum tertiary reflected pressure for the first tertiary shock wave plotted against the initial maximum reflected pressure. In general, the larger the initial reflected pressure, the larger the tertiary reflected pressure. This trend does not seem to have any direct relationship to the driver length. The arrival time of the tertiary shock wave is plotted against the reflected pressure of the initial shock wave for various driver lengths in Figure 10. This figure shows that the arrival time of the tertiary shock wave appears to be primarily dependent on the driver length and relatively independent of the peak reflected pressure. In general, tertiary shock wave arrival time increases with increasing driver length as internal shock wave reflections must traverse longer distances.

Shock Wave Planarity – A series of tests were performed in order to establish the planarity of the shock front. A rigid steel reflecting plate was mounted to the testing frame and instrumented with piezoelectric dynamic pressure gauges mounted at twelve locations, illustrated in Figure 11. These gauges were used to record reflected pressure time histories and different arrival times. The time histories were integrated to form reflected impulse time histories. The peak reflected pressures, reflected impulse, relative arrival times, and positive phase durations were then compared at these points along the face of the reflecting surface to determine the shape of the shock wave in the plane of the testing frame at a specific time. It was found that shock waves produced within the shock tube are planar at the location of the test frame with a maximum variation in arrival time of 0.29 ms. Variation in pressures were virtually indistinguishable from signal noise. The maximum variation of the positive phase impulse across the test area was less than 8 % in all cases. The results of these tests show that the shock front is effectively planar at the testing frame.

Wave Front Velocity – The wave front velocity of the shock waves generated with the shock tube was measured using two pressure gauges located on the walls of the expansion section located 50 mm (2 in) and 1067 mm (42 in) away from the reflecting surface on the test frame. Wave front velocities, plotted in Figure 12, were computed by dividing the distance between gauge locations by the difference in arrival time at these two locations. In general, the wave front velocities are greater than the speed of sound 0.34029 m/ms (13.3972 in/ms), at standard temperature and pressure (20 °C, sea level atmosphere). The wave front velocities have a slight positive trend, increasing with reflected pressures.

Dynamic Pressure – The relationship between pressure, density, shock front velocity, and particle velocity developed by Rankine and Hugoniot (Rankine 1870) and later modified by Lipman and Roshko (1957) were used to explore the recorded wave front velocity with respect to dynamic pressures. Dynamic pressure is the pressure associated with the impact of moving air particles when the shock wave interacts with an object. This pressure is dependent on the density of the compressed air within the reflected shock wave and the velocity of the air particles. Since particle velocity was not measured directly and since the shock tube simulates far range explosions, dynamic pressure was assumed to be negligible (Smith and Hetherington 1994). Using this assumption, incident overpressure is calculated as half the reflected pressure in the Rankine and Hugoniot equations (Smith and Hetherington 1994).

The velocity of the shock front is assumed to be a function of the incident overpressure, the ambient atmospheric pressure and the speed of sound in ambient atmosphere in the Rankine and Hugoniot equations. Since all these variables were known, the shock wave front velocity was calculated as a function of reflected pressure and compared with the measured values. The relative agreement between the measured (represented by open circles) and predicted (straight line) wave front velocity, shown in Figure 12, indicates that the initial assumption of negligible dynamic pressures is correct. If the shock wave was allowed to transverse the test specimen, the low wave front velocities and corresponding low particle velocities would result in small diffraction forces. As a result, use of the shock tube for diffraction tests is not recommended.

ELEMENT TESTING AND SUPPORT CONDITIONS

Planar Elements

Structural components that are planar in the direction loaded by the shock wave, such as slabs and walls, are suited to shock tube testing under reflected pressures. These elements are mounted directly to the test frame at the end of the shock tube and cover the entire opening of the test area. Element boundary conditions allowing for one or

two-way bending, under simply-supported or fully-clamped boundary conditions are easily accommodated. Since specimens are mounted directly to the shock tube, there is no relative displacement of specimen ends. Figure 13 shows a two-way reinforced concrete slab connected to the shock tube. In this configuration, fixed-fixed end conditions are provided by clamping the slab between the shock tube test frame and a welded steel frame.

Figure 14 shows the pressure-time and displacement-time histories for the slab shown in Figure 13 under shock wave testing. The slab was 75 mm (2.95 in) thick with a clear span between supports of 2132 mm (83.94 in) in both directions. The slab had 60 MPa (8702 psi) concrete and was doubly reinforced with eleven 580 MPa (84122 psi) yield stress 6.3 mm (0.25 in) diameter non-deformed steel wires evenly spaced in each principle direction. The reflected pressure was 91.8 kPa (13.3 psi) and the reflected impulse was 2621 kPa-ms (380 psi-ms) over the positive phase. This resulted in a maximum displacement of 79.0 mm (3.11 in) and a residual displacement of 26.1 mm (1.03 in) at the center of the slab.

Other planar elements, such as masonry walls, are similarly mounted directly to the testing frame. In the case of infill masonry walls, a steel frame is provided around the perimeter of the wall to simulate the interaction between a structural frame and the wall. For masonry wall testing, steel beams may be placed at the top and bottom of the wall to prevent global wall failure and promote flexural and arching behaviour. Figure 15 shows an infill masonry wall mounted to the shock tube prior to testing.

Non-Planar Elements

Since the shock tube cannot simulate diffraction loading due to the relatively small wave front velocity of the shock waves, structural components which do not cover the entire face of the shock tube, such as reinforced concrete columns, need a load transferring device to transfer the pressure from the shock tube to the element. This device consists of light gauge steel sheet metal that covers the entire face of the shock tube. Attached to the sheet metal are a series of steel beams running perpendicular to the primary plane of bending and equally spaced along the height of the shock tube opening. The reflected pressures from the shock wave are transferred through the rigid beams as a series of point loads along the length of the element, effectively simulating a uniformly distributed load. The combination of flexible sheet metal and rigid steel beams allows for full transfer of reflected shock waves from the entire opening of the shock tube to the face of individual structural elements, without modifying the stiffness of the element being tested. Figure 16 shows a reinforced concrete column mounted to the shock tube that has been fitted with such a load transfer device.

