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SYMPOSIUM VOLUME



Analytical and Finite Element Concrete Material Models - Comparison of Blast Response Analysis of One Way Slabs with Experimental Data

Editors: Ganesh Thiagarajan and Eric Williamson

SP-306



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First printing, March 2016

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Printed in the United States of America

Editorial production: Ryan Jay

ISBN-13: 978-1-942727-77-4

Preface

The mission of ACI-ASCE Committee 447 is to develop and report information on the application of finite element analysis methods to concrete structures. The mission of ACI 370 is to develop and report information on the design of concrete structures subjected to blast, impact, and other short-duration dynamic loads. In this Special Publication (SP) and the accompanying presentations made at the ACI Fall 2013 Convention in Phoenix, Arizona, these committees have joined efforts to report on the state of practice in determining the Behavior of Concrete Structures Subjected to Blast and Impact Loadings. Recently, the (2008-2014) National Science Foundation (NSF) funded a study by University of Missouri Kansas City (UMKC) (CMMI Award No: 0748085, PI: Ganesh Thiagarajan) to perform a series of blast resistance tests on reinforced concrete slabs. Based on these results, a Blast Blind Simulation Contest was sponsored in collaboration with American Concrete Institute (ACI) Committees 447 (Finite Element of Reinforced Concrete Structures) and 370 (Blast and Impact Load Effects) and UMKC School of Computing and Engineering. The goal of the contest was to predict the response of reinforced concrete slabs subjected to a specified blast load using a variety of simulation methods. The blast experiments were performed using a Shock Tube (Blast Loading Simulator) located at the Engineering Research and Design Center, U.S. Army Corps of Engineers at Vicksburg, Mississippi.

Over 40 entries were received from researchers and practitioners worldwide; the competition was open to methods used in both research and practice. There were four categories in the contest: 1) Advanced Modeling of slabs with Normal Strength Concrete and Normal Strength Steel, 2) Analytical or Single-Degree-of-Freedom (SDOF) Modeling of slabs with Normal Strength Concrete and Normal Strength Steel, 3) Advanced Modeling of slabs with High Strength Concrete and High Strength Steel, and 4) Analytical or SDOF Modeling of slabs with High Strength Concrete and High Strength Steel. The first- and second-place winners were invited to present their work at the Fall 2013 convention. Furthermore, all teams were invited to submit papers for this SP, and original experimental data were provided to allow the teams to compare their results with those measured. This SP is a result of all the papers that were submitted and reviewed in accordance with ACI peer review requirements. In this SP, there are three papers from academic researchers and six from industry personnel, providing a healthy cross section of the community that works in this area.

The editors gratefully acknowledge all the hard work by the authors, the reviewers, and ACI staff, especially Ms. Barbara Coleman, who have helped very enthusiastically during every stage of the process. The editors also thank members of ACI Committees 447 and 370 for their continuous support in reviewing the papers.

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PARAMETERS INFLUENCING FINITE ELEMENT RESULTS FOR CONCRETE STRUCTURES

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Synopsis: It is widely recognized that a competent constitutive model for concrete, and a set of calibrated parameters for it, are important to producing accurate response predictions using the finite element method (FEM). What is not obvious, without having access to a large database of test data and practical experience using and validating the FEM models, is that a host of parameters for the FEM calculation can significantly influence the results. The objective of this paper is to identify these parameters and illustrate their effect by computing the response of some simple concrete structure tests subject to transient loads. Calculations for each of these structures demonstrate that in addition to some of the more nuanced material model parameters, parameters involving boundary conditions, hourglass energy suppression, interface friction, and loading-rate effects, all have a strong effect on the response predictions. The results demonstrate that any of four concrete constitutive models considered in this paper can be used to match any one set of test data, even though they differ in their assumptions and the behaviors modeled through their formulation; however, it is difficult to match the larger set of data without carefully considering each of these parameters. Guidance is provided to produce meaningful computational results using the constitutive model developed by the authors.

Keywords: Concrete; Reinforced Concrete; Finite Element Method; Constitutive Model; Parameters; Transient Loads; Blast; Shock.

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1. INTRODUCTION

Three-dimensional finite element method (FEM) calculations for concrete structures are now commonplace in many applications. Common applications range from academic studies of structural behaviors under transient loads to informing designs for new structures or retrofits. The objective of this paper is to identify factors that influence response predictions obtained with FEM models for concrete structures under transient loadings. This is accomplished by computing the response of several simple concrete structures and comparing the results with data and observations taken from experiments. The FEM calculations focus on codes solving nonlinear continuum mechanics equations [1] capable of treating concrete behaviors under transient loads and large deformations. All of the computational results presented here are executed using the LS-DYNA code [2], which is a widely used commercial FEM software.

It is widely recognized that a competent constitutive model for concrete, and a set of calibrated constitutive parameters for it, are important to producing accurate response predictions using FEM. Several studies already exist comparing many of the widely used models [3] [4] [5] [6]. In this paper, four concrete models available in LS-DYNA are considered. The goal is not to redo some of the comparisons presented in those publications or advocate for use of any one model; instead, our purpose is to demonstrate that although each of these models differ in their assumptions and formulations, the accuracy of structural response predictions computed with them depend on nuanced material model parameters, and often, parameters of the FEM calculations that have little to do with the material model. For this reason, the discussion herein is primarily focused on results obtained using the constitutive model developed and enhanced over the last twenty years by the authors and the late Dr. L. Javier Malvar [7] [8] [9], which is known as the Karagozian & Case Concrete model (KCCM). In LS-DYNA, the model is implemented as MAT072_REL3. The others considered are: the Winfrith Concrete Model (WCM, MAT084); the Continuous Surface Cap Model (CSCM, MAT159); and the Reidel-Hiermaier-Thoma model (RHTM, MAT272). Each of the concrete models can be exercised by entering all of the model variables directly, if mechanical data is available to calibrate those variables, or values can be automatically generated using a “simple input” feature available in each model (e.g., see [8] and [9] for the KCCM). Because data needed to calibrate these models for a specific concrete is costly, and therefore not commonly acquired in practice, the latter is more widely used and the paper focuses on results obtained using this option.

Five concrete structures tests are selected for the FEM calculations. Each serves to illustrate an important behavior, feature, or response regime for concrete structures that are highly desirable for the FEM calculation to replicate.

- The first are standardized concrete cylinder and cube specimens placed under compression to measure the unconfined compressive strength (f'_c). When comparing standard cylinders and cubes, experimental evidence suggests that cube strength is typically greater than cylinder strength, by a factor that varies up to 1.54 for normal strength concretes [10] [11] [12]. In practice, specimen size is standardized to 152-mm diameter, 304-mm tall (6-in diameter, 12-inch tall) cylinders, or 150-mm (6-in) cubes. Typically, cylinder specimens are capped (following ASTM C 617) and placed between grooved platens having various amounts of friction that result in lateral restraint at the top and bottom. An unrestrained form of testing cylinders for compressive strength is relatively recent (ASTM C 1231 was first established in 1993). The DIN 1048 standard for the cube specimens, however, does not prevent restraint at the top and bottom of the cube specimen. The FEM calculations address the factors that affect these specimens, i.e., specimen shape and end restraint.
- The second is the splitting tension (or Brazilian) test (DIN 1048 and ASTM C 496), where compression is used to generate failure by indirect tensile stresses. Rupture strengths (f_r) measured with the splitting tension tests are known to result in values exceeding the direct tensile strength (f_t). CEB-FIP reports the ratio of f_r/f_t as 1.11 [11]. Rocco et al. report this ratio between about 1.05 and 1.25 depending on specimen shape, aggregate size, concrete strength, and plywood strip width [13]. For 25-mm (1-inch) aggregates, cylindrical specimens, and increasing strengths from 30 to 80 MPa (4,350 to 11,600 psi), the ratio decreases from 1.16 to 1.08.
- The third consists of a split-Hopkinson Pressure Bar (SHPB) test [14], in both compression and tension. SHPB tests generate a dynamic uniaxial stress pulse into a concrete sample, thereby deforming the material rapidly. SHPB tests for concrete are not commonly performed nor are they standardized. SHPB and sample diameters are commonly several times the maximum aggregate size (MAS) of the concrete. In compression, the sample is placed between the two bars and lubricated; in tension, the sample is adhered to the bars. Loading rate effects have been observed from SHPB measurements for decades [15-18]. The data exhibits a two branch behavior when plotted with normalized peak strength versus strain rate in both compression and tension and are often termed Dynamic Increase Factor (DIF) curves [11] [15] [16] [17]. Rate effects in concrete are believed to be due initially to moisture (for example, below 1 s^{-1}), and, at higher strain rates, to inertia effects [18].

- The fourth structure is a thin reinforced concrete (RC) panel blast tested by the U.S. Army Corp of Engineers Engineering Research and Development Center (ERDC) using a blast load simulator. Details of the test, the panel, and the support frame are described in [19]. The RC panel, PH-Set 1a, is loaded by a dynamic pressure loading applied over the front surface of the panel, which deforms the panel in flexure. The panel is reinforced with longitudinal and horizontal mild steel rebars placed on the tensile-side of the panel; no transverse reinforcement is provided that can provide confinement to the concrete. The panel is placed in a support frame with its two long sides supported with structural steel tubes on both sides of it (i.e., on the front and back face of the panel).
- The last structure consists of a series of confined RC column tests described in [20]. The column is loaded with a monotonically-increasing uniform lateral loading by way of servo-hydraulic actuators operated under displacement-control. An initial axial load is held constant during the lateral loading. Confinement to the column is provided by Carbon Fiber Reinforced Plastic (CFRP) wraps placed around the column's cross-section. As the RC column is loaded laterally, the concrete cracks, due to the flexure and shear deformations, dilates, and the CFRP acts to resist the lateral expansion of the concrete, thereby increasing confinement. Contrary to the thin RC panel, the confinement affords the column large levels of strength and ductility until deformations are large enough to break the CFRP wraps. Four tests are of interest, corresponding to the number of CFRP wraps placed around the column (0 wraps, 2 wraps, 4 wraps, and 6 wraps).

This paper is organized into five sections. Section 2 provides a brief description of the numerical methods employed in the FEM calculations, the constitutive models, and the FEM models developed for each test. Comparisons produced using each constitutive model are shown for simple triaxial compression and unconfined tension load paths. Triaxial compression behaviors are important in highly confined RC structures while unconfined tension behaviors can dominate in lightly confined RC structures. In Section 3, the results of the FEM calculations are presented. Section 4 provide a brief discussion of the results and Section 5 states our conclusions.

2. METHODS

FEM Solver

A structural dynamics code solving the nonlinear continuum mechanics equations is used to compute response predictions for concrete structures under transient loads. The conservation of momentum equation is solved:

$$\rho \cdot \ddot{u}_i = \sigma_{ij,j} + b_i \quad (1)$$

where ρ is the density, $\ddot{u}_i$ is the acceleration, $\sigma_{ij,j}$ are internal forces, and b_i are external forces acting on the body.

Equation (1) is solved in the Lagrangian framework using the Galerkin weighted residual approximation [1]. This results in the following weak form of the partial differential equations:

$$\int_{\Omega_x} \delta u_i \cdot \rho \cdot \ddot{u}_i \cdot d\Omega + \int_{\Omega_x} \delta u_{i,j} \cdot \sigma_{ij} \cdot d\Omega - \int_{\Omega_x} \delta u_i \cdot b_i \cdot d\Omega - \int_{\Gamma_x^h} \delta u_i \cdot b_i \cdot d\Gamma = 0 \quad (2)$$

Equation 2 is then discretized in space and time using the FEM. In the Lagrangian frame, mass and energy are conserved and a constitutive model provides closure of the system of equations implied by Equation (2). In its semi-discrete form, Equation (3) typically appears as:

$$\mathbf{M} \cdot \Delta \ddot{\mathbf{u}} + \mathbf{K} \cdot \Delta \mathbf{u} = \Delta \mathbf{f} \quad (3)$$

The semi-discrete Equation (3) is then solved in time using a predictor-corrector algorithm [1]. For the types of problems of interest in this paper, the explicit approach is used, which is implemented in LS-DYNA.

Constitutive Models

Each of the constitutive models available in LS-DYNA are formulated using plasticity [21]. The differences between them are subtle and manifested in how the deviatoric and volumetric responses are characterized. The response of concrete depend on pressure and the general form of the plastic yield function is written as:

$$Y(I_1, J_2, J_3) = 0 \quad (4)$$

where I_1 is the 1st invariant of stress tensor ($I_1 = tr(\sigma_{ij})$), which represents volumetric response; J_2 and J_3 are the 2nd and 3rd invariants of deviatoric stress tensor ($\sigma_{ij} = s_{ij} + I_1 \delta_{ij} / 3$, $|s_{ij} - \lambda \cdot \delta_{ij}| = \lambda^3 - J_1 \lambda^2 - J_2 \lambda - J_3 = 0$).

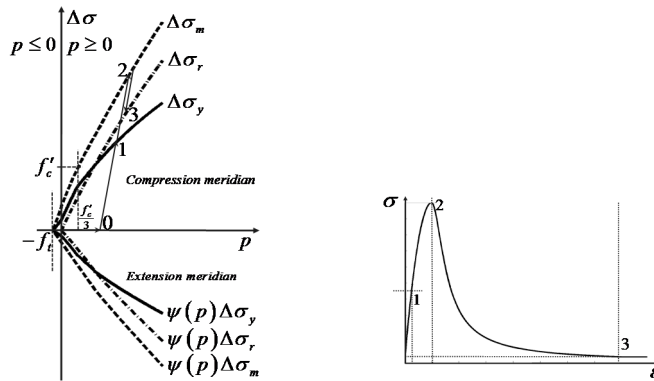
K&C Concrete Model (KCCM)—The KCCM was first released in DYNA3D in 1994 [7] and was motivated by the need for response calculations for RC structures under blast and impact effects. In MAT072_REL3 a plastic surface is derived by interpolating between two of three independent surfaces using an internal damage variable, λ . The three surfaces denoted by $\Delta\sigma_y$, $\Delta\sigma_m$, and $\Delta\sigma_r$ represent yield, maximum, and residual stress difference, respectively where $\Delta\sigma = \sqrt{3 \cdot J_2}$. The yield function is defined as:

$$Y(I_1, J_2, J_3, \lambda) = \sqrt{3 \cdot J_2} - \gamma(I_1, J_3, \lambda) \quad (5)$$

where

$$\gamma = \begin{cases} r(J_3) \cdot [\eta(\lambda) \cdot (\Delta\sigma_m - \Delta\sigma_y) + \Delta\sigma_y] & \lambda \leq \lambda_m \\ r(J_3) \cdot [\eta(\lambda) \cdot (\Delta\sigma_m - \Delta\sigma_r) + \Delta\sigma_r] & \lambda > \lambda_m \end{cases} \quad (6)$$

where $\eta(\lambda)$ is a nonlinear function that ranges from zero to unity for $\lambda \leq \lambda_m$ and from unity to zero when $\lambda > \lambda_m$, and λ is accumulated as a function of the effective plastic strain using three damage accumulation parameters: b_1 , b_2 , b_3 [7]. The three surfaces are illustrated in Figure 1a, along with an arbitrary triaxial compression loading path in the pressure ($p=I_1/3$) and stress difference, $p-\Delta\sigma$, plane.



(a) Plastic surfaces in the $p-\Delta\sigma$ plane. (b) Idealized uniaxial stress-strain behavior.

Figure 1—Illustration of the function of the plastic surfaces for MAT072_REL3.

