Simplified methodologies for preliminary blast-resistant design of precast concrete wall panels

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- This paper evaluates two simplified methodologies for the preliminary design of precast concrete wall panels to resist blast loading for use by design consultants, construction contractors, and precast concrete suppliers in cost estimating during the bidding process of a project.
- The methodologies use pressure-impulse diagrams and include a normalization approach and a curve-fitting approach.
- The authors have created a spreadsheet-based tool, following the normalization approach, for use during preliminary design.

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Precast concrete wall panels are commonly used for exterior building envelopes due to their installation efficiency, high quality control, and design flexibility. For facilities that are vulnerable to explosive threats, these panels often serve as the first line of defense against blast loading and are commonly detailed to resist these severe impulsive loads, in addition to conventional loading requirements. Blast demands are considered for antiterrorism and force protection applications, such as government buildings and military installations, or for facilities at risk of accidental vapor cloud explosions, such as petrochemical or industrial processing facilities. Due to the dynamic nature of blast loading conditions, specialized design and analysis methods are needed to quantify structural response and determine the extent of component damage following a blast event. These methods can be computationally expensive and require the knowledge and expertise of a blast design specialist. For these reasons, it is not always feasible for precast concrete producers to readily assess the blast-resistant performance of a wall panel system during the early design stages and bidding processes. This paper evaluates two simplified methodologies for the preliminary design of precast concrete wall panels to resist blast loading. Using efficient and computationally inexpensive approaches, these methods allow the user to rapidly assess the blast resistance of a given panel design, thereby facilitating a more accurate estimation of fabrication and installation costs during the bidding phase. An interactive design tool based on one of these methods has been developed by the authors to facilitate the evaluation of a broad range of panel constitutive parameters and increase the ease of implementation for precast concrete producers (this tool can be found at https://www.pci.org/2019July-Appx).

Precast concrete wall panels provide an attractive design solution for blast-resistant applications due to their flexural performance, inertial mass, and customizability. When developing a bid for a facility with blast design requirements, precast concrete producers must rely on previous experience, internal expertise, or preliminary analyses from a specialized blast design consultant to develop a reliable cost estimate. If the bid is successful, the full extent of blast-resistant design calculations is then typically performed by an external consultant hired by the contractor with the successful bid. In addition to determining expected deformations and corresponding damage to the panel in accordance with specified performance levels, the expert blast consultant must calculate the resulting reaction forces, which are then used by the precast concrete producer to design appropriate connection details.

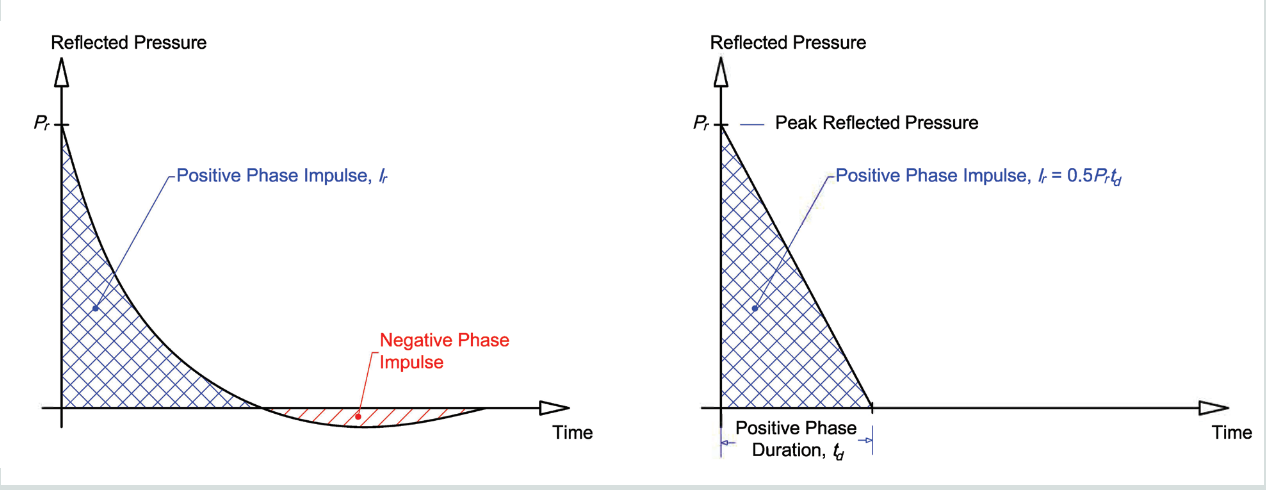
Blast-induced reaction forces often lead to large and expensive connections. Because this information usually becomes apparent once the project has already been awarded, cost estimates that were initially provided by the precast concrete producer may no longer be representative of actual construction costs. To address these issues, two simplified blast evaluation methods that allow rapid preliminary design of precast concrete wall panels are presented herein. The approaches use pressure-impulse (*P-I*) diagrams, that is, iso-damage curves that represent the potential combinations of reflected pressure and impulse demands that produce a given level of component response. Once *P-I* curves are determined at critical response levels for a panel design configuration (for example, at a low, medium, or high level of damage), the performance of the panel under an array of potential blast hazards can be rapidly assessed. This process facilitates a cost-effective estimation of expected panel response and eliminates the need for complicated dynamic analyses during the bidding phase.