Although the use of a load transfer device does not change member stiffness, it will cause a significant change in the mass of the dynamic system. For instance, the reinforced concrete column shown in Figure 16 had a mass of 80 kg (176 lbs) while the load transferring device had a mass of 545 kg (1200 lbs). The additional mass of the load transfer the system has been found to act on the elements for the entire duration of response, up to the time of maximum displacement. Using equivalent single degree of freedom (SDOF) mass factors (Biggs, 1964), the mass of the load transfer device in this configuration increases the natural period of the sample by a factor of 2.8 to that of the column alone, making the system less susceptible to impulsive loading. Response of a non-planar element after maximum displacement cannot be modeled with a simple SDOF model, rather a coupled system considering the load transfer device and the structural element as each having independent degrees of freedom would be needed.

Analysis of member response has been performed using the equivalent SDOF procedures described in “Structures to Resist Effects of Accidental Explosions” (UFC 03-340-02 2008). These analyses have shown that close agreement between experimental and predicted results may be achieved if the mass of the load transfer device is included in the analysis. A sample of the results of a SDOF analysis is shown in Figure 17. In this figure, the experimentally recorded mid-height displacement of a reinforced concrete column is plotted along with the predicted SDOF displacement and the pressure time-histories. The analysis incorporated the recorded pressure-time history, which was assumed to act uniformly over the entire area of the load transfer device. The load transfer device was assumed to act on the column up to the point of maximum displacement. Displacements were predicted up to maximum by applying resistance curves, developed from a static analysis using dynamic strength increase factors, to the dynamic equation of motion.

UPCOMING WORK

To date, the shock tube has been used to simulate blast loading of individual structural elements, such as reinforced concrete columns, one-way and two-way slabs and masonry walls. Additionally, the shock tube has been used to evaluate the effectiveness of various retrofit techniques to promote blast resistant behaviour. These techniques include: the addition of externally bonded flexural and confinement fibre reinforced polymers (FRP) composites; the use of mechanical anchorage to prevent or delay premature debonding failure of surfaced bonded FRP laminates; and masonry retrofits to promote compression membrane action. Results from these testing programmes have shown that the shock tube is able to effectively and efficiently simulate the dynamic effects of shock wave loading on individual structural elements. However, the shock tube has many applications beyond testing individual elements.

The proper development of anchorage of reinforcing steel and concrete forms the basis of the composite action of reinforced concrete members and allows for the full mobilization of flexural and shear capacity. The bond between typical deformed steel reinforcing and concrete depends primarily on lug resistance, friction and chemical adherence (Solomos 2009). The effect of monotonic and hysteretic loading on anchorage slip is generally well understood, and design codes have incorporated clauses to account for these types of load on the development length for reinforcing bar in concrete (CSA A23.3 2004; ACI 318 2005). However, the current understanding of anchorage slip phenomenon for reinforced concrete members subject to impulsive loads is less well defined and in need of further investigation before development lengths to resist dynamic loading may be proposed. To this end, an upcoming series of experimental tests will be focused on the effect of high rates of loading on the development length of reinforcing steel. A load transfer mechanism will be used to apply dynamic forces to perform reinforcing bar development length pull out tests.

SHOCK TUBE AND HIGH EXPLOSIVE TESTING

Advantages of Shock Tube Testing

The University of Ottawa Shock Tube Testing Facility is ideally suited for simulated blast testing of reinforced concrete structures with a focus on the development of blast resistant code clauses and retrofit techniques. There are several advantages to shock tube testing over high explosive testing.

Safety – By only simulating the high pressure shock waves generated by blast, shock tube testing eliminates any risks associated with the secondary effects of explosions: fire balls, intense heat, debris and projectiles. Additionally, high pressure dynamic loads are only localized on test specimens: shock wave pressures quickly dissipate to atmospheric pressure away from the shock tube. The primary danger associated with simulated blast testing using the shock tube is the noise of the tests, measured at up to 130 decibels at the location of the remote control station. Other dangers, such as debris from the damaged specimen, are mitigated by remotely controlling shock tube firing and ensuring that no personal are in the vicinity of the shock tube during testing.

Cost – After the initial capital investment of the shock tube and instrumentation, the only costs associated with operation of the shock tube are the material cost of test specimens, support conditions and aluminum foil for the diaphragm. The cost of performing similar experiments with live explosives renders the shock tube an attractive research alternative.

Range of Experiments – A large 2032mm x 2032mm square loaded area at the mouth of the shock tube is ideally suited to testing large scale specimens. Furthermore, a 203 mm x 12.7 mm stiffened steel plate bounds the perimeter of the test area and is sufficiently rigid to accommodate a variety of support conditions and test apparatuses. In addition to the testing of individual structural elements – columns, walls, slabs, etc – non-structural components such as exterior cladding, windows and interior partition walls are all topics of research which are well suited to simulated blast testing with the shock tube. Furthermore, load transfer mechanisms have been used to focus shock wave pressures, acting over the entire loaded area, to small points of application, therefore, making the shock tube ideally suited for testing the dynamic increase in compressive strength of concrete and studying the effect of dynamic loading on anchorage slip of concrete reinforcement.

Repeatability of Experiments – The double diaphragm firing mechanism allows for accurate repeatability of shock wave parameters including reflected pressure, impulse, and the positive phase duration.