The three independent surfaces, denoted here as $\Delta\sigma_i$, use a simple function to account for the effects of pressure and are calibrated to triaxial compression data. Three parameters a_{0i} , a_{1i} , and a_{2i} (9 parameters total for the three surfaces) define each of the failure surfaces:

$$\Delta\sigma_i = a_{0i} + \frac{p}{a_{1i} + a_{2i} \cdot p} \quad (7)$$

As seen in Equation (6), each of the surfaces are functions of J_3 via the function, r , using the formulation proposed by William and Warnke [21]:

$$r = \frac{2(1 - \alpha^2) \cos \theta + (2\alpha - 1) \sqrt{4(1 + \alpha^2) \cos^2 \theta + 5\alpha^2 - 4\alpha}}{4(1 - \alpha^2) \cos^2 \theta (1 - 2\alpha)^2} \quad (8)$$

where α is a parameter that depends on pressure and the Lode angle θ can be determined from:

$$\cos 3\theta = \frac{3\sqrt{3}}{2} \frac{J_3}{J_2^{3/2}} \quad (9)$$

Strain hardening and softening behaviors are efficiently modeled by formulating the plasticity model in this way. The $\eta(\lambda)$ damage function provides the mechanism by which MAT072_REL3 may reproduce hardening until the damage is sufficient to reach surface $\Delta\sigma_m$ and then softening until $\Delta\sigma_r$ is reached. Such a feature allows modeling brittle, ductile, and brittle-to-ductile transition behaviors. Figure 1b shows an illustration of a typical axial stress (σ or σ_{11}) versus axial strain (ϵ or ϵ_{11}) response for a generic material sample subjected to loading. Subsequent to loading the

material specimen hydrostatically (Point 0), sample behavior is approximately linear up to roughly 35-65% of its peak strength (Point 0 to Point 1) and then exhibits strain hardening up to the peak strength (Point 2). Strain-softening may be observed thereafter to a residual strength whose value also depends on the level of confinement (Point 3 and on). Figure 1a illustrates the functionality of Equation (6), where γ (the current yield surface) is interpolated between the yield and maximum surfaces when $\lambda \leq \lambda_m$ and the maximum and residual surfaces for $\lambda > \lambda_m$.

For the trial stress state in the plasticity algorithm, the plastic potential function is defined as:

$$g = \sqrt{3 \cdot J_2^*} - \omega \cdot \gamma(I_1^*, J_3^*, \lambda) \quad (10)$$

where ω is a dilatancy parameter (0 for non-associative, 1 for fully associative). By defining the plastic potential function in this way, the parameter ω functions to control the macroscopic behavior of shear-induced dilatancy. Using Equation (9), one can then derive a consistency condition for the plasticity algorithm [7] [9]. A classical radial return algorithm is then used to correct the trial stress.

MAT072_REL3 was developed with the intent of being used in conjunction with any equation of state (EOS) model that could capture volumetric hardening, and for this reason, is a very simple and flexible model capable of modeling various types of cementitious or geological materials. Hydrostatic compression tests are typically used to obtain pressure versus volume strain pairs for the EOS. Additionally, the MAT072_REL3 incorporates rate effects via a radial rate enhancement to the plastic surface based on the effective deviatoric strain rate and the user-defined DIF curve. This provides a means to enhance strength as a function of loading rate that is easily calibrated to SHPB data.

Winfrith Concrete Model (WCM)—The WCM (MAT084) was developed in the 1980s for computing response of RC structures subjected to impact loadings, and was implemented into LS-DYNA in 1991. This model has mainly been applied in obtaining responses of RC structures subjected to impact loadings [22]. The Winfrith concrete model is based upon the so-called four parameter model proposed by Ottosen [23]:

$$Y(I_1, J_2, J_3) = aJ_2 + \xi\sqrt{J_2} + bI_1 - 1 \quad (11)$$

with

$$\xi = \begin{cases} k_1 \cos\left[\frac{1}{3}\cos^{-1}\left(k_2 \cos(3\theta)\right)\right] & \cos(3\theta) \geq 0 \\ k_1 \cos\left[\frac{\pi}{3} - \frac{1}{3}\cos^{-1}\left(-k_2 \cos(3\theta)\right)\right] & \cos(3\theta) \leq 0 \end{cases} \quad (12)$$

The four parameters a , b , k_1 , and k_2 , are functions of the ratio of f_t / f'_c , and they are determined from uniaxial compression (correspondingly, $\theta = 60^\circ$), uniaxial tension ($\theta = 0^\circ$), biaxial compression ($\theta = 0^\circ$), and triaxial compression ($\theta = 60^\circ$) tests.

Continuous Surface Cap Model (CSCM)—Aimed at roadside safety analyses, MAT159 was developed in the 1990s and was made available in LS-DYNA around 2005. The CSCM couples the shear (failure) surface with a hardening-compaction surface (known as a “cap”) using a multiplicative formulation. The yield function is also defined in terms of three stress invariants [24]:

$$Y(I_1, J_2, J_3) = J_2 - \Re(J_3)^2 F_f^2(I_1) F_c(I_1, \kappa) \quad (13)$$

Here $F_f(I_1)$ is the shear failure surface, $F_c(I_1, \kappa)$ is the hardening cap with κ to be the cap hardening parameter, and $\Re(J_3)$ is the Rubin three-invariant reduction factor. The multiplicative form allows the cap and shear surfaces to be combined continuously and smoothly at their intersection.

The shear failure surface $F_f(I_1)$ is defined as:

$$F_f(I_1) = \alpha - \lambda \exp^{-\beta I_1} + \theta I_1 \quad (14)$$

The material constants α , β , λ , and θ are determined from triaxial compression test data.

The cap hardening surface is expressed as:

$$F_c(I_1, \kappa) = \begin{cases} 1 - \frac{(I_1 - L(\kappa))^2}{(X(\kappa) - L(\kappa))^2} & I_1 \geq L(\kappa) \\ 1 & I_1 \leq L(\kappa) \end{cases} \quad (15)$$

with

$$L(\kappa) = \begin{cases} \kappa & \kappa \geq \kappa_0 \\ \kappa_0 & \kappa \leq \kappa_0 \end{cases} \quad (16)$$

$$X(\kappa) = L(\kappa) + RF_f(I_1) \quad (17)$$

Eq. (13) describes an elliptical cap for $I_1 \geq L(\kappa)$. The shear failure surface intersects the cap at $I_1 = L(\kappa)$. κ_0 is the value of I_1 where the shear surface and the cap intersect initially (before cap expands or shrinks). The cap expands (i.e., $X(\kappa)$ and κ increase) when plastic volume compaction occurs, and the cap shrinks (i.e., $X(\kappa)$ and κ decrease) when plastic volume dilation occurs. The motion of the cap is controlled by the hardening rule specified by:

$$\varepsilon_v^p = W \left[1 - \exp^{(-D_1(X-X_0) - D_2(X-X_0)^2)} \right] \quad (18)$$

where ε_v^p is the plastic volumetric strain, W is the maximum plastic volumetric strain, X_0 is the initial location of the cap when $\kappa = \kappa_0$. The five parameters, X_0 , R , W , D_1 , and D_2 , are determined from hydrostatic compression and uniaxial strain tests.

Reidel-Hiermaier-Thoma Model (RHTM)—The RHTM model (RHT in LS-DYNA, MAT272) was designed for use in wave propagation codes and validated with penetration data [25]. The model was implemented made available in LS-DYNA in 2011 [26]. Similar to the KCCM, the model has an initial yield plastic surface, a maximum plastic surface, and a residual plastic surface termed the friction surface, which are interpolated using an effective plastic strain, ε_p , and the strain-rate, $\dot{\varepsilon}$. The yield function is formulated similar to that defined in the CSCM and is given by:

$$Y(I_1, J_2, J_3) = \sqrt{3 \cdot J_2} - f'_c \cdot \sigma_Y(I_1, \varepsilon_p, \dot{\varepsilon}) \cdot R_3(J_3) \quad (19)$$

where

$$\sigma_Y = \sigma_F \cdot \chi \quad (20)$$

$$\sigma_f = \begin{cases} A \left(p^* - F_r / 3 + (A / F_r)^{-1/n} \right)^n \\ F_r \cdot f_s^* / Q_1 + 3p^* (1 - f_s^* / Q_1) \\ F_r \cdot f_s^* / Q_1 - 3p^* (1 / Q_2 - \frac{f_s^*}{Q_1 \cdot f_t^*}) \\ 0 \end{cases} \quad (21)$$

$$\chi = \varepsilon_p + (1 - \varepsilon_p) \cdot F_e \cdot F_c \quad (22)$$

and $p^* = p / f'_c$, R_3 is given by Equation (8), and

$$F_e = \begin{cases} g_c^* - \frac{3p^* - F_r^c \cdot g_c^*}{F_r^c \cdot g_c^* + F_r^c \cdot g_t^* \cdot f_t^*} \cdot (g_t^* - g_c^*) \\ g_t^* \end{cases} \quad (23)$$

F_c defines the cap surface and is computed in a manner similar to Equation (15), except that motion of the cap is evolved using a parameter representing the porosity, α , using the Mie-Grunesien form of a polynomial Hugoniot curve and p - α relation. A good description of the RHTM's implementation in LS-DYNA and each of the RHTM parameters

is provided in [26]. Because the formulation of the model is quite complex, the reader is referred to that document to gain a better understanding of this formulation.

Comparison of Models Under Simple Load Paths

Figure 2 shows measurements for concrete under triaxial compression and unconfined uniaxial tension from a concrete material characterization program described in [27]. The measurements are normalized by the compressive and tensile strength of that concrete. The behaviors predicted with each material model described in the Introduction (Page 3) are shown in Figure 3 to Figure 5, which were computed by subjecting a 2.25 inch (57.15 mm) single cube element (i.e., a single point integration) to each load path. The following observations are made from these figures:

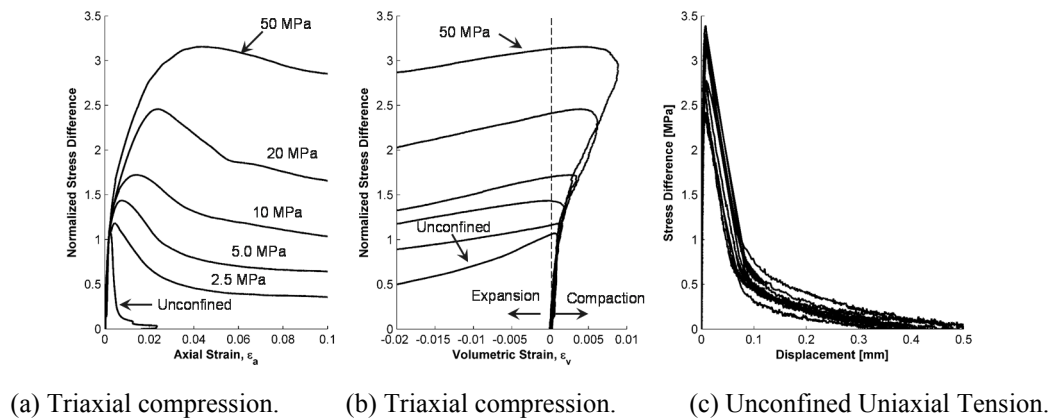
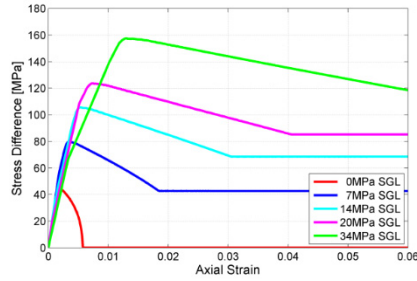


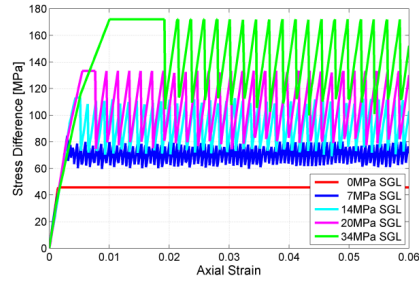
Figure 2—Measured concrete behaviors in a recent material characterization program.

- Each model displays increases in $\Delta\sigma$ with increasing confining pressure and axial strain, as should be expected; however, each computes different post-peak behaviors. MAT084, for example, does not strain-soften, while the others do. MAT159 exhibits abrupt strain-softening to a residual strength, while MAT072_REL3 and MAT272 display a more gradual strain-softening more consistent with test data (e.g., Figure 2a).
- Each concrete model computes a peak strength equivalent to f'_c under zero confining pressure except for MAT272. In other words, employing the simple-input feature with MAT272 does not return the desired unconfined compressive strength, while the others do.
- Of each concrete model, only MAT072_REL3 and MAT272 account for dilatancy. This is observable when plotting $\Delta\sigma$ versus volumetric strain (Figure 2b and Figure 4). MAT072_REL3 begins dilating before the peak strength is reached, while MAT272 appears to do this only after reaching its residual strength. The former is more consistent with the test data shown in Figure 2b.
- In tension, each model produces strain-softening behaviors but differ in the magnitude of the stress and energy dissipated (see Figure 5). The following quantities are computed: MAT072_REL3, $f_t=3.81$ MPa, $G_f=99.9$ N/m; MAT084, $f_t=3.78$ MPa, $G_f=243$ N/m; MAT159, $f_t=3.01$ MPa, $G_f=85.2$ N/m; MAT272, $f_t=5.88$ MPa, $G_f \gg 800$ N/m. Based on CEB-FIP, recommended values are $f_t=3.00$ MPa, $G_f=95.7$ N/m for a concrete having 40 MPa compressive strength and a MAS of 19 mm (0.75 inch). The data in Figure 2c measured: $f_t=2.75$ MPa, $G_f=212.5$ N/m.

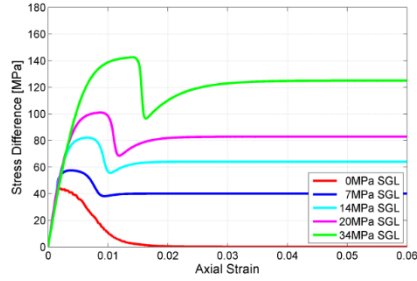
A more complete set of comparisons for these concrete models, including MAT072_REL3, MAT159, MAT272, and MAT084, as well as some other models available in LS-DYNA, under various common mechanical tests are given in a number of other references [3] [4] [5] [6]. Because these results and our own experience shows that MAT072_REL3 affords effective means to compute these basic behaviors for concrete materials, including computing responses for shear dilation behaviors and confinement effects, most of the calculations shown in the remainder of the paper use this model. This does not mean that acceptable results could not be obtained using these other models given enough understanding and experience with each model.



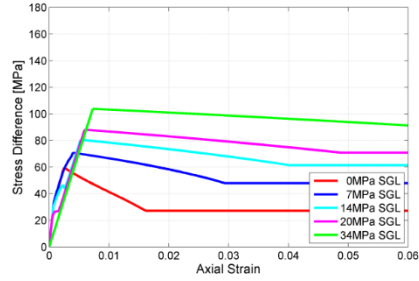
(a) KCCM (MAT072_REL3).



(b) WCM (MAT084).

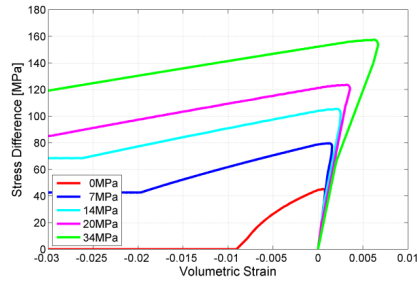


(c) CSCM (MAT159).

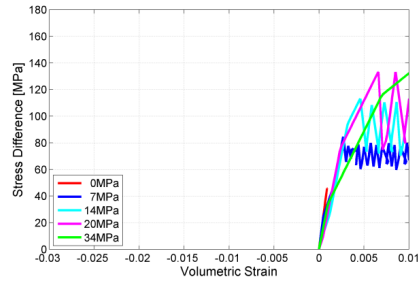


(d) RHTM (MAT272).