Background

Single degree of freedom analysis methodology

Explosive events generate a shock or pressure wave that radiates outward from the point of detonation. Blast pressure loading initiates when the shock wave makes contact with the surface of the component. A realistic representative blast loading time history is composed of a large initial positive pressure that rapidly decays (over a timescale of milliseconds) until a small negative blast pressure region is produced as the shock wave clears (**Fig. 1**). Because it is small and can slightly counteract deformation induced by the larger positive pressure, the negative phase is often neglected. As a further simplification, the positive phase is often idealized as a triangular pulse function (Fig. 1). This representation of blast loading is widely used in design and therefore was used for the study presented in this paper. The magnitude of the peak reflected pressure and duration of the positive phase are a function of the charge size and standoff distance in accordance with empirical relationships documented in *Unified Facilities Criteria* (UFC) 3-340-02.[1] The impulse of the blast loading was calculated as the area under the positive pressure time history.

To properly determine the response of structural components to blast events, dynamic analysis methods were used that consider the characteristics of the blast-induced shock wave as well as the flexural behavior of the structural component. Flexural performance was assessed using idealized resistance functions, as addressed in the UFC,[1] with a generalized single degree of freedom (SDOF) analysis approach, as outlined in



Realistic representation of both positive and negative pressure phases

Idealized, conservative triangular pulse function with positive phase only

Figure 1. Blast pressure time histories.

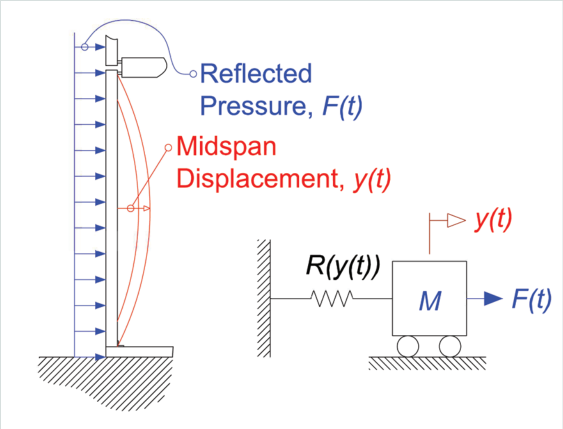


Figure 2. Building component subject to blast (left) and idealized single degree of freedom system (right). Note: M = lumped mass of the system; $R(y(t))$ = resistance function of the component.