Instrumentation and Monitoring – An interior, low pressure environment outside the shock tube allows for ease of instrumentation and testing. The low pressures outside the shock tube allow cameras and other instrumentation to be placed in the immediate vicinity of the test specimen. The relatively short length of signal and power cables ensures there is little signal degradation. Unlike high explosive testing, no magnetic field is created during testing that could interfere with measurement signals. Piezoelectric pressure sensors are permanently installed within the shock tube ensuring that pressure readings are consistently measured at the same locations for every test.

Ease of Operation – Up to eight tests per day may be conducted with the shock tube. These tests are done in an interior laboratory environment so there are no limitations on testing due to time of day or exterior weather conditions. The University of Ottawa Structures Laboratory is equipped with a 10 tonne (11 ton) overhead crane that facilitates specimen movement and mounting. The shock tube is located directly below a 914 mm (36 in) thick strong floor which provides bearing for applying axial load to specimens.

Limitations of Shock Tube Testing

Although quite appealing when compared with other forms of experimental testing, shock tube testing is not without limitations.

Only Member and Component Testing – The shock tube is only capable of simulating blast loading of individual structural members and components. Other phenomena, such as the interaction between structural systems during an explosion or the redistribution of forces from one structural element to another, are outside of the working capability of the shock tube. These types of structural systems cannot be tested due to size limitations of the facility and an upper limit of reflected pressures generated during testing.

Scale – Due to the geometry and maximum operational range of the shock tube, only scaled elements may be tested. The nature of the dynamic response of structures requires special attention be afforded to ensuring structural similitude between scaled specimens and full sized structures. The mass of a member increases in proportion to its volume. The stiffness of a member increases proportionally to its moment of inertia. Member strength, either in pure flexure or more complicated modes (such as compression or tension membrane), is a function of both material properties and internal moment arms. All of these important member properties scale at different rates when compared to simple geometry. Beyond member properties, further difficulties arise because the effect of small construction defects resulting from inaccuracies in reinforcing steel placement or concrete voids may become significant in scaled testing.

Far Range (High Impulse) – Shock waves generated with the shock tube only simulate explosives located at relatively large standoff distances from targets. Depending on testing requirements, the relatively long positive phase durations (between 5 and 60 ms) of shock tube induced shock waves may necessitate companion testing with high explosives.

CONCLUSION

This paper summarizes the capabilities, parameters, limitations and effectiveness of using a shock tube to induce shock waves as a safe, economical and reliable alternative to live explosive testing. The following conclusions can be drawn from the experimental program undertaken with the University of Ottawa shock tube testing facility and discussion presented in this paper:

- The shock tube at the University of Ottawa Structural Engineering Laboratory has a well defined range of operation. The shock waves are predictable and repeatable and are planar when interacting with test specimens.
- The small wave front velocity and corresponding small particle velocity of the shock tube generated shock waves result in negligible dynamic pressure. This small particle velocity also precludes destructive diffraction tests. The shock tube is intended to form a full reflected shock wave over the entire face of the test area.
- A wide range of structural and non-structural elements may be tested with the shock tube. A load transferring device has been used to distribute pressure from the test frame opening onto elements that do not cover the entire area. When using a load transferring device, the mass of the device must be

incorporated into analysis of the results. Planar elements which cover the entire shock tube opening may be tested without a load transferring device.

- Shock tube testing has many advantages over live high explosive testing. These advantages include the ease of testing, low operating cost and high levels of safety. Significant advantages are found in the accurate prediction of shock wave parameters.
- There are some significant disadvantages of shock tube testing when compared with high explosive testing. One disadvantage is that the shock tube generated shock waves have relatively large positive phase durations and, therefore, are not suited for short standoff distance simulation. A further disadvantage comes from the relatively small size of the shock tube test frame which places a practical size limit on specimen and the possibility of scaling test specimen sizes.

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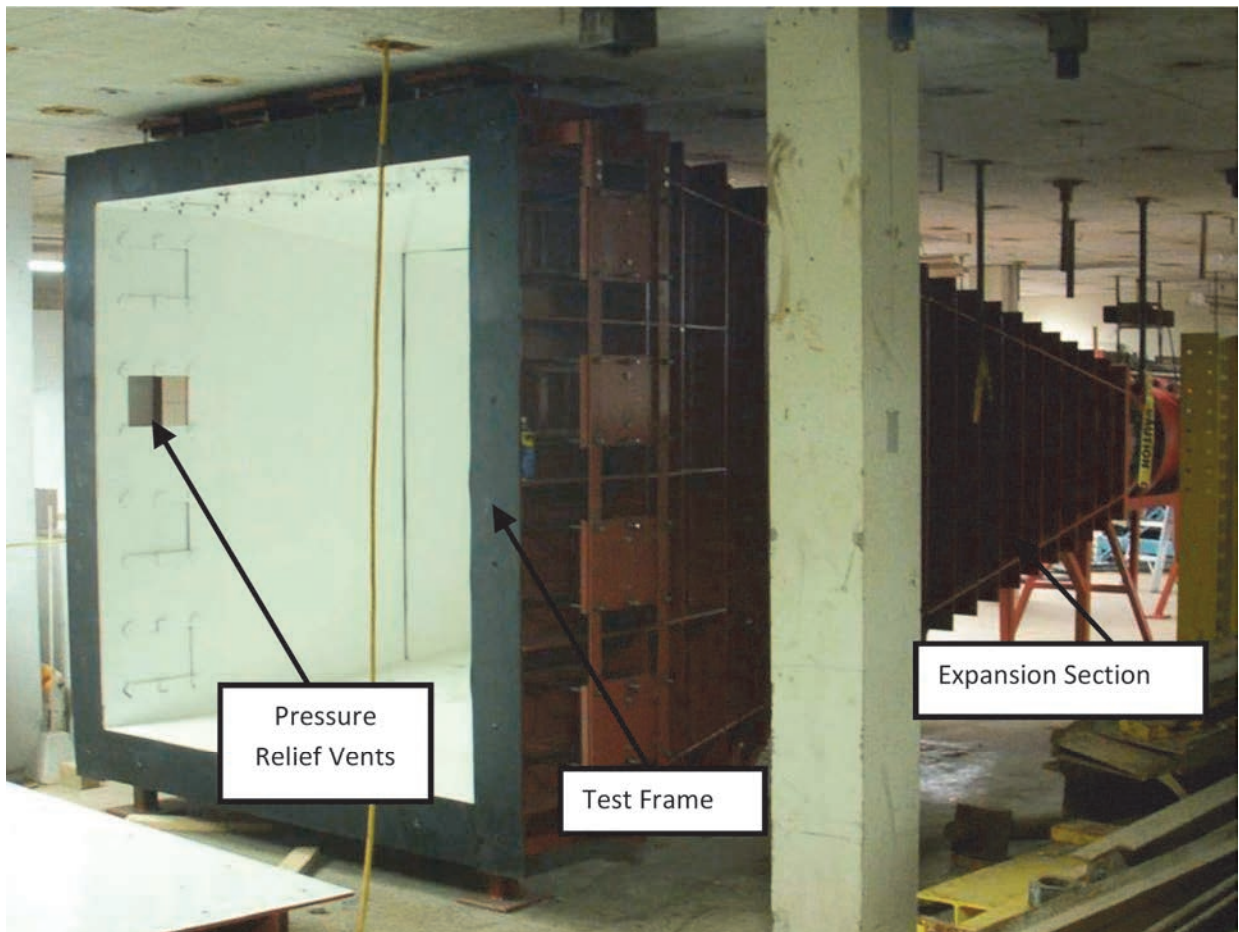
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Table 1 – Maximum shock tube range