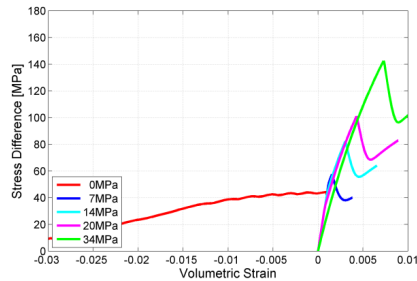
Figure 3—Stress-difference versus net axial strain computed using each constitutive model under TXC.



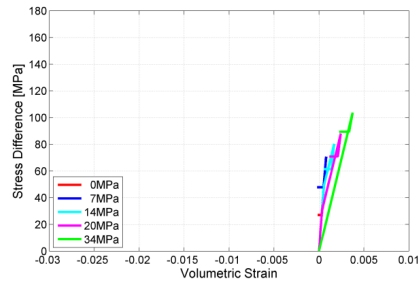
(a) KCCM (MAT072_REL3).



(b) WCM (MAT084).



(c) CSCM (MAT159).



(d) RHTM (MAT272).

Figure 4—Stress-difference versus net volumetric strain computed using each constitutive model under TXC.

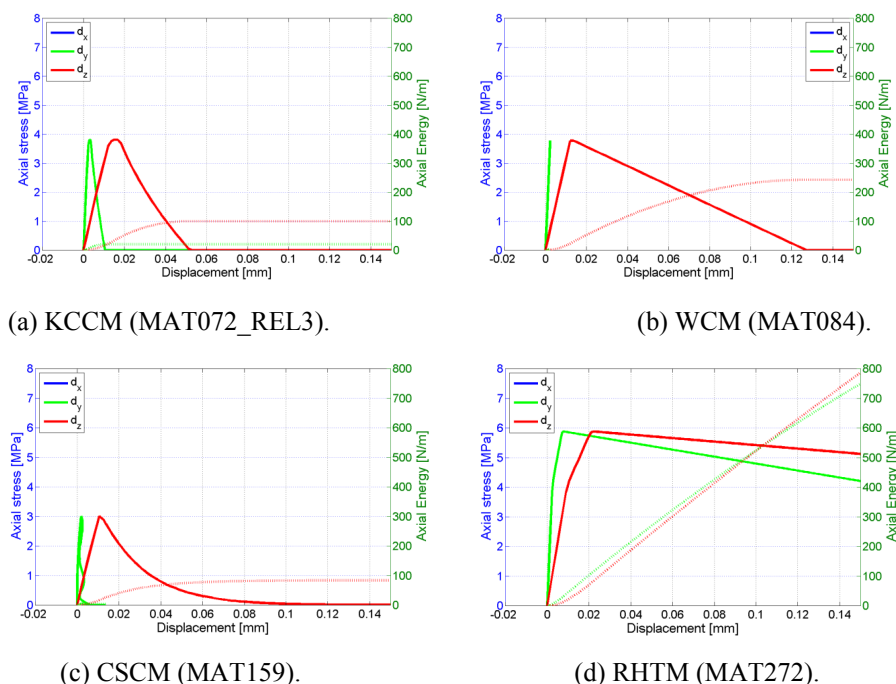


Figure 5—Axial stress versus displacement computed using each constitutive model under uniaxial tension.

FEM Models

Finite element models developed for each of the tests are shown in Figure 6 to Figure 8. An unconfined compressive strength of 40 MPa (5,800 psi) is used as input to the constitutive model for the compression, split-tension, and SHPB tests, unless the test specified otherwise. The KCCM internally determined the tensile strength of the concrete to be 3.52 MPa (510 psi), based on a relationship similar to that proposed by CEB-FIP [11]. In the numerical model, the elements are integrated at a single point for each element with viscous or stiffness-based stresses introduced to control hourglass deformation while minimizing hourglass energies. Viscous-based hourglass was used for all test except the split-tension model, where stiffness-based hourglass was needed to minimize hourglass deformations.

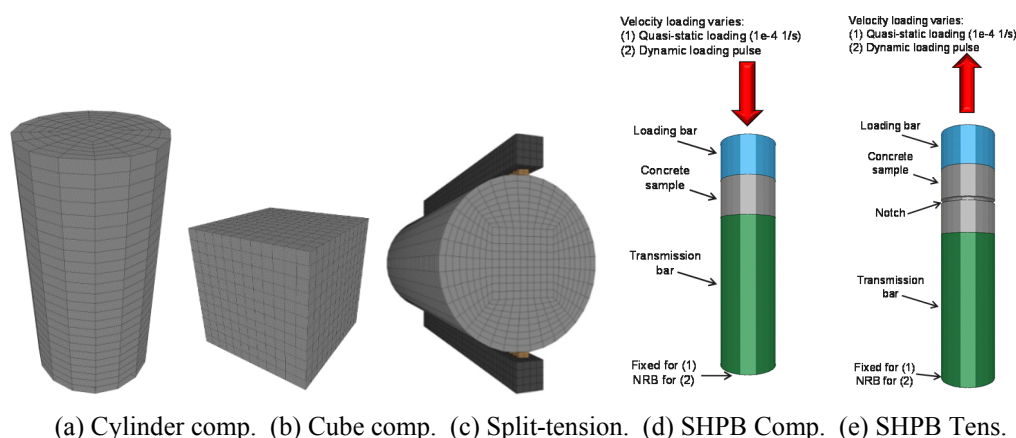


Figure 6—FEM models for compression, split-tension, and split-Hopkinson pressure bar tests.

Standard Cylinder Compression Test—The cylinder compression model (Figure 6a) uses a cylinder that is 152-mm (6-inch) in diameter, and 305-mm (12-inch) tall. The model is discretized using elements that are on average 10 mm cubed. The top and bottom faces of the cylinder are prescribed boundary conditions that are either unrestrained (i.e., allowed to translate freely in the plane of the cross section) or restrained (i.e., not allowed to translate in the plane of the cross section). The unrestrained condition is representative of ASTM C 1231, where neoprene caps prevent friction, while the restrained case is an upper bound of ASTM C 39, where some restraint is usually present.

Compression is applied to the model by applying a constant downward velocity along the axis of the cylinder to the top nodes of the model at a rate of 1.27 mm/s (0.05 inch/s). Initial simulations indicated that the results obtained using this model were not sensitive to this applied loading rate, which is faster than those specified in the ASTM standards, nor sensitive to the chosen range of hourglass coefficients, at least early into the post-peak response.

Standard Cube Compression Test—The cube compression model (Figure 6b) follows DIN 1048, using a 150-mm (5.9-inch) cube specimen. It has element dimensions that are 12.7-mm (0.5-inch) cubed. Identical boundary conditions applied to the cylinder compression model are applied to this cube compression model, including the downward loading rate of 1.27 mm/s (0.05-inch/s). As for the cylinder, initial simulations confirmed that the results from this model were not significantly affected by the applied loading rate or the coefficient for hourglass control chosen, at least until failure takes place.

Standard Splitting Tension Test—The model for the splitting tension test uses a similar 152-mm (6-inch) diameter by 305-mm (12-inch) finite element mesh loaded in the radial direction of the cylinder as shown in Figure 6c. The DIN 1048 and ASTM C 496 specifications for this test are nearly identical in the overall dimensions of the specimen. The only significant difference are in the dimensions of the plywood bearing strips (ASTM C 496 specifies 3.2 mm (0.13-inch) thick by 25 mm (1-inch) wide strips while DIN 1048 specifies 5 mm (0.2-inch) thick by 10 mm (0.4-inch) wide plywood strips) and the applied loading rate (0.70-1.40 MPa/min for the DIN standard to ASTM C 496's 1.8-4.2 MPa/min). For this model, a finer element discretization is used in the finite element mesh, consisting of 6.35 mm elements on average to resolve the localized stress distribution near the loading points. The splitting tension loading is represented by prescribing a velocity boundary condition to an elastic steel platen located at the top of the cylinder, while another elastic steel platen located at the bottom of the cylinder is held fixed. Two ASTM C 496 shims were given elastic properties of plywood (Elastic Modulus, $E = 1,300$ MPa; Poisson's Ratio, $\nu = 0.10$), which directly contact the cylinder and the lower and upper platens. The orientation of the cylinder was selected so that one concrete element contacts the wood shims using a contact surface; this avoided numerical problems observed in earlier mesh designs occurring from contact surface "chatter" and excessive hourglass energies in the concrete elements in contact with the wood shims.

Split-Hopkinson Pressure Bar Test—The SHPB test uses a 76.2 mm (3 inch) sample diameter. The models are shown in Figure 6d and Figure 6e, in compression and tension respectively. The compression sample exhibits 1:1 length-to-diameter ratio and is placed in contact with the incident and transmission bars by a frictionless contact surface. The tension sample is longer, as shown in Figure 6e, and adhered to the bars having shared nodes with the incident and transmission bars. The bars are Aluminum and treated as elastic ($\rho = 2.70$ g/cc, $E = 69,000$ MPa, $\nu = 0.355$). Two series of loadings are of interest: (a) idealized quasi-static and dynamic loadings on the simplified SHPB geometries shown in Figure 6d and Figure 6e, to illustrate the effect of inertia; and (b) three stress-pulses measured in actual SHPB tests and applied to a model of the entire length of the SHPB test, to compare with the test data. The idealized loading variants are performed by either (a) a quasi-static loading ($1.0e^{-4}$ s $^{-1}$) by way of a constant velocity applied to the incident bar (while holding the opposite end of the transmission bar fixed) and (b) a dynamic loading pulse that induces global strain rates of approximately 100 s $^{-1}$ (using a triangular velocity pulse applied to the bar [8]).

Thin RC Panel Under a Blast Load—The RC panel (Figure 7) is 4-in thick, 33.75-in wide, 64-in tall (0.1016 m x 0.8573 m x 1.625 m), and reinforced with #3 ASTM Gr. 60 longitudinal reinforcing bar at an average spacing of 4 inches (0.1016 m) on center and horizontal bars at an average spacing of 12 inches (0.3048 m) on center. The unconfined compressive strength of the concrete is 34.5 MPa (5,000 psi) and the reinforcement is Gr. 60 steel. The concrete in the slab is discretized with 25.4 mm (1 inch) cube elements; the reinforcement is modeled by 25.4 mm (length) Hughes-Liu beam elements sharing nodes with the concrete. The structural tubes used by the reaction structure to support the slab are discretized by 25.4 mm squared shell elements; the top and bottom support frames are modeled by 25.4 mm cubic solid elements as well.

For the material modeling, the concrete is modeled using all four constitutive models, since each can produce strain-softening behaviors in tension. The Gr.60 reinforcement is modeled using a piecewise linear von Mises plasticity model, and the structural tubes and support frames are modeled as ASTM A36 steel with the same model.

The two ends of the structural tubes are fixed and the support frames are fixed at their back face, as can be seen in Figure 7. Contact interfaces are defined between the slab and the structural tubes and support frames. LS-DYNA's single-surface contact is defined between the slab and the structural tubes and support frames using a coefficient of friction (COF) of 0.25. Results for one other contact algorithm with various COF values are also computed.

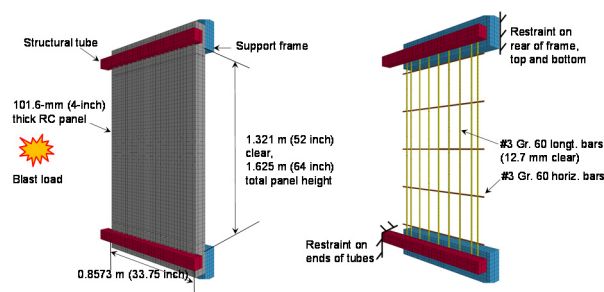


Figure 7—FEM model for the RC panel blast test.

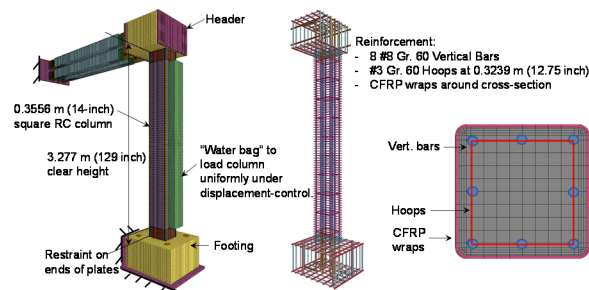


Figure 8—FEM model for the quasi-static RC column tests.

Confined RC Column Under a Monotonically-Increasing Uniform Lateral Load—The RC column model includes the 14-in (0.3556-m) square, 129-in (3.277-m) tall column, its header and footing, and the link beams supporting the header, as they appeared in the test. The reinforcement is indicated in the Figure 8. The concrete is 38.6 MPa (5,600 psi) and modeled with 38 mm (1.5 in) cube elements. Results are obtained with the KCCM, CSCM, and RHTM; results for the WCM are not shown primarily because of its lack of computing shear-dilatancy (Figure 4). The CFRP wraps are modeled using truss elements oriented in the hoop direction, as they are unidirectional materials. When CFRP wraps are applied, the four column corners are rounded to a 1-inch radius as in the actual test specimens (Figure 8); this is necessary to resolve the confining stresses induced in the concrete by the CFRP wraps. The same material models used in the RC panel calculations are used for the concrete and steel. The CFRP are modeled as an elastic material with brittle failure.

The column is loaded as shown in Figure 8 with the following procedure. An axial load of 100 kips is applied at the outset as was done in the tests (except for Test 5). Then a uniform pressure was applied over the face of the column to mimic the lateral loading applied in the test with three displacement-controlled hydraulic loaders. The loading is imparted in such a manner as to achieve this loading through an artifice of a “water bag;” a uniform displacement is applied to the outer surface of a water bag placed over the front face of the column model. This artifice provides a means to apply a uniform pressure loading to the column (i.e., as measured by the water pressure), while still being able to capture any softening behavior that might occur.

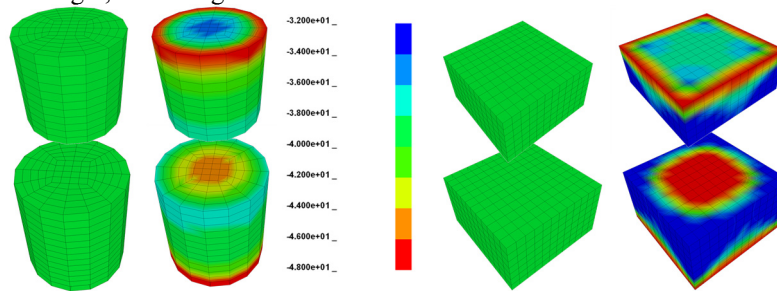
3. RESULTS

Standard Cylinder and Cube Compression Results

If the cylinder is compressed using platens at the top and bottom ends that do not impart any lateral restraint (e.g. by following ASTM C 1231), the vertical compressive stress in the cylinder is uniform within the whole cylinder, at least until the peak stress is reached. This is shown in Figure 9a, which shows the cylinder model with a uniform fringe of vertical stress near the time the peak strength is reached. The average calculated stress-strain response is shown in Figure 10a, which shows that the unrestrained model achieved its 40 MPa compressive strength. This model (and the test it represents) is therefore consistent: the compressive strength used as input material property for each element is reproduced in the cylinder structural test. To assess the radial variation of stress, 4 integration points are chosen on a horizontal plane near mid-height, at 4 normalized radial distances r/R (radial distance r from the center divided by the radius R) of 0.10, 0.44, 0.70, and 0.95. Figure 10b shows a uniform (normalized) stress-strain response for all 4 points, at least while the cylinder is stable (until just past peak load). Under compression, the cylinder expands laterally but retains its cylindrical shape.