Biggs.[2] Each component was equilibrated to a mass-spring system and allowed only one translational degree of freedom normal to the span length (**Fig. 2**). This approach relies on the assumption of far-field explosive conditions, which implies that the component of interest is far enough from the epicenter of the explosion to justify the approximation of uniform pressure demands along the entire span length. The response of elements to near-field explosions, typically governed by brittle mechanisms such as spall and breach, were not included in the scope of this study because the majority of blast-resistant design for precast concrete facade panels is performed for far-field hazards.

Wall panels on a building can be modeled with idealized boundary conditions, such as fixed-fixed, fixed-simple, or simple-simple, depending on the connection detailing of the system and the goals of the analysis. For example, elements being evaluated for maximum deflection under blast loading are commonly analyzed with simple-simple boundary conditions, and evaluations for maximum shear may be performed using fixed supports at one or both ends. Once the component is idealized as a generalized SDOF system, its deformation history can be calculated by solving the dynamic equation of motion in Eq. (1).

$$K_{LM} M y''\left(t\right) + R\left(y\left(t\right)\right) = F\left(t\right) \tag{1}$$

where

K_{LM} = load-mass transformation factor

M = lumped mass of the system

$R(y(t))$ = resistance function of the component

$y(t)$ = midspan displacement of the panel as a function of time t

$F(t)$ = blast pressure versus time history

The load-mass transformation factor K_{LM} is used to equate the distribution of mass and applied blast pressure along the span of the component as an SDOF system. This factor is calculated using Eq. (2) as the ratio of the load and mass factors K_L and K_M, respectively. For uniform mass and pressure, these factors are calculated using an appropriate shape function for the actual element $\phi(x)$ and the span length L of the member (Eq. [3] and [4]).

$$K_{LM} = \frac{K_M}{K_L} \tag{2}$$

$$K_L = \frac{1}{L}\int_0^L \phi\left(x\right) dx \tag{3}$$

$$K_M = \frac{1}{L}\int_0^L \left[\phi\left(x\right)\right]^2 dx \tag{4}$$

An example for values of K_{LM} that are commonly used in SDOF analyses are 0.78 (elastic) and 0.66 (plastic) for a simply supported one-way component with uniform pressure and assuming that a blast-loaded element will experience the same deflected shape as for static loading.[2] The mechanical behavior of the component is represented using the resistance function R, which describes the relationship between the magnitude of applied load and the resulting midspan deformation. The resistance function R can be calculated using traditional structural analysis approaches that incorporate the constitutive properties of the materials, compatibility, and force equilibrium. In this approach, it is commonly assumed that a discrete plastic hinge will form at locations where the cross section yields. An idealized elastic–perfectly plastic resistance function is often used by assuming a linear elastic response up to the point of component yield, after which the resistance remains constant for all subsequent values of plastic deformation. For reinforced concrete components, the elastic stiffness is calculated assuming a moment of inertia equal to the average of values for gross cross section and fully cracked behavior. This assumption does not consider the effects of strain hardening in the reinforcing steel or the softening that occurs once the concrete crushing strength is reached. After determining the mass of the component, resistance function, transformation factors, and blast loading demands, midspan deformation of the SDOF model is generated as a function of time using any numerical method suited for dynamic structural analyses. The maximum displacement is then converted to either a ductility ratio or equivalent support rotation for comparison with appropriate blast response criteria to determine the extent of damage to the component. Response criteria have been developed for several intentional and accidental blast hazards, including antiterrorism, petrochemical, and nuclear design applications.

P-I capacity curves

It is possible for a structural element to experience the same maximum response (and therefore the same level of protection) when subjected to various combinations of reflected pressure and impulse. A series of these combinations can be identified using SDOF analyses and assembled to produce a *P-I* capacity curve for a component at a given level of protection. *P-I* diagrams are commonly used in blast-resistant applications, such as determining safe standoff distances and establishing acceptable thresholds for human injuries. An early application of obtaining safety distances in this manner was initiated following World War II, where buildings damaged by blast loading in the United Kingdom were analyzed for different response limits.[3] Currently, *P-I* diagrams are commonly used as a design tool for a wide array of building facade components, including proprietary window systems, curtain walls, and concrete wall systems.