						Approximate Equivalent TNT (Hemispherical-Reflected)					
Driver Length		Reflected Pressure		Reflected Impulse		Mass		Standoff		Scaled Distance	
mm	(ft)	kPa	(psi)	kPa-ms	psi-ms	kg	(lb)	m	(ft)	m/kg ^{1/3}	(ft/lb ^{1/3})
305	(1)	78	(11.3)	217	(31.5)	8	(3.6)	12	(39.4)	6.0	(25.6)
915	(3)	92	(13.3)	410	(59.5)	42	(19.1)	18	(59.1)	5.2	(22.1)
1830	(6)	100	(14.5)	840	(122)	290	(132)	33	(108)	5.0	(21.3)
3355	(11)	103	(14.9)	1760	(255)	2500	(1136)	67	(220)	4.9	(21.1)
4880	(16)	104	(15.1)	2690	(390)	10000	(4545)	106	(348)	4.9	(21.0)



a) Driver section



b) Expansion section and test frame

Figure 1 – Shock tube sections

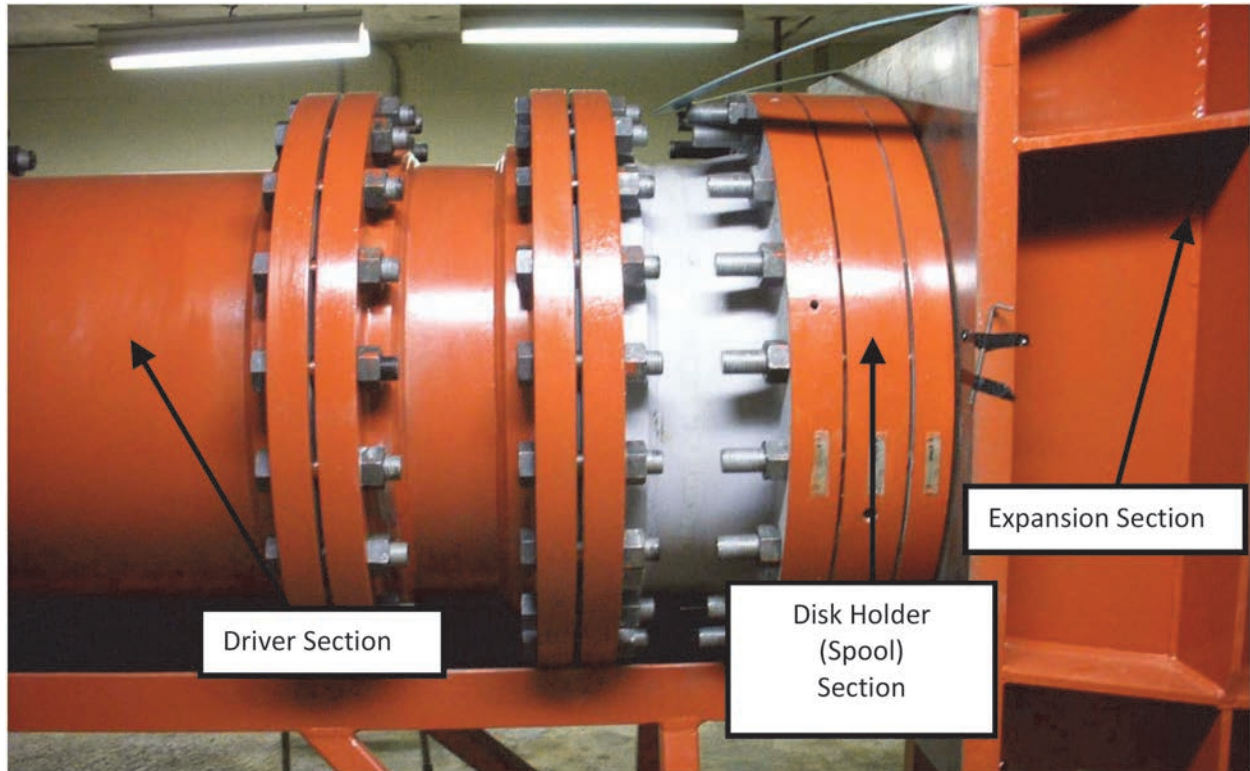


Figure 2 – Shock tube double diaphragm firing control (Spool) section

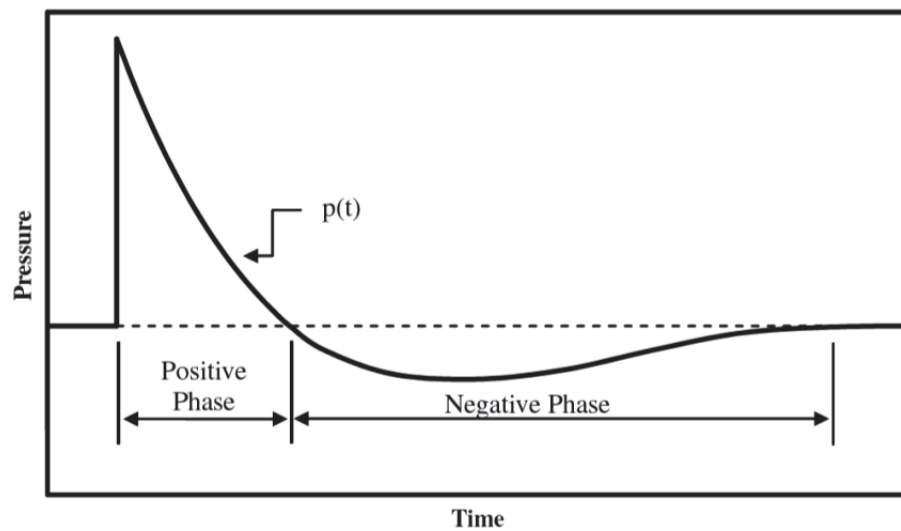


Figure 3 – Theoretical reflected pressure and impulse time history for high explosive generated shock wave

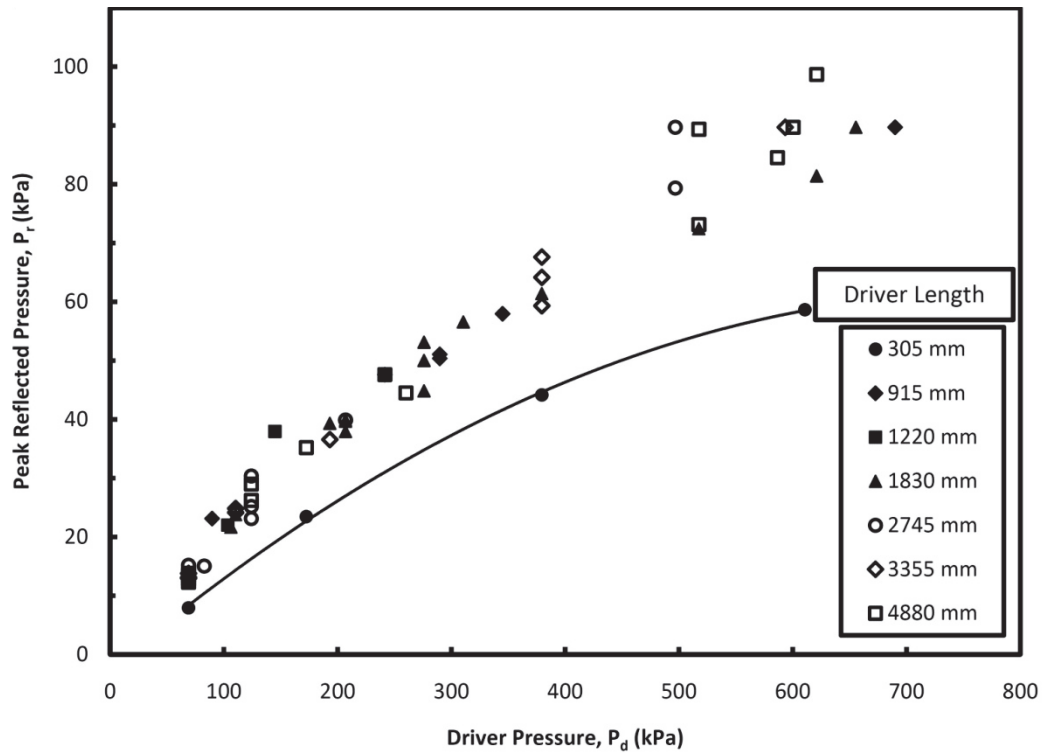


Figure 4 – Reflected pressures as a function of driver pressure for various driver lengths