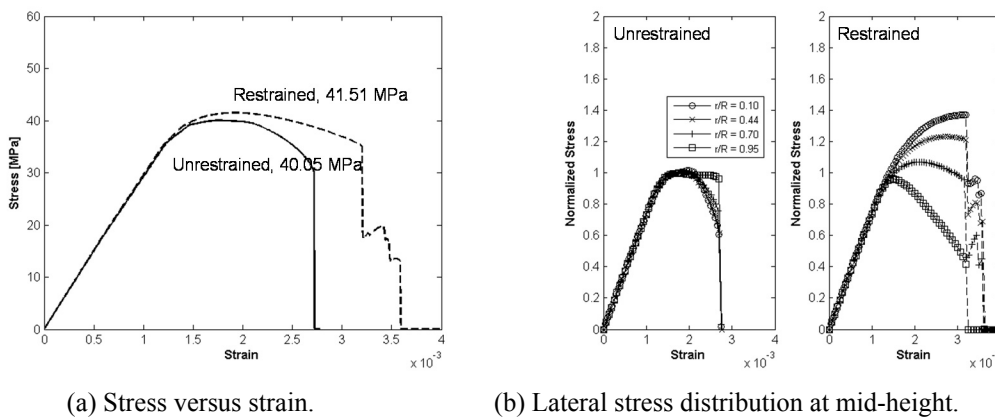
The restrained cylinder results in the center of the cylinder becoming stronger, a non-uniform stress distribution, and a deformed barrel shape (Figure 9b). Figure 10a also shows the calculated average stress-strain response for the perfectly restrained case. A compressive strength of 41.5 MPa is reached, which is 3.65% higher than in the unrestrained case. While this is slightly higher than the input material property (40 MPa), the model is still mostly consistent, indicating that the specimen shape and end restraint had little effect in the determination of a value that can be considered a material property. However, Figure 10b shows that the stress-strain response at the same 4 points along a radius near mid-height is very non-uniform. The location near the center ($r/R=0.10$) reaches a strength about 40% higher than the measured compressive strength, whereas the location near the free edge ($r/R=0.95$) started decaying long before the compressive strength was reached.

Similar behaviors are observed in the cube specimens under compression, except that the restrained cube displays a much higher peak strength than that obtained using the cylinder in both the restrained and unrestrained end conditions. If the platens at the top and bottom of the cube do not impart any lateral restraint, the vertical stress is effectively uniform (Figure 9c). The average stress-strain response is shown in Figure 11a, which indicates a peak strength of 40.04 MPa. For a perfectly restrained boundary condition, the numerical model shows sharp gradients in vertical stress throughout the body of the cube (Figure 9d). The average stress-strain response for this restrained case is also shown in Figure 11a, which indicates a peak strength of 61.6 MPa, which is 54% higher than the compressive strength input to the material model. This ratio is the approximately the same as the upper bound of 1.54 for the experimental ratios indicated earlier. Figure 11b shows the variation of stress radially at four integration points selected on a horizontal plane near midheight, at 4 normalized radial distances r/R (radial distance r from the center divided by the distance R to the corner of the cube, or 106 mm) of 0.08, 0.42, 0.75, and 0.92. For the unrestrained case, the stresses were all uniform. For the restrained case, the location near the center ($r/R=0.08$) reaches a strength nearly 300% higher than the input compressive strength, indicating that the amount of restraint influences the measured material property.



(a) Cyl., unrestrained. (b) Cyl., restrained. [MPa] (c) Cube, unrestrained. (d) Cube, restrained.

Figure 9—Vertical stress contours in cylinder and cube specimens.



(a) Stress versus strain.

(b) Lateral stress distribution at mid-height.

Figure 10—Predicted strength of concrete cylinder.

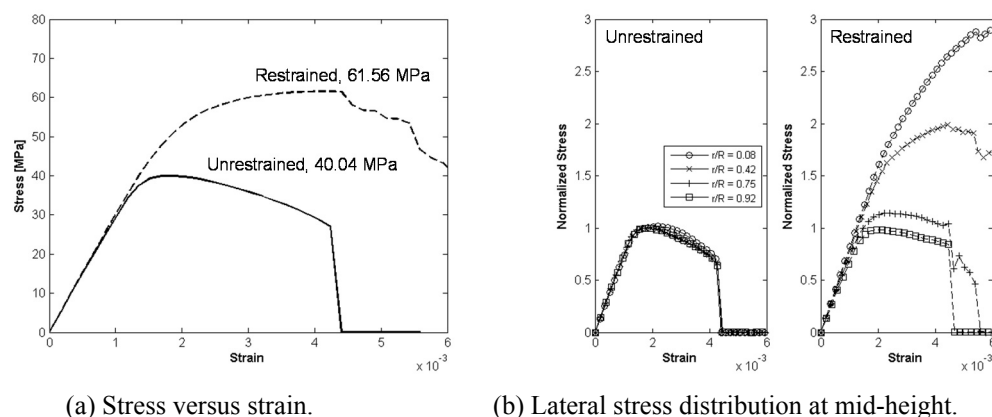


Figure 11—Predicted strength of concrete cube.

Standard Splitting Tension Results

The splitting tension model indicates crack formation along the centerline of the cylinder in the direction of the applied loading. Figure 12 shows fringes of the model's damage parameters at crack formation (a) and after failure (b). The crack plane is made visible by the elements having the maximum damage (2.0) indicating a complete loss of cohesion. Figure 12c shows the p - $\Delta\sigma$ trajectories for four integration points selected along the crack plane just prior to crack formation. The points at a r/R of 0.95 and 0.00 represent points closest to the bearing strips and at the center of the cylinder, respectively. The plot also shows the KCCM's $\Delta\sigma_m$ and an ideal unconfined uniaxial tension loading path. This plot shows that strength is limited by (1) $\Delta\sigma_m$ in the compressive pressure space away from the unconfined uniaxial tension path and (2) the damage starts at the center and moves outwards towards the strips.

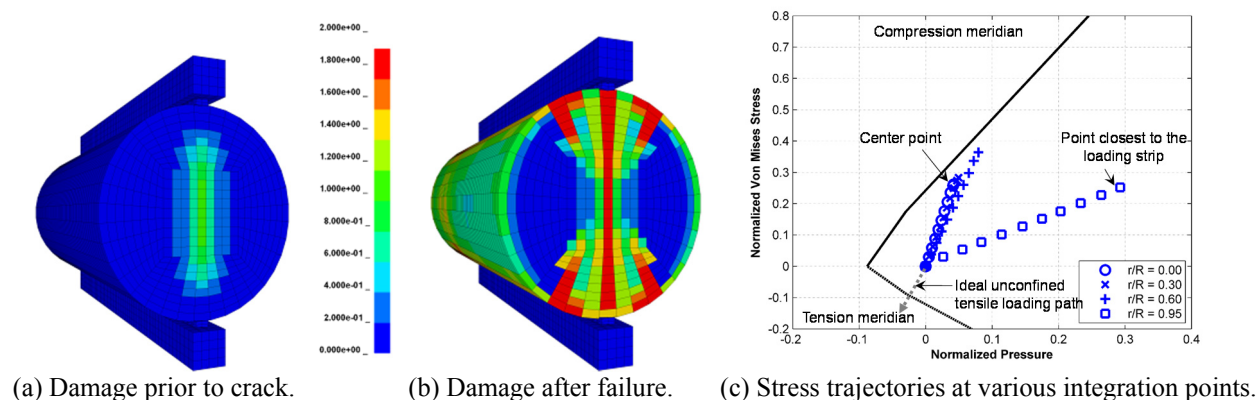


Figure 12—Damage before and after splitting.

The computed results are highly influenced by the coefficient used for the hourglass control. The elements adjacent to the loaded ones are being deformed into an hourglass mode, and it is necessary to prevent such deformation without using a significant amount of the total energy. Figure 13a shows the normalized splitting stress obtained from the model as a function of total displacement of the steel platen for four selected coefficients of hourglass control (0.02, 0.14, 0.22, and 0.24). The normalized splitting stress was obtained by computing f_r/f_t where:

$$f_r = \frac{2 \cdot P}{\pi \cdot l \cdot d} \quad (24)$$

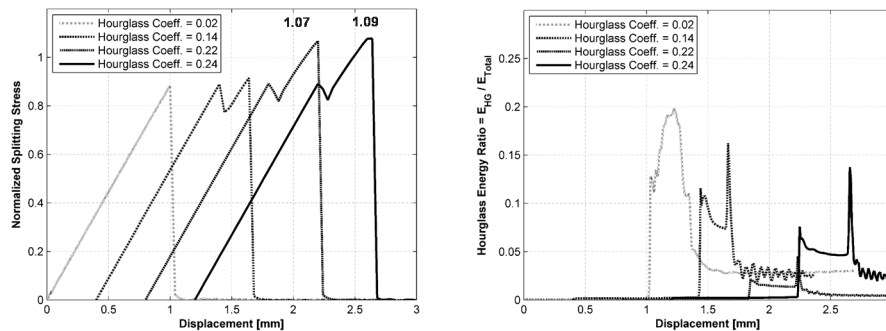
and P = Total computed load, L = Cylinder length = 152 mm, d = Cylinder diameter = 305 mm.

As is indicated in Figure 13a, crack formation is initiated around 90% of the normalized splitting stress, depending on the coefficient used for the hourglass control (note that the curves obtained using each hourglass coefficient have been offset from each other so that the features of each response may be scrutinized individually.) For the largest three coefficients, a brief unloading phase is observed after crack formation that is followed by increasing resistance to the

applied loading. A peak normalized splitting stress of 1.07 and 1.09 is computed with the largest two coefficients, which provided very similar responses. As indicated earlier, the expected range is 1.08 to 1.16.

Figure 14b shows the ratio of hourglass to total energy, E_{HG}/E_{Total} . This figure shows that as the initial crack develops, there is an increase in spurious hourglass energy while the stresses are redistributed. At failure there is a large but transient spike in the ratios, but this is of less interest as the peak strength has already been reached. The lowest hourglass energy yields a normalized splitting stress of 1.07.

Prior to crack formation, the horizontal stresses allow for wide distribution of the vertical compressive stresses. After cracking, the vertical stresses become more concentrated on either side of the vertical crack. Figure 14 shows stress contours in the horizontal (perpendicular to the applied loading) and vertical (parallel to the applied loading) radial directions of the cylinder just prior to the crack formation; the horizontal stresses are primarily tensile while the vertical stresses are compressive. Figure 14 shows stress contours prior to failure, which indicate that loads redistribute around the cracked material and little if any portion of the cylinder exhibits horizontal tensile stresses after the crack forms.



(a) Ratio of splitting to direct (input) tensile strength. (b) Ratio of hourglass to total energy.

Figure 13—Results of splitting tensile strength test.

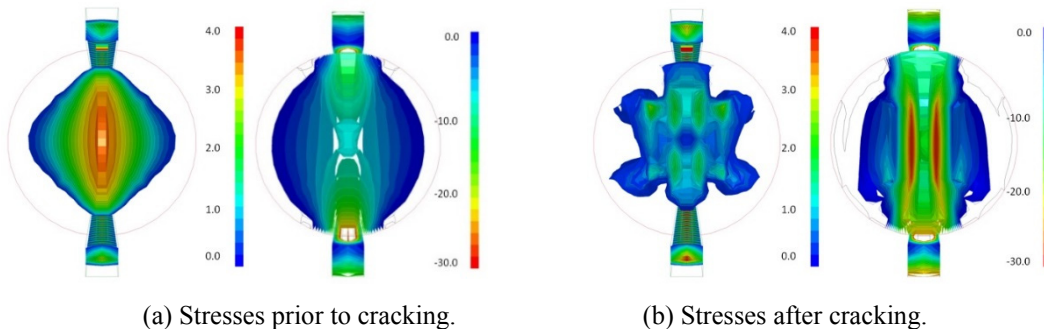


Figure 14—Stresses computed for the ASTM C 496 test (left: horizontal stresses; right: vertical stresses).

SHPB Results

Figure 15 shows results from the SHPB calculations using the idealized model and loadings. Three different mesh sizes are examined using the *rate-independent* version of the KCCM (i.e., by setting the DIF versus strain rate curve equal to unity). In compression, there is a clear increase in strength computed in the simulation when loaded dynamically as compared with the quasi-static case. This indicates that the FE model captures an inertia effect. For tension, no such increase is discernable, which indicates that a DIF function is needed to properly represent the inertia effect. Recently analytic and experimental studies have reinforced these notions [28] [29] [30].

Figure 16 shows results from the calculations using the entire length of the SHPB bars and the actual stress pulses induced in the tests. They show that, in compression, only the initial branch of the DIF is needed to properly represent data from SHPB tests. Figure 16a shows the 76.2 mm diameter SHPB compression test simulated with the detailed FE model of the test and KCCM parameters calibrated to quasi-static material characterization data [27]. The actual loading pulses measured by the incident bar strain gauge are applied to the model for three different loading pulse levels designated “low”, “medium”, and “high” strain rates, and the results are compared with the output stress wave measured by the transmitter bar strain gauge. Three models are used to obtain numerical results for each of the three

compression loading pulses: (1) Model 1, which uses the *rate-independent* model, (2) Model 2, which uses the modified CEB DIF with both branches of the function, and (3) Model 3, which uses the modified CEB DIF with only the first branch of the function (Figure 16b). The resulting transmitted pulses are shown in Figure 16c that can be compared with the experimental data. As expected, Model 1 produces peak strengths that are lower than the experiments (because that model does not account for the rate effects from moisture in the sample). Model 2 produces much higher peak strengths (in essence, double counting for inertia in the sample). Model 3, on the other hand, provides good agreement with the dataset, even though the DIF function (using only the first branch) was not calibrated to this specific concrete (i.e., the modified CEB-FIP model was used). These DIF factors (i.e., using both branches in tension and only the first one in compression) are representative of those needed as input to the model.

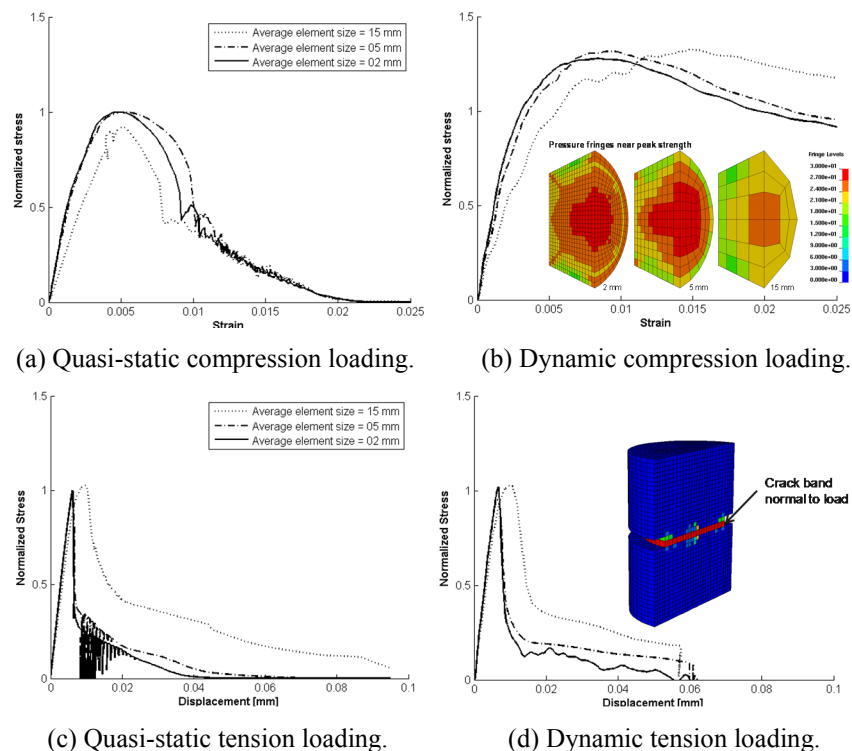


Figure 15—Simulations of a concrete sample subjected to compression and tension; all results are computed using the rate-independent version of the KCC model.