Generating a *P-I* curve requires multiple iterations of SDOF analyses to identify the relevant response limit, which can be tedious, especially in a preliminary design phase. For this reason, several recent research efforts have introduced multiple ways of developing normalized *P-I* diagrams for blast-loaded components. Li and Meng[4] developed a normalized *P-I* curve for an elastic SDOF system that is compatible with varying pulse loading shapes. Fallah and Louca[5] approximated the *P-I* curves of idealized structural components by deriving analytical formulas. The formulas depend on an SDOF system with a bilinear resistance deflection curve subjected to different pulse loading shapes. Using this method, the curve can be generated by using one known point in the dynamic range of the plot. Shi et al.[6] derived analytical formulas as a function of constitutive properties to develop a normalized *P-I* curve for reinforced concrete columns. Dragos and Wu[7] proposed an analytical methodology to develop a normalized curve for any pulse loading shape and any bilinear resistance function based on an empirical approach. Dragos et al.[8] derived two equations that can be used to normalize a *P-I* curve for simply supported, one-way, and ultra-high-performance concrete slabs. Wang et al.[9] developed an analytical formula to generate *P-I* curves for a one-way reinforced concrete slab using pressure and impulse asymptotes. The aforementioned methodologies have certain limitations: Li and Meng can only be used for elastic SDOF systems, Shi et al. and Dragos et al. work for a relatively small set of components, Fallah and Louca requires an SDOF model to be analyzed for at least one point on the dynamic region, and Dragos and Wu requires iteration and integration, which increases the computational cost of developing a point on the normalized *P-I* curve. Two simplified approaches that allow rapid initial assessment of a wide array of reinforced concrete blast loaded components for a wide range of far-field blast loads were therefore developed in this paper. The approaches presented in this paper build on these previous studies and are tailored specifically to precast concrete wall panels.

A set of *P-I* diagrams for use as a prescriptive design aid can be readily developed using a limited number of one-time SDOF analyses if the user is only concerned with a narrow range of design parameters. However, if designers must consider a broader range of parameters in their blast-resistant projects, the possible variation of *P-I* diagrams can increase significantly due to a large range of available component types, design configurations, and detailing schemes. For example, solid concrete non-load-bearing wall panels analyzed for possible combinations consisting of three response limits, three boundary conditions, five span lengths, five concrete compressive strengths, and 13 reinforcement ratios would require 2925 unique *P-I* diagrams. To more efficiently represent these combinations of possible curves, two simplified approaches are evaluated in this paper to calculate *P-I* curves for solid nonprestressed concrete wall panels: a *P-I* normalization approach and a curve-fitting methodology. The effectiveness of each method is compared with conventional SDOF analyses. A spreadsheet-based tool was developed in conjunction with this study to facilitate seamless integration of one of the simplified design methodologies into preliminary design practices. This tool was developed for use in preliminary design phases and is not intended for preparing official engineering calculations in applications where blast-resistant design provisions are required. The approach will, however, facilitate increased accuracy when estimating panel design and detailing requirements during the bidding phase. These tools can thereby allow precast concrete producers to gain a competitive advantage when considering projects involving blast-resistant facilities.

Calculation of minimum pressure and impulse asymptotes

A *P-I* curve consists of three regions: impulsive, dynamic, and quasi-static (**Fig. 3**). For increased computational efficiency, each region can be calculated separately and then assembled to complete the full curve. The quasi-static and impulsive regions can be characterized by the minimum pressure P_0 and impulse I_0 that the component can resist. These limits can be represented as asymptotes using Eq. (5) and (6), respectively, where E is the strain energy of the resistance function (the area under the resistance function curve) up to the deformation corresponding to the desired level of protection at y_{limit}.