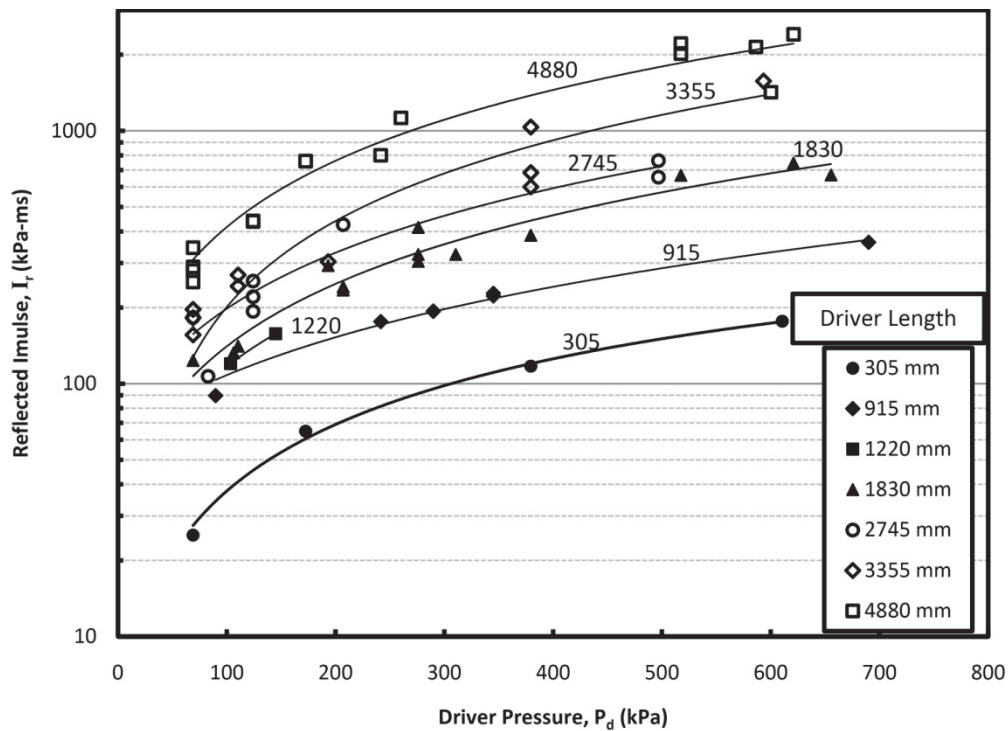


Figure 5 – Reflected impulses over the positive phase as a function of driver pressure for various driver lengths

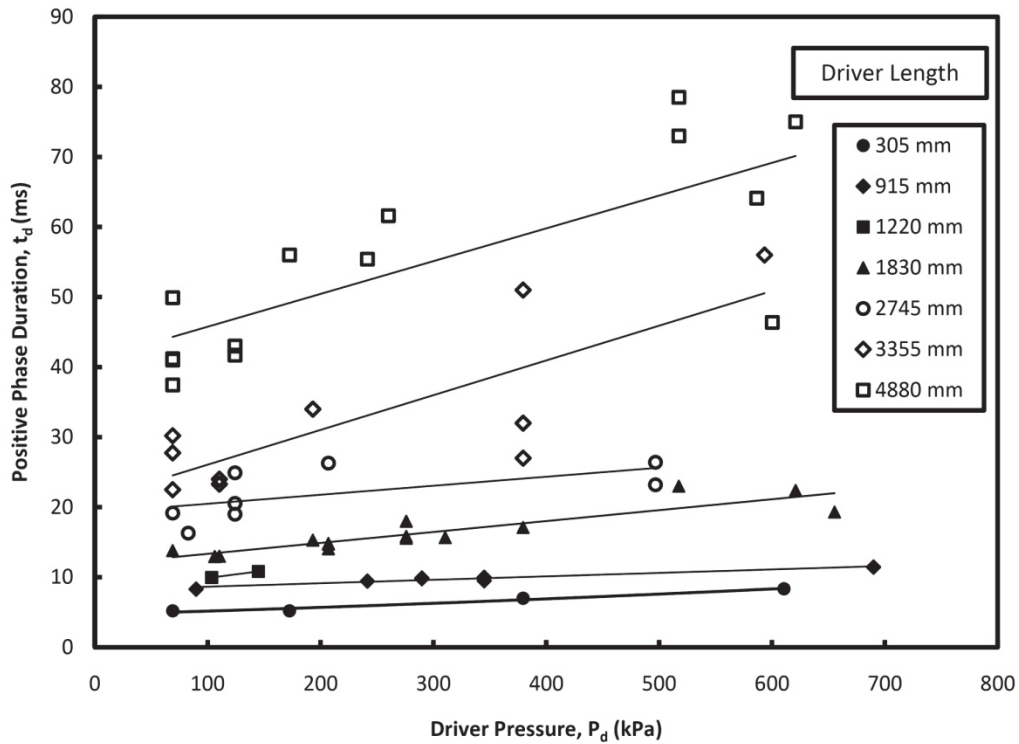


Figure 6 – Positive phase durations as a function of driver pressure for various driver lengths