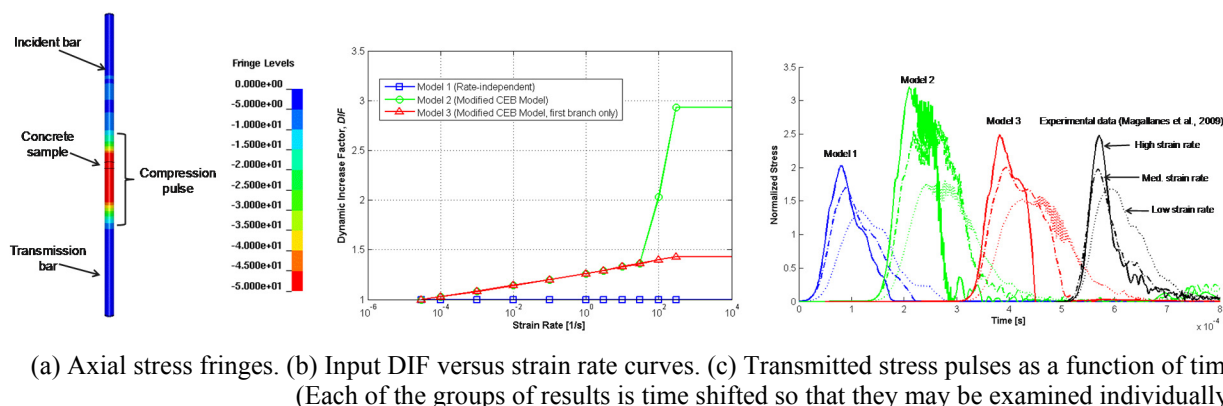


Figure 16—Comparisons of compression SHPB data with simulations using the KCC model.

RC Panel Results

Comparisons of the computed mid-height lateral deflections for the four constitutive models, with and without rate enhancement, are shown in Figure 17, which were obtained in a recent study by Wu et al. [6]. Rate effects are handled

differently by each concrete model. The KCCM uses the DIF curve, while the CSCM and WCM use an internal rate enhancement. For the RHTM, the rate enhancement depends on the reference strain rate (E_{0C} and E_{0T}) on the input cards. For example, the reference rate is $E_{0C}/E_{0T}=10^{-9}$ for the results shown.

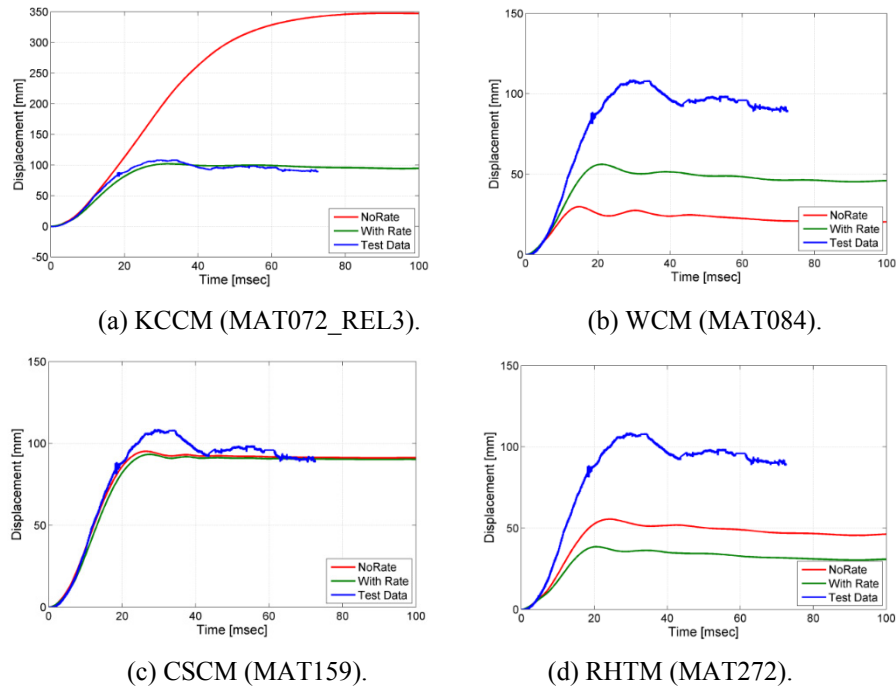


Figure 17—Lateral deflection histories at central mid-height on back face.

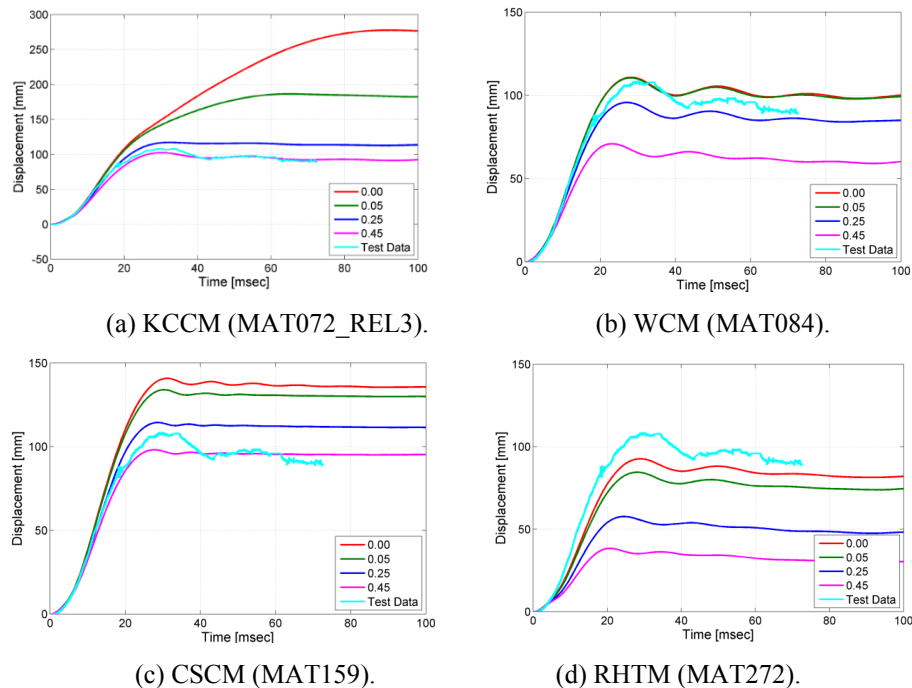


Figure 18—Lateral deflection histories for various coefficients of friction.

The KCCM provides a good match to the deflection-history data when rate effects are included, and when these are ignored, very large deflections are computed. The CSCM also provides good agreement with the data and shows little influence on the predictions when including its rate effect enhancement approach. The WCM and RHTM greatly

under-predict the measured deflections. Moreover, the WCM presents a rather odd trend in that the deflections get larger when its rate effects algorithm is included.

Figure 18 shows the lateral deflection histories computed with each model except using LS-DYNA's automatic surface-to-surface algorithm between the concrete and support tube parts and using various COFs [6]. The KCC model and CSCM compute results that match the test data when the COF is relatively high (0.25 or more), while the WCM and RHTM require a COF value near zero to match the data. The COF between concrete and steel is generally reported to be around 0.45 for quasi-static loads.

RC Column Results

Measured values for the lateral force (P_L) and vertical axial force (P_V) are shown in Figure 19a while those computed using the KCCM, CSCM, and RHTM are shown in Figure 19b-d, respectively. The lateral load is plotted as positive and the axial load, which is compressive, is plotted as negative values. The first column, without any CFRP confinement, results in a brittle shear-failure, which results in a drop in the axial load. As the number of wraps increases, the lateral strength and ductility (by way of its lateral deformation) of the RC column increases.

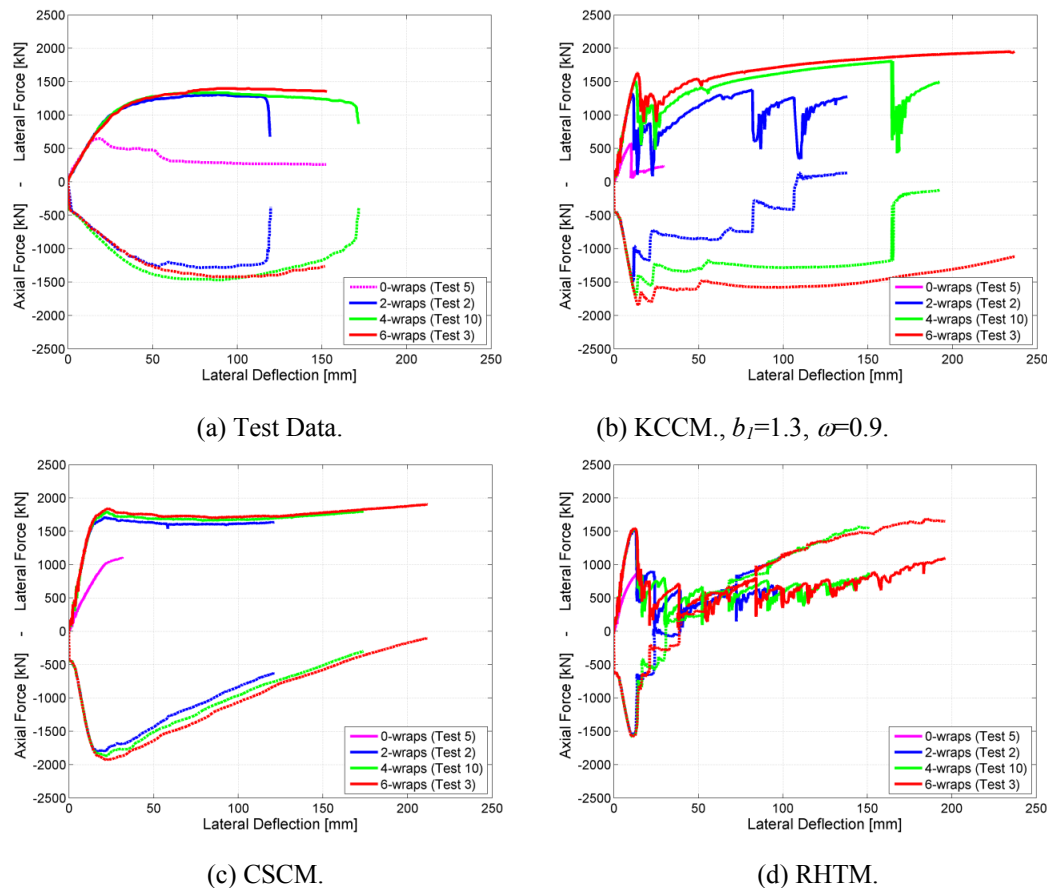


Figure 19—Results computed for the UCSD RC column tests.

Results provided using the KCCM, using a value for a b_l of 1.3 and ω of 0.9, provide a reasonably good estimate of the peak deflections prior to rupturing the CFRP wraps. These two parameters, which control the compression softening and dilatancy behaviors of the model under large confining pressures, were recently calibrated to a different set of tests [5]; without default parameters, the predictions were shown to be poor. While the peak resistance (P_L force) computed is higher than observed in the tests, there is a good correlation between the test and computed response for the overall energy absorption and the deflections associated with FRP rupture. As is discussed below, the initial over prediction of the lateral force (i.e., the deflections around 0.5 to 1-inch (12 to 25 mm) seems to be related to the fidelity of the FE model at the corners, and possibly, the way the CFRP nodes were perfectly tied to the concrete nodes. The P_V force histories deviate somewhat from those measured, particularly in regard to their initial stiffness and their post-

peak responses. The P_r results for Test 2, in particular, show a loss of strength not seen in the test. This may be related to mesh fidelity, an area where more study is needed.

The results computed using the CSCM and RHTM provide very different behaviors. For the CSCM, the predicted strengths are much higher. For the RHTM, they are much too low. Neither of the models predict the failure of the CFRP wraps observed at the larger deformations. Additionally, the RHTM does not show the post-peak behaviors evident in the tests; the increase in strength observed 50 mm occurs from tension-membrane loads developed in the vertical reinforcing bars after all the concrete is damaged.

4. DISCUSSION

The structural tests examined in this paper illustrate that FEM calculations involving concrete structures under transient loads require careful consideration of a host of parameters, many of which have little to do with modeling the concrete material itself.

- With the compression tests on concrete cylinders and cubes, the models using the KCCM model produced results for f'_c that are consistent with observations from tests. That is, the cube specimens are computed to be much stronger than cylinder specimens by 54%. This is attributed to confining pressures developed from the lateral restraint at the ends of the specimens. Faithful treatment of actual boundary conditions at or near concrete interfaces in the FEM model are therefore essential to producing results consistent with test data.
- The split-tension tests calculations using the KCCM are highly sensitive to the hourglass coefficient selected to mitigate spurious hourglass deformations that developed in the FEM calculation. When the hourglass energy ratio was too high (over 10%), the predicted ratio of f_r/f_t was too low. When the hourglass energy ratio is held below 10%, values between 1.07 and 1.09 are predicted, which are close to the range of 1.08 to 1.16 found in experiments. The calculations show that stress distributions within the sample body are complex and change after initial crack formation. It also appears that failure of the splitting tension cylinder is induced by compressive rather than tensile pressures (i.e., tensile mean normal stresses). Consequently, the splitting tension test does not explicitly measure the direct tensile strength of concrete, but instead, induces a biaxial stress field, where very few tests can be performed for concrete to define its failure surface. These results confirm that the KCCM's failure-surface formulation is adequate in this region of p - $\Delta\sigma$ space.
- The SHPB calculations show that inertia effects can be captured by the FEM calculation, if adequate mesh resolution is provided through the mesh discretization, and that DIF curves are still needed when used with the KCCM to account for the rate-enhancements observed in SHPB experiments. In compression, calculations using the first branch of the modified CEB-FIP DIF curve produced results consistent with test data, while those ignoring rate enhancements produced strengths much too low, and those including both branches of the DIF curve appeared to double-count for the inertia effects. FEM meshes having too coarse a mesh discretization may still need both branches of the DIF curves to properly account for rate effects observed in compression. In tension, the FEM calculations showed that both legs of the modified CEB-FIP model are needed to account for rate effects observed in the SHPB tests.
- The calculations for the RC panel blast tests showed that any of four concrete constitutive models, when using their simple input feature, can be used to match the deflection-history data measured during the test. The major parameter that appeared to affect the predictions, was the coefficient of friction used to define the contact between the RC panel and the steel supports. The KCCM and the CSCM models required friction coefficients above 0.25 to provide a good match to the data, while the WCM and the RHTM needed values close to zero. Reported values for steel on concrete for quasi-static tests are close to 0.45. These results are not surprising; the RC panel is lightly reinforced, which provides little if any confinement to the concrete, and each material model's response in tension will have a strong influence the panel's response in flexure. The KCCM and CSCM compute tensile strength and fracture energies consistent with those recommended by CEB-FIP, while the WSM and RHTM exhibit fracture energies that are much greater.
- For the RC column lateral loading tests, where confinement is provided to the concrete by CFRP wraps, the KCCM provides a reasonably good match to the test data when (a) adequate mesh resolution is provided to resolve the confining pressures and (b) two damage accumulation parameters controlling the rheology and dilatancy of the KCCM at high pressures (b_f and ω) are properly calibrated. The RHTM and CSCM models fail to predict the FRP rupture for any of the tests, which is likely traceable to the lack of ability in capturing the dilatancy phenomena. Also of significance is the inability of the RHT model to effectively capture the post peak plateau clearly evident in the tests.

Although the intent of this paper is not to recommend a particular concrete material model, primarily because it should be possible to calibrate a particular model with a good understanding of the model and data to calibrate and validate it, the following guidance is provided to make best use of MAT072_REL3 for problems involving concrete structures under transient loads based on our knowledge and experience with this model. First, the simple input feature described in [8] should only be used as a tool when little is known other than the unconfined compressive strength; otherwise, the model's parameters can be calibrated to the mechanical test described in [27]. This tool is offered in codes like LS-DYNA as a convenience and it should only be used with an appreciation of the assumptions and limitations described in this paper. Second, boundary conditions and other numerical artifices of the FEM calculation should be carefully considered, verified, and validated. These include parameters involving mesh resolution/biases, contact interfaces, friction coefficients, hourglass coefficients, etc. There are also many other parameters not described in this paper that should be considered depending on the problem (e.g., Bulk Viscosity coefficients, regularization methods, etc.). Finally, some of the more esoteric parameters of the KCC model can then be adjusted or calibrated to test data. Parameters include the KCCM's damage accumulation parameters b_1 , b_2 , b_3 , ω , and the DIF curves. When using MAT072_REL3's simple input feature, the following values are recommended:

Parameter b_1

This parameter that controls the damage evolution under positive (compressive) pressures. Because this parameter controls strain-softening, results can depend on the resolution of the mesh. Without resorting to more rigorous mesh regularization approaches, a simple way to help mitigate mesh-dependency can be obtained by computing b_1 as [5]:

$$b_1 = 0.34h + 0.79 \quad (25)$$

where h is the characteristic length of the element in the units of inch. However, to apply this equation, the localization width w_{lc} (typically taken as $3xMAS$) should be set smaller than the smallest h in the FEM mesh.

Parameter b_2

The parameter b_2 controls the damage evolution under negative (tensile) pressures and can also produce mesh-dependent results. In a similar way, b_2 can be calculated using the table presented in [8] or using the following relation:

$$b_2 = (0.09 \cdot w_{lc}^2 - 0.98 \cdot w_{lc} + 3.06) (1 - 0.004 \cdot f_c'^2 + 0.097 \cdot f_c' - 0.484) \quad (26)$$

where w_{lc} is the localization width in the units inch, f_c' is the unconfined compressive strength in the units of "ksi" (or 1,000 psi). This equation provides a good estimate of the fracture energy dissipation recommended by CEB-FIP.

Parameter b_3

The default value of b_3 is 1.15. For problems similar to those explored in this paper, b_3 is the least important, as this controls damage in the very rare hydrostatic extension load path. There is very little data from which to calibrate b_3 .

Parameter ω

Omega governs volume expansion during plasticity. The default value is 0.50. Suggested values for structure having low confinement range from 0.50 to 0.75 and 0.90 for problems involving heavily confined structures [5].

DIF Curve

The compression DIF can be calculated by [11]:

$$DIF_c = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1.026\alpha} & \dot{\epsilon} \leq 10^6 / \text{sec} \\ \gamma \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1/3} & \dot{\epsilon} > 10^6 / \text{sec} \end{cases} \quad (27)$$

where $\gamma = 10^{6.156\alpha-2}$, $\alpha = \frac{1}{5 + 9f'_c / f'_{co}}$, $\dot{\epsilon}_s = 3 \times 10^{-5} \text{ s}^{-1}$ is a reference strain rate, and $f'_{co} = 1,450 \text{ psi}$. If the resolution of

the mesh can resolve inertial stresses, the second branch (i.e., $\dot{\epsilon} > 10^6 / \text{sec}$ in Equation (27)) should be omitted.

The tension DIF can be calculated by [15] [9]:

$$DIF_t = \begin{cases} \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{\delta} & \dot{\epsilon} \leq 1.0 / \text{sec} \\ \beta \left(\frac{\dot{\epsilon}}{\dot{\epsilon}_s} \right)^{1/3} & \dot{\epsilon} > 1.0 / \text{sec} \end{cases} \quad (28)$$

where $\beta = 10^{6\delta-2}$, $\delta = \frac{1}{1 + 8f'_c / f'_{co}}$, and $\dot{\epsilon}_s = 1 \times 10^{-6} \text{ s}^{-1}$ is the reference strain rate. Both branches are recommended for use with the KCCM.

5. CONCLUSIONS

FEM calculations for each of five concrete structure tests demonstrate that in addition to some of the more nuanced material model parameters, parameters involving boundary conditions, hourglass energy suppression, interface friction, and loading-rate effects, all have a strong effect on the response predictions. The results demonstrate that any of four concrete constitutive models considered in this paper can be used to match any one set of test data, even though they differ in their assumptions and the behaviors modeled through their formulation; however, it is difficult to match the larger set of data without carefully considering each of these parameters. Guidance is provided to produce meaningful computational results for concrete structures under transient loads and large deformations using FEM and the constitutive model developed by the authors. Using these recommended approaches and this constitutive model, good response predictions, when considering the assumptions and limitations of the model, can be obtained for the types of concrete structure tests described in this paper.

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Numerical Computation of Reinforced Concrete Slabs Subjected to Blast Loading

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Synopsis:

A batch of blast resistance reinforced concrete slabs were tested in the shock tube facility at the University of Missouri Kansas City (UMKC). Based on the results from the tests, a blind numerical simulation contest was announced by UMKC in collaboration with the American Concrete Institute (ACI). The authors of this paper participated in the contest and received the test results only after completing their simulations. In this paper two basic numerical approaches are described. The first is a preliminary section rigidity assessment and the second is a full numerical simulation performed with the LS-DYNA code. The numerical results are compared with the UMKC test results and the influence of the numerical parameters is further discussed. The section rigidity assessment approach is then used to explain some unexpected results.

Keywords: Blast load, Reinforced concrete, Concrete dynamic response, LS-DYNA

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INTRODUCTION

The ability to correctly assess the response of reinforced concrete structures to blast pressure loads is the most basic ingredient of the protective structure profession. This ability to correctly compute the response of the structure will enable designers to build better protective structures, which on one hand do not underestimate the effect of the loading and on the other hand, do not use too much material and reinforcements. Furthermore, the ability to correctly predict the response and damage level of a reinforced concrete structure will enable a rigorous investigation of any existing structure and will help to assess the need to strengthen the structure in order to mitigate new threats.

High rate dynamic response of reinforced concrete structures presents a major engineering challenge, especially when the structure materials are taken far into the plastic range and when cracking or failure is involved. This high rate response raises the need to model correctly the strain rate influence on the stress-strain relation which is highly non-linear and depends also on the stress state (triaxiality). Furthermore, modeling the progressive damage of the concrete until crack initiation is both a physical and mathematical challenge.

There exist a few commercial numerical codes, which enable computations of the dynamic response of reinforced concrete structures to blast loads. In order to evaluate the predictions of the currently existing state of the art codes and to improve them, it is necessary to conduct experimental tests that provide detailed data on the dynamic response and failure development of a blast loaded reinforced concrete structure. Such tests are limited by the required setup complexity. It is required to produce a high pressure blast wave with enough impulse that the reaction of the target specimen will be large enough in order to elaborate the modeling difficulties that need to be investigated. The specimen dimensions should have a minimal thickness corresponding to a realistic dimension of ceiling, walls or other structural elements. In such experiments scaling is not recommended because it can influence the mechanical behavior of the reinforced concrete. Conducting tests with real blasts is expensive. Moreover, actual blasts involve expansion of the explosive products and dust which interfere with high rate photography essential for following the damage development. The tests that are used for this study were done in the Blast Loading Simulator (shock tube) facility at Missouri University as part of a blast blind simulation contest 2012 [1]. These tests provide the important data needed to improve constitutive laws of concrete and to obtain practical recommendations for users of simulation codes. The conductors of the experiments provided the data required for the simulations includes full description of the reinforced concrete slab geometry, the material data, the boundary conditions and the pressure loading histories. Two types of reinforced concrete were tested: (a) normal strength concrete with normal steel reinforcements, referred to as NCNR; (b) high strength concrete with improved steel reinforcements (enriched with vanadium), referred to as HSCV. Each type of panel was subjected to two levels of pressure loads. Middle span deflections were recorded dynamically by means of a laser detector.

In this study we present the numerical results that we obtained using the commercial code LS-DYNA® [2] as compared to the results of the experiments. The accuracy of the models and the effects of some parameters of the used material models are further discussed and practical recommendations for code users are outlined. Three concrete material models available in LS-DYNA were tested in this study: MAT72_R3, MAT72 with the BGU calibration [3] and MAT159 (CSCM) [4]. Only the latter (CSCM) was chosen for a detailed investigation since it performed best and is very easy to apply. This model was calibrated by its developers for different compressive strengths up to 58 MPa (8.4 ksi) [5].

Test setup and slabs

A general description of the experimental setup based on the provided data is given in Figure 1. The concrete slabs dimensions are 1.63 m (64") x 0.86 m (33.75") x 0.1 m (4") with one layer of #3 reinforcements located 12.7 mm (0.5") from the bottom of the slab (the opposite face of the loading side), see Figure 2.

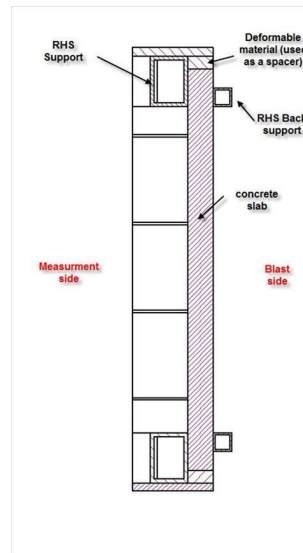


Figure 1- General description of the slab position on the steel frame at the end of the shock tube

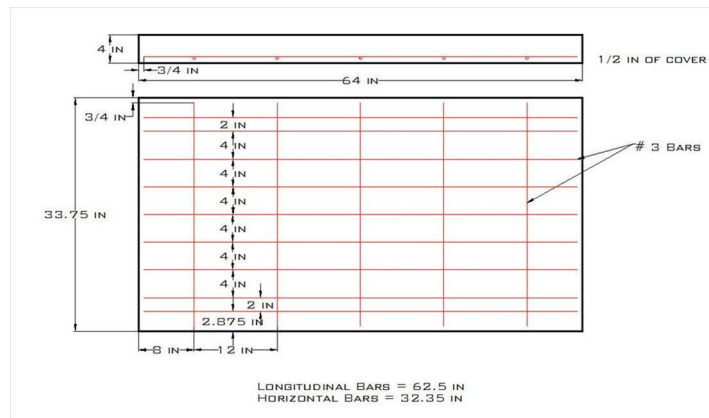


Figure 2- Reinforcements arrangement

According to the rules of the contest, the authors (who were not involved in the tests) received data on the geometry and materials involved (figures 1-3). Although not implicitly declared, it is reasonable to assume that the tolerances of the placement of the reinforcement are in the order of a 0.5" (see figure 4 in which a person is standing on the reinforcement prior to concrete casting). A view of the slab from the outer side of the shock tube can be seen in Figure 5. The location of the deflection measuring device can be seen on the left part of Figure 5.

Two combinations of concrete and reinforcements were tested: (a) normal strength concrete of unconfined compressive strength 38 MPa with normal steel reinforcements (its stress-strain curve is given in Figure 3; and (b) high strength concrete of unconfined compressive strength 80 MPa (11.6 ksi) with improved steel reinforcements (enriched with vanadium) (Figure 3). Each type of slab was loaded with two different loads,

namely P_a (high pressure loading) and P_b (reduced pressure loading). These load histories are shown in Figure 6. $P1_a$ and $P2_a$ are two repetitions of the pressure load P_a to which both types of slabs (NCNR and HSCV) were subjected separately. These loading repetitions were very similar to each other with impulse difference of about 4%. $P1_b$ and $P2_b$ are two repetitions of the pressure load P_b and their impulse difference is about 2%.

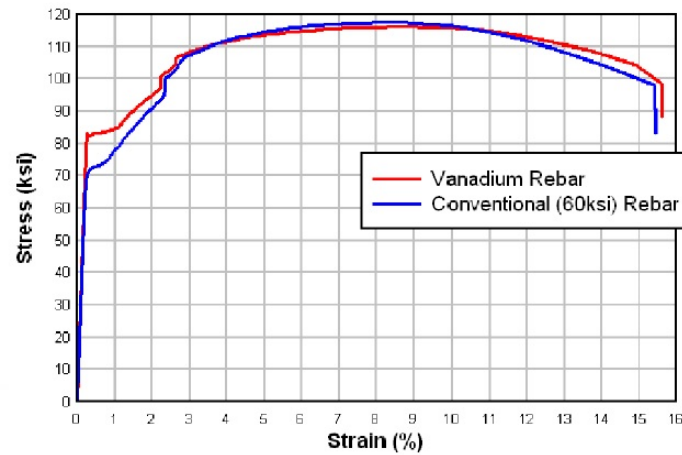


Figure 3- Stress-strain curves of the steel reinforcement



Figure 4- Preparation of the slabs. The tolerance of the reinforcement vertical location seems to be the wider among other dimensional tolerances

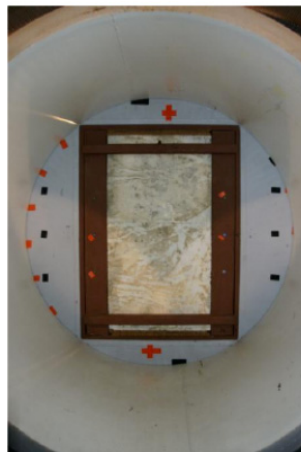


Figure 5- A look from outside (left) and inside (right) of the shock tube

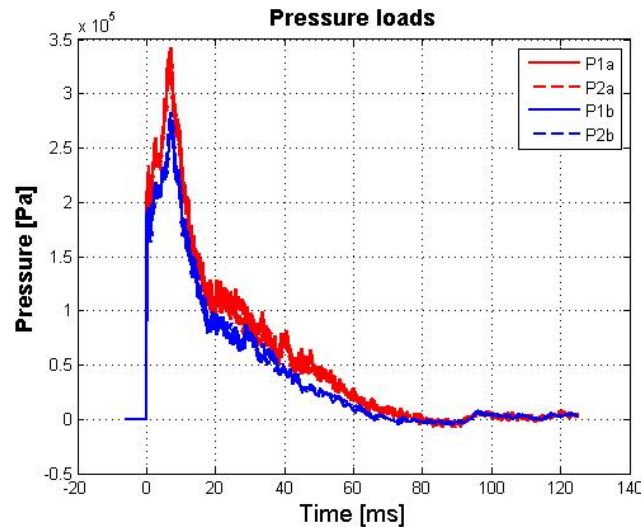


Figure 6- Pressure histories

Preliminary assessment of the section rigidity

Prior to numerical modeling, it is a good practice to evaluate the moment-curvature relation of the slab. This basic property enables us to compare between different cross sections and to estimate their relative response to the load. Such evaluations can be done with the well-known software “Response-2000” [6], which uses the Vecchio-Collins [7] compressive softening model and the Bentz [8] tension stiffening of cracked concrete. This code calculates the non-linear cross section moment capacity taking into account the non-linear stress-strain relations of the steel reinforcements and the concrete including cracking effects.

Using this code for both the NCNR and HSCV cross sections, it was predicted that up to a curvature of 170 rad/km, the moment difference between the two kinds of slabs is about 0-15% (Figure 7). The tests resulted with a 0÷15% difference in residual deflections (Figure 11 and Figure 12). This is consistent with our preliminary assessment.