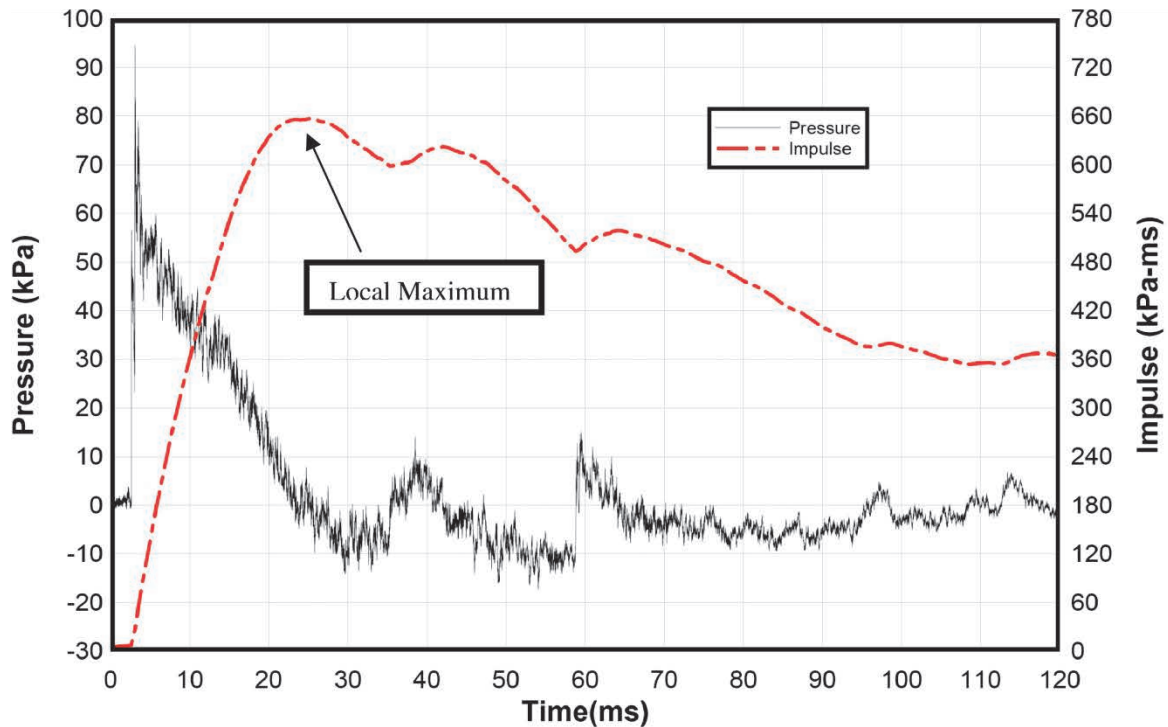


Figure 7 – Reflected pressure- and impulse-time history for a shock tube generated shock wave

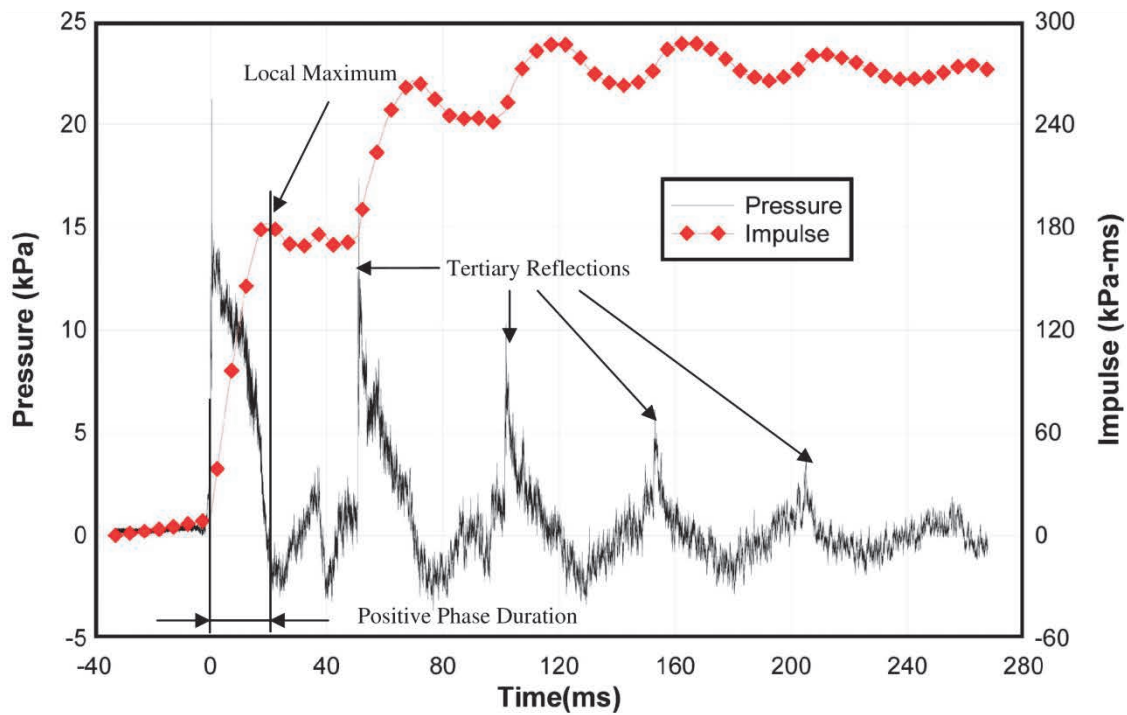


Figure 8 – Tertiary reflected pressures and impulses for a shock tube generated shock wave for a driver length of 2745 mm (108 in) and a driver pressure of 69.0 kPa (10 psi)