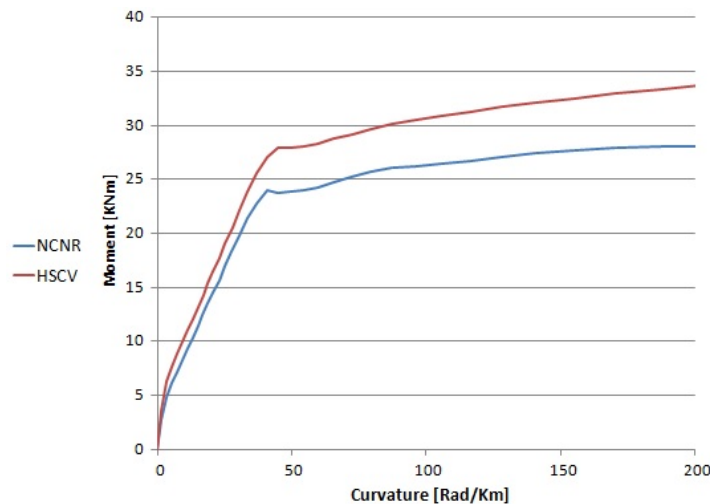


Figure 7- Response 2000 moment-curvature calculation for NCNR and HSCV sections.

THE NUMERICAL MODEL

The responses of the reinforced concrete slabs were modeled using the numerical code LS-DYNA® [2]. A general view of the model can be seen in Figure 8. The structure was modeled using two symmetry planes where the left side of Figure 8 represents the middle of the slab span. The section area of the reinforcements lying on the symmetry planes is divided by 2. The RHS tubes supporting the slab were modeled using shell elements and a bi-linear plasticity model. The assumption of fixed boundary conditions of the RHS tubes at their edges (see Figure 9) represents the rigidity of the shock tube frame, which supports the edge of the slabs. The interaction of the slab and the RHS tubes is modeled using contact condition accounting for their resistance to the slab rotation at the support. The pressure history is applied by defining nodal forces at the top of the slab, which remain vertical regardless of the deflection. The concrete is modeled using cubic solid elements with three different element sizes: 25.4 mm (1"), 12.7 mm (0.5") and 6.35 mm (0.25") (per the slab's thickness there were 4, 8 and 16 elements respectively -herein designated as R4, R8 and R16). The aggregate size was set to 3 values: 19 mm (0.748"), and ratios of 0.5 and 0.75 of the element size (herein referred to as D19 mm, D0.75Es and D0.5Es, respectively). The reinforcements are modeled with 1D beam elements, which are constrained to the concrete with a contact algorithm (*Constraind_Lagrange_in_solid keyword in LSDYNA). Using this method, there is no need to link the beam nodes to the solid nodes, resulting in a more convenient and flexible modeling method and allowing the use of larger elements. The reinforcements material was modeled with the piecewise linear plasticity material model MAT24. Strain rate sensitivity is modeled using the Cowper-Symonds model that is given by Eq. 1. The Cowper-Symonds parameters suggested by Alves [9] for steel reinforcements in a reinforced concrete are: $c=0.55$ m/s, $q=3.439$.

$$DIF = 1 + \left(\frac{\dot{\epsilon}}{c} \right)^{\frac{1}{q}} \quad \text{Eq. 1}$$

Concrete material model MAT159 (CSCM – Continuous Surface Cap Model) was chosen for modeling after trying two additional models: (a) MAT72 with the BGU calibration [1] (b) MAT72R3, which enable using the unconfined compressive strength as the single input. The choice was made based on a preliminary comparison of the performance and stability of the three material models. An example of the results obtained with the BGU calibrated model and CSCM model is shown in Figure 10. The BGU calibrated damage model was significantly stiffer than the CSCM model when rate dependency was used but when the rate dependency was removed its performance was too flexible. Material model *Mat_Concrete_Damage_Rel3 with the automatic option for material parameters was tested and resulted in significant instability. From this preliminary study the conclusion is that for this particular case, the CSCM model is more robust.

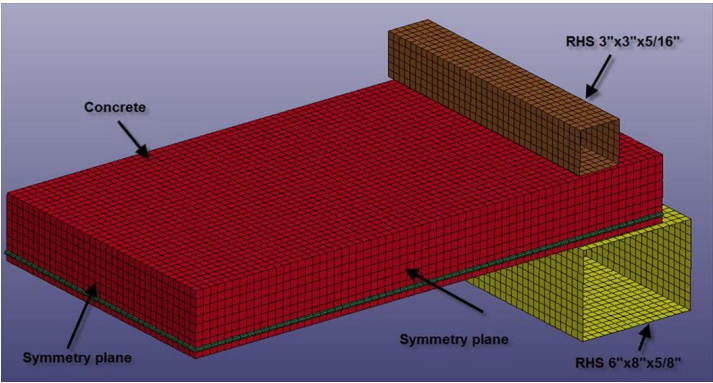


Figure 8- Finite element model of the reinforced concrete slab

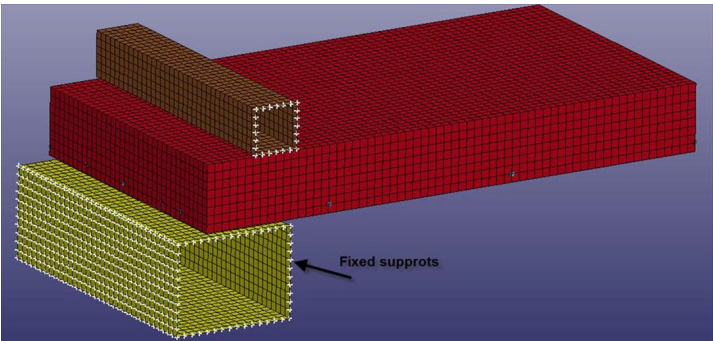


Figure 9- Boundary conditions on the RHS profiles

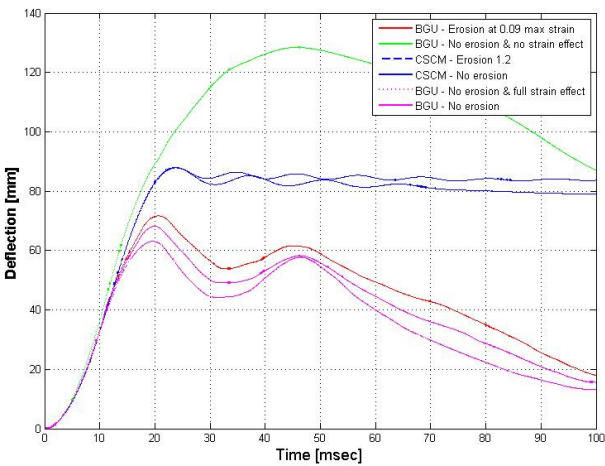


Figure 10- Preliminary performance comparison of the different available material models for concrete

Table 1- MAT159 input parameters

Parameter	Description	Range for sensitivity test
RO	Density	2350 [kg/m ³] (146.7 pcf)
IRATE	Strain rate sensitivity flag	1,0 [including and excluding strain rate effect]
ERODE	Erosion of strained elements	0.9, 1.1, 1.2 [no erosion occurs, erosion occurs at 10% plastic strain, erosion occurs at 20% plastic strain]
FPC	Unconfined compressive strength	38 MPa (5.5 ksi), 58 MPa (8.4 ksi) (normal concrete and high strength concrete).
DAGG	Aggregate size	19 mm (0.748"), 0.75 of element size, 0.5 of element size
RECOV	Recovery	0 [no recovery]
UNITS	The unit set type	1 [kg, mm, N, MPa, ms]

The concrete is defined using the CSCM model [4] with the parameters shown in table 1. The sensitivity of the slab response to each of the parameter was studied within a specified range of values as described in the table.

The CSCM model was calibrated for concretes of unconfined strengths in the range of 28÷58 MPa (4.1÷8.4 ksi) [5]. To represent the high strength concrete used in the experiments it would be natural to use a value of 80 MPa (11.6 ksi) for the FPC parameter (unconfined compressive strength) as given by the conductors of the tests [1]. When using this value the computed HSCV slab was too rigid, probably because the model was not calibrated for strengths higher than 58 MPa (8.4 ksi) [4, 5]. When a value of 58 MPa (8.4 ksi) for FPC was used, the results were good as shown in the next section.

SHOCK TUBE TESTS RESULTS

Figure 11 and Figure 12 summarize the experimental results (deflections history at the center of all the tested slabs) as provided by UMKC. Several important observations can be made. The results of the NCNR reinforced concrete loaded with the higher pressure histories P_a are very close to each other (blue curves in Figure 11). Their initial dynamic rigidities (deflection vs. time), their maximal and final deflections are similar. This fact shows that the test setup was very repeatable in the cases of standard concrete and steel combination. On the other hand, the residual deflections of the HSCV reinforced concrete slabs differ significantly from each other (red curves in Figure 11). More surprising is the fact that one of the HSCV result is almost identical to the NCNR results. For the case of reduced pressure loading, P_b , we see that the residual deflection is relatively equal for the HSCV and NCNR reinforced concretes (Figure 12). The slope of the deflections up to 20 ms is the same, which could imply that both kinds of slabs have the same dynamic rigidity up to yielding. The slabs have different maximal deflections but the residual deflections are similar.

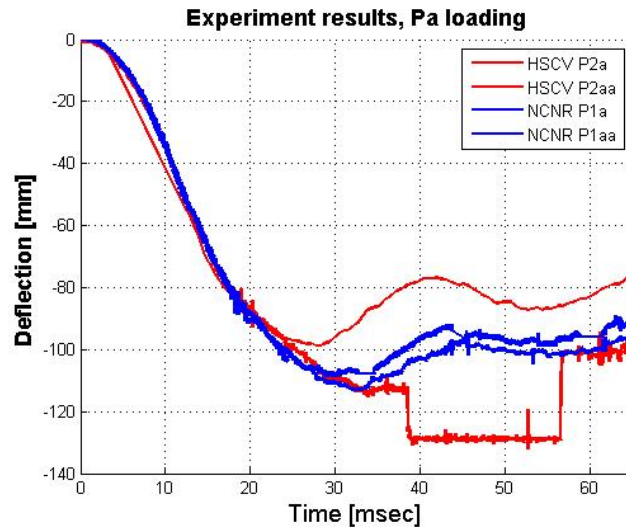


Figure 11- Experimental results of loading with P_a (high pressure load)

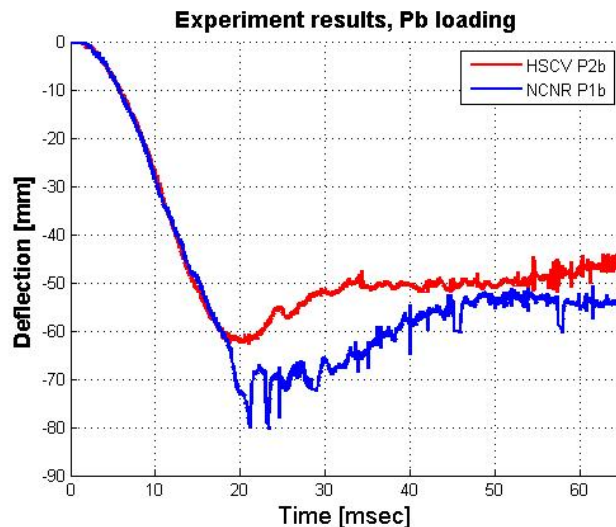


Figure 12- Experimental results of loading with P_b (reduced pressure load)

FINITE ELEMENT MODEL RESULTS

The comparison of the experimental and numerical responses of the NCNR slab to the high pressure loading, P_a , is shown in Figure 13. In these computations the strain rate effects were included only for the concrete. It is clear that reducing the element size yields a stiffer slab, mainly after crack initiation (after about 10 ms the curves divert from each other). This phenomenon was observed by other investigators [10] and is related to the cracking process and the ability of the model to represent correctly the dissipation of fracture energy. It can be seen that the use of smaller values for aggregate size parameter (DAGG) results in a more flexible behavior, which results from reducing the tensile fracture energies [4]. One can see that the maximal response and the final deflection is closely estimated with the R4 and R8 resolutions with a small variance depending on the aggregate

size. Up to a deflection of 90 mm, the response of the numerical model and the experiment are very similar except for the R16 resolution. Inspecting the vibration frequency after reaching the maximal deflection one can see that the numerical results demonstrate a higher frequency (approximately 4 times higher). One can conclude from this that the elastic part of the cracked reinforced concrete is above 4 times more rigid than the experimental slabs.

The response of NCNR slab to the reduced pressure loading, P_b , is shown in Figure 14. In this case the maximal response is underestimated (which is an unwanted situation for protective purposes). The residual deflection is relatively well assessed by using the R4 and R8 resolutions with an approximately 10% error.

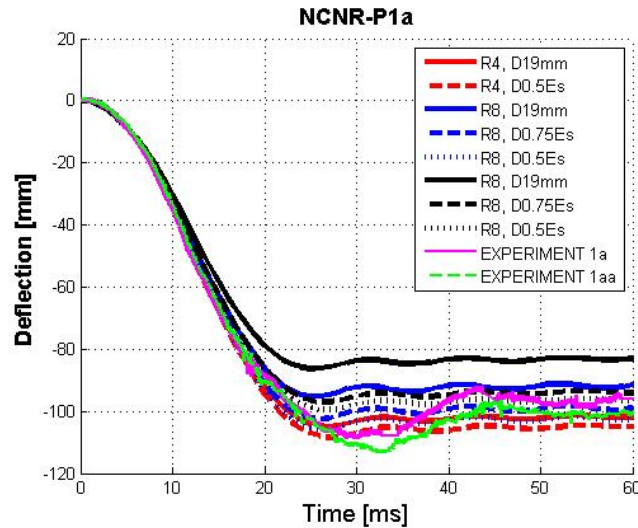


Figure 13- Experimental and numerical responses of NCNR slab loaded with P_a pressures.

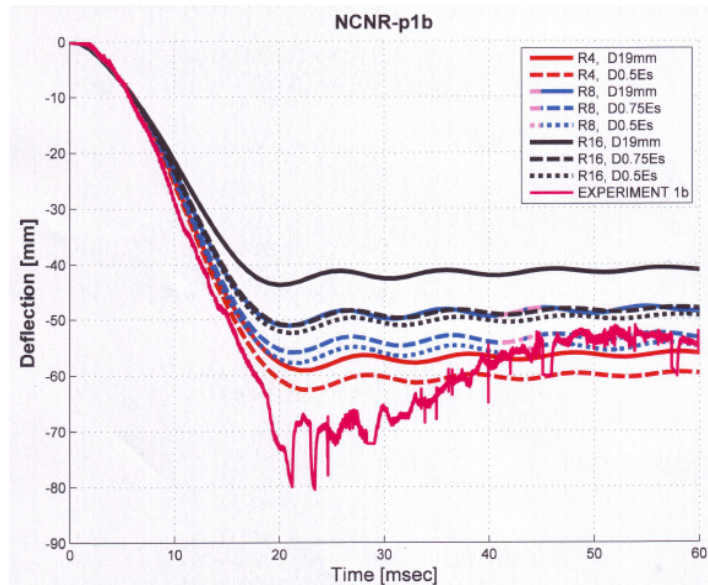


Figure 14- Experimental and numerical responses of NCNR slab loaded with P_b pressures.

Figure 16 describes the sensitivity of the model to strain rate effects and to other CSCM parameters. For this study the results were obtained with R8 resolution (12.7 mm (0.5") element size) and with an aggregate size

of 6.35 mm (half of the element size). The concrete strain rate effect increases the rigidity by 10% and is related to the tension strength, which is the dominant failure parameter in the current bending situation. The 10% increase in tension strength is in agreement with available experimental observations [11, 13] for relatively low strain rates. Rate enhancement effects may arise from the dynamic behavior of the structure rather than from the constitutive material laws, which should be treated with caution as was demonstrated by Schwer [12]. The possible tolerance of the exact position of the reinforcement, 0.5", has a similar influence of 10% on the increase of the rigidity. The suggested Cowper-Symonds steel parameters by Alves [9] ($c=550/s$, $q=3.439$) for the strain rate influence on the steel (16% dynamic increase factor for the current case) seem to correctly estimate the dynamic response (nominal case in Figure 16) with a relatively small increase of the rigidity. It is also demonstrated that the use of a too high dynamic increase factor (resulting from $c=0.55/s$ and equal to 118% increase) results with a too rigid response. Results with a closer residual deflection to tests results were obtained using the built-in strain effect of the CSCM model for the concrete and the Cowper-Symonds parameters ($c=550/s$, $q=3.439$) for the steel.

The numerical prediction obtained for the HSCV slabs were in considerable disagreement with the experimental. The strength of the actual concrete is relatively high and the CSCM material model was not calibrated for this level of unconfined strength [5]. But, due to the discrepancy between test results obtained with two similar HSCV slabs (figure 11), there was no point in trying to find a better calibration.

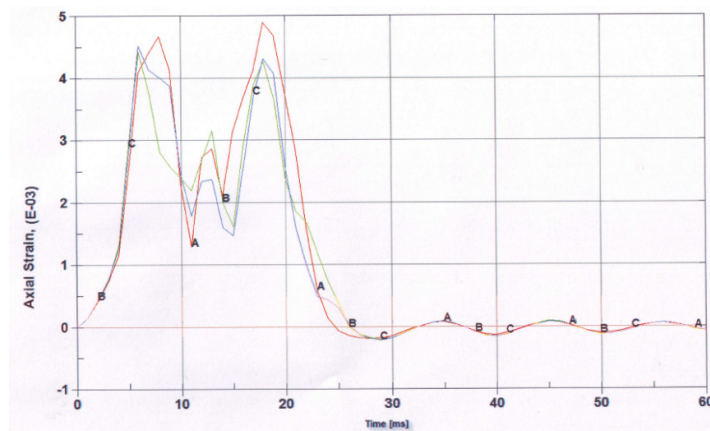


Figure 15- Strain rates at the most stressed positions of the reinforcement (NCNR, P_a loads)

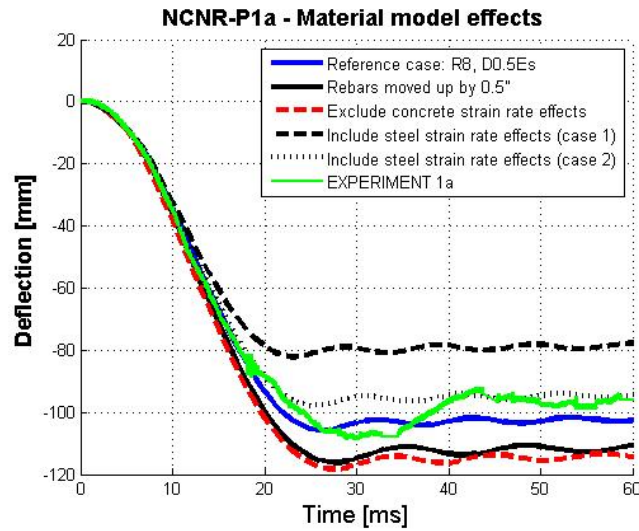


Figure 16- Model sensitivity check for strain rate effects (steel and concrete) and general parameters of the CSCM model

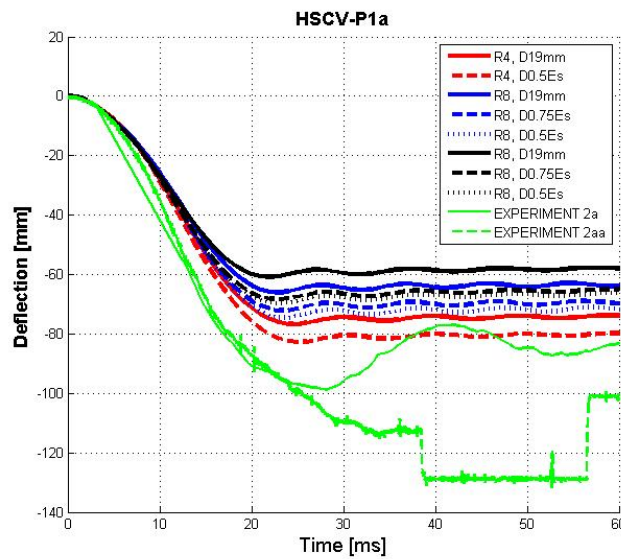


Figure 17- Experimental and numerical responses of HSCV slab loaded with P_a pressures

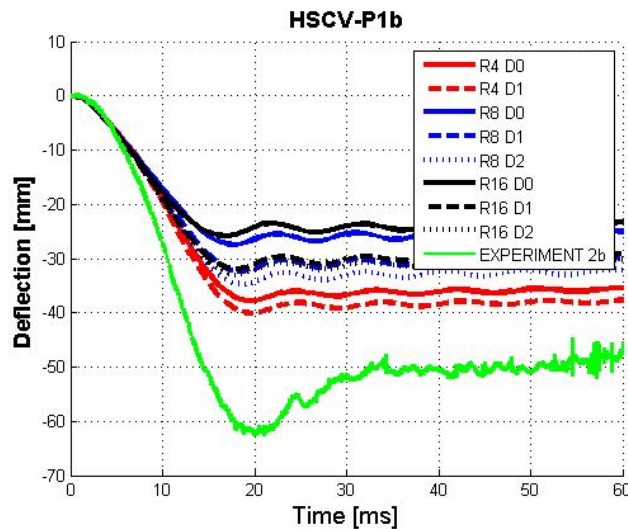


Figure 18- Experimental and numerical responses of HSCV slab loaded with P_b pressures

DISCUSSION

As demonstrated in the above section, a preliminary rigidity assessment may provide preliminary estimates of the reinforced concrete moment-curvature response by using relatively simple and available software like “Response 2000”. Assessment on the relative rigidity of different cross sections and their resistance potential can be easily obtained prior to performing time consuming numerical modeling. Slab production tolerances should also be accounted for during this preliminary step. For example, small variations in the position of the reinforcement can significantly influence rigidity, depending on the dimensions of the slab. Other considerations are small variations in the concrete mixture that can result in large differences in compressive strength, especially when high strength concretes are considered.

In problems involving bending of reinforced concrete slabs the level of confinement pressures during the bending process is relatively small (a fraction of the bending stress). Since the concrete material is brittle and damage occurs mainly in tension, the state of stress will not change considerably during the damage process. Due to this uniaxial stress behavior, the standard unconfined compressive strength can be used as the only material data required for simplified estimations. When a detailed numerical model is used with a more complex material model, such as the one used in the present study, the exact behavior of the material under high confinement pressures is never used, and is of almost no importance for a purely bended slab.

Many parameters affect the numerical simulation quality. We found out that for the current case the choice of CSCM model gave the most stable and qualitative response compared to the experimental results. Mesh resolution highly affects the cracking energy and as a consequence the overall rigidity. When using a concrete material model, a finer mesh does not necessarily lead to better result. We recommend meshing with 4 and 8 divisions of the slab thickness and setting the aggregate size parameter to half of the element size for getting reasonable estimates. Strain rate effects should be included, especially for the concrete. Though, it is a good practice to investigate the influence of strain rate effects on the results and to take precaution with actual designs.

From the steel stress–strain curves (Figure 3) it can be seen that the strengths of the two kinds of steel differ only in the 0.2-2.5% strain range, and are very similar beyond strains of 2.5%. Since the maximum strain level reached in the NCNR (Figure 19) slab subjected to the reduced pressure load is 2.5% (Figure 20), one can expect a difference in the slabs maximal deflection response due to the difference of the reinforcement rigidities. For other cases when the axial strains become much larger than 2.5% the difference in the response will become smaller.

It was observed during the simulations that the hourglass dissipation energy becomes approximately 10% of the total energy. Since this energy dissipation is rather numerical and not physical, its influence on the results should further be investigated.

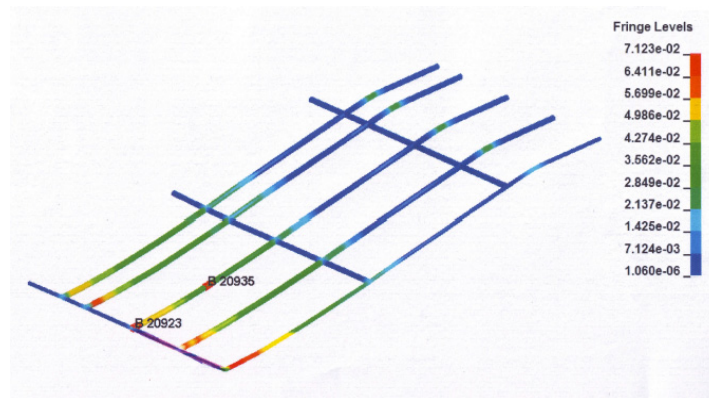


Figure 19- Steel reinforcement strain at the maximal deflected stage of NCNR P1_a

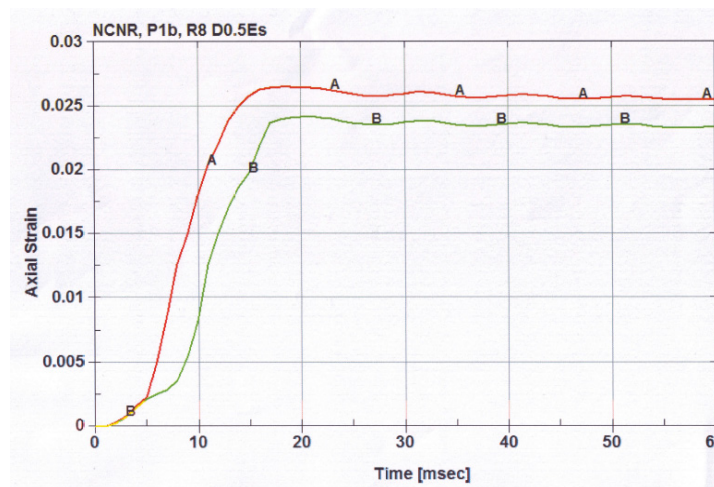


Figure 20- Steel reinforcement axial strain vs time, NCNR P1_b case

SUMMARY

Detailed simulations of the response of normal and high strength concrete slabs were demonstrated in this work. It is shown that simulations of the bending failure mode of normal strength reinforced concrete slabs can easily be obtained using an available concrete material model, such the CSCM with automatic values

generation. The obtained results were in close agreement with the test results. The ratio of the maximum deflection to the span of the slab was approximately 1/16 which is considered as a deep failure for practical protection purposes. It means that simulations such as those performed in the course of the present study can be used for wide range of applications for protection. On the other hand, it was shown that for high strength concretes, the simulation results using the same material model CSCM underestimated the dynamic response since the material model was not calibrated for compressive strengths higher than 58 MPa.

The pre-assessment of the concrete section non-linear rigidity is a preliminary step that any designer should begin with. It was shown that such a pre-assessment can be obtained using the shareware software "Response 2000".

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NUMERICAL MODELLING FOR REINFORCED CONCRETE RESPONSE TO BLAST LOAD: UNDERSTANDING THE DEMANDS ON MATERIAL MODELS

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Synopsis: Numerical modelling is nowadays commonly employed in the analysis of concrete structures subjected to extreme dynamic loadings such as blast. Sophisticated material models, particularly concrete, are available in commercial codes and they are often applied in their default settings in a diverse range of modelling applications. However, the mechanisms governing different load response scenarios can be characteristically different and as such the actual demands on specific aspects of a material model differ. It is therefore not surprising that a well-calibrated material model may exhibit satisfactory performance in many applications but behave unfavourably in certain other cases. Modelling the response of reinforced concrete structures to blast load presents such an important scenario in which the demands on the concrete material model are considerably different from high-pressure scenarios for example high-velocity impact or penetration. This paper stems from an initial modelling undertaking in association with the Blind Blast Contest organised by the ACI Committee 370, and extends to a detailed scrutiny of the demands on the concrete material model in terms of preserving a realistic representation of the tension/shear behaviour and the implications in a reinforced concrete response environment. Targeted modifications are proposed which demonstrate satisfactory results in terms of rectifying the identified shortcomings and ensuring more robust simulation of reinforced concrete response to blast loading.

Keywords: reinforced concrete, blast load, numerical simulation, concrete material model, damage mechanism, failure mode

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INTRODUCTION

The response of reinforced concrete (RC) structures to extreme dynamic loading such as blast and impact is complicated due to a combination of several factors, of widely cited ones being stress wave and strain rate (e.g. Bischoff and Perry¹), inertia-induced confining effect (e.g. Donze et al.², Li and Meng³, Song and Lu⁴), and large deformations. Extensive research has been devoted over the past decade or so to understanding each of these phenomena, and the research work has seen increasing involvement of the use of high fidelity numerical simulation techniques in more recent years. Indeed, numerical simulation has made it possible to scrutinise in great detail the development of the stress-strain fields and the evolution of damage during and after the transient phase of the blast effect. Sophisticated material models have been developed and continuously evaluated and calibrated to cater to the numerical simulation needs in a variety of dynamic loading conditions.

At the backdrop of all the developments it is worth pointing out that the response of a concrete structure or structural component subjected to a high impulsive load would invariably experience distinctive response phases from the initial contact with the incoming load to the later global deformation response. While the majority of the so-called hydrocode models for concrete-like materials have been validated to a varying extent in applications where high pressure and localised material response plays a dominant role in impact and penetration (e.g. Polanco-Loria et al.⁵, Unosson and Nilsson⁶), near-field blast (e.g. Tu and Lu⁷, Zakrisson et al.⁸), systematic examination of the performance of such material models extending into the global deformation phase is relatively limited. It is generally understood that the dominate mechanisms in the global deformation phase can be significantly different from those in the high-intensity transient local response phase; the fact that the pressure level becomes low in this phase of response requires the material model to be able to accommodate tension, shear, as well as the relatively simple compression behaviour under low pressure but still complex multi-axial stress condition. The demand on the material model being able to exhibit appropriate behaviour in tension and shear becomes particularly important in reinforced concrete structures to ensure that transfer of stresses between concrete and reinforcing bars can take place in a realistic “reinforced concrete” composite manner.

A numerical model for reinforced concrete that is capable of simulating the entire process from direct blast loading through to the final global deformation phase is generally required in civil engineering applications where the residual state and the residual capacities of the affected members can play an important role in the design and evaluation of such effects. However, as discussed above a concrete material model which has shown sound performance in high-intensity stress applications may not necessarily perform as well in a reinforced concrete environment under a global deformation scenario.

The recent blind blast contest organised by ACI Committee 370 provided an excellent and rather unique opportunities for an in-depth examination into the modelling of reinforced concrete response to blast loading with hydrocode material models. The test RC slab was reinforced by a single layer of steel bars without any shear links, and this reinforcement scheme poses high demands on the residual behaviour of the concrete model in tension and shear in maintaining an adequate degree of integration with the rebar, thus maximises the exposure of any issues of the concrete material model in this category of numerical simulation for RC structures.

This paper is mainly concerned about the behaviour of a representative concrete material model in hydrocode platform LS-DYNA⁹, namely the Karagozian & Case Concrete Damage (KCC) model¹⁰, when it is applied in modelling blast response involving global deformation. The main objectives are to demonstrate its performance under this regime of the responses, to investigate into the cause of some abnormal behaviour, and subsequently to propose and demonstrate possible remedies. For a comparison, another widely used model in LS-DYNA, the Continuous Surface Concrete (CSC) model¹¹, is also employed and discussed.

In the present paper, securitisation of the detailed response in relation to the basic behaviour of the material model suggests that the particular issue with the KCC model seems to originate from the abrupt descending of the material towards a zero strength state following a tension/shear dominated damage process, and the consequent diminish of the interaction between the steel rebar and the surrounding concrete. In contrast, the CSC model always retains