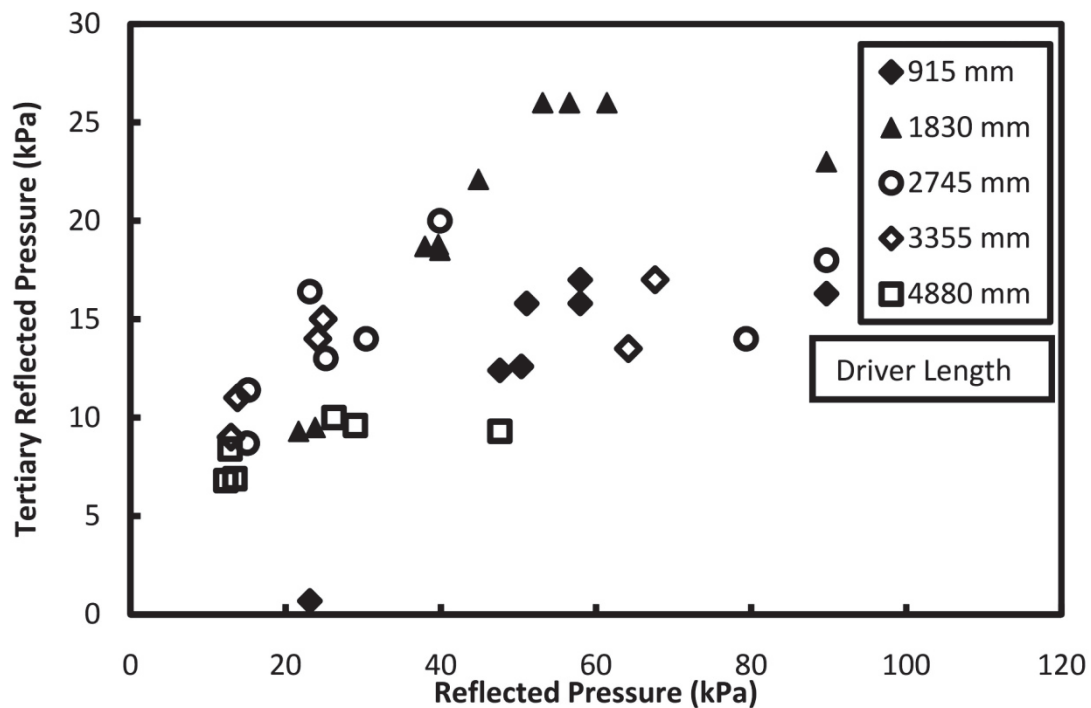


Figure 9 – Maximum tertiary reflected pressure versus maximum initial reflected pressure for various driver lengths

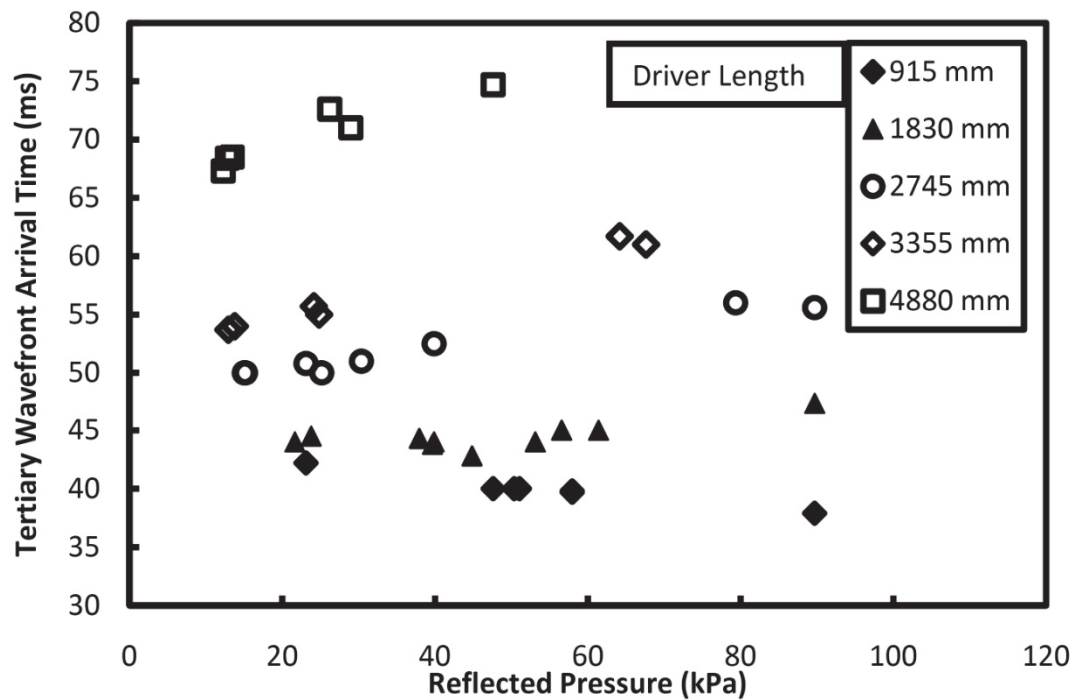


Figure 10 – Arrival time of tertiary shock wave versus maximum initial reflected pressure for various driver lengths

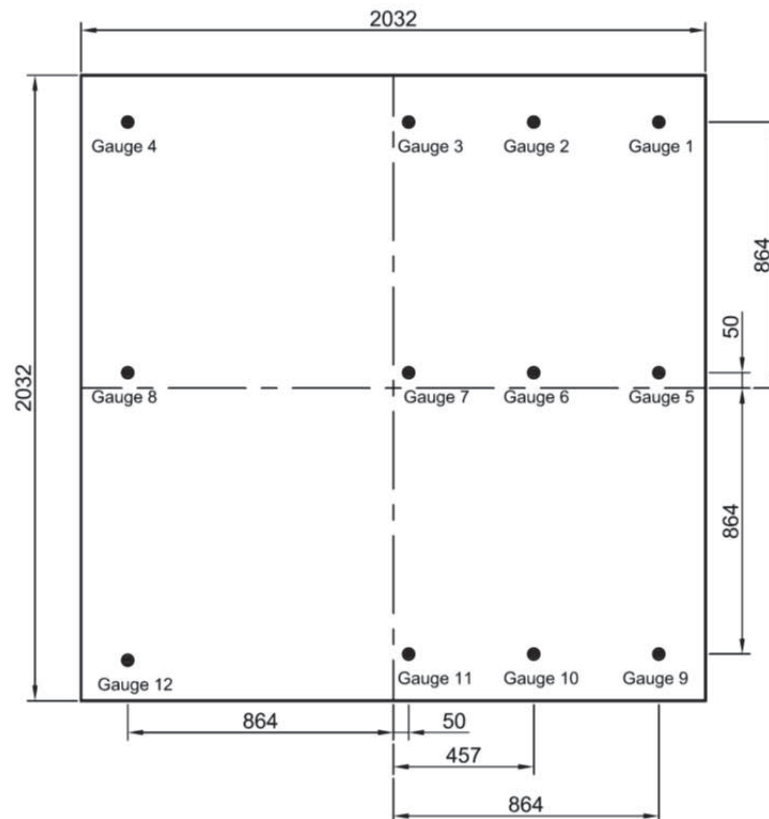


Figure 11 – Locations of pressure gauges on rigid plate for determining shock wave planarity