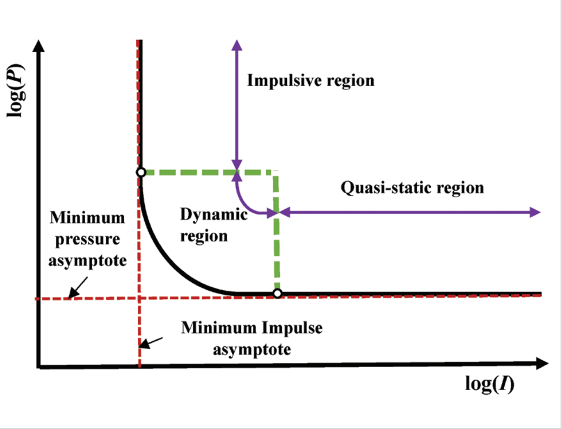


Figure 3. Representative loading regions of a pressure-impulse curve. Note: I = impulse; P = reflected pressure.

K_{LM} is the load-mass transformation factor corresponding to the range of the resistance function (elastic or plastic) in which the desired level of protection falls.

$$I_0 = \sqrt{2 \times E \times K_{LM} \times M} \quad (5)$$

$$P_0 = \frac{E}{y_{Limit}} \quad (6)$$

The minimum pressure and impulse asymptotes serve as the baseline for properly calculating the entire *P-I* curve for both simplified methods that are evaluated in this paper. The dynamic region, which provides connectivity between these asymptotes, can be generated using one of two simplified approaches detailed in this paper. The first method, a normalization approach, uses two dimensionless factors to shift the asymptotes of a control *P-I* curve, which was obtained from one SDOF calculation of a control element that has representative characteristics of a blast-resistant precast concrete panel. Using the *P-I* curve from the control element provides appropriate curvature to the dynamic region of a *P-I* curve for the panel of interest. The second method, a curve-fitting approach, is performed by first calculating both asymptotes and then using an analytical formula that considers the magnitude of each asymptote to define the dynamic region of the *P-I* curve between those asymptotes. For these reasons, careful consideration is given to properly characterizing and calculating the asymptotes.

Both methods are applicable for one-way, single-span, non-load-bearing reinforced concrete solid wall panels with simple-simple, simple-fixed, and fixed-fixed boundary conditions. The proposed approach has not been validated for use on prestressed and insulated concrete wall panels and would require further development. Like most simplified blast design calculations, far-field explosive conditions and uniform pressure distributions are assumed, and the blast-pressure versus time history is idealized as a triangular pulse load (neglecting the negative phase). Near-field explosive conditions are not considered. A more detailed discussion of each simplified method is presented in the following sections.

Simplified method 1: Normalization approach

The first approach generates the *P-I* curve for a given wall panel by shifting a baseline *P-I* curve for a control component according to the ratio of the pressure and impulse asymptotes between the element of interest and the control component. This approach builds on a normalization analysis strategy from Dragos et al.[8] and introduces additional features to facilitate ease of implementation and use with precast concrete wall panels. To provide a basis for the normalization strategy, a control component is introduced. A *P-I* curve for the control component is developed using traditional SDOF methodology and acts as a baseline for generating curves for other component configurations. Because it is fully defined, the control curve can be scaled to determine the *P-I* curve for the component of interest. The shift between the *P-I* curves for the control component and the component of interest is based on the ratio of the asymptotes calculated using Eq. (7) and (8) for impulse ψ_I and pressure ψ_P, respectively.

$$\psi_I = \frac{I_0}{I_{0,c}} \quad (7)$$

$$\psi_P = \frac{P_0}{P_{0,c}} \quad (8)$$

The control asymptotes are defined as $P_{0,c}$ and $I_{0,c}$, and those for the component of interest are identified as P_0 and I_0. These factors are used to shift the control *P-I* curve and generate the *P-I* curve for the component of interest at the desired level of protection. The control component is shifted by multiplying the respective control component impulse and pressure vectors I_c and P_c by these factors (Eq. [9] and [10]), resulting in the *P-I* curve for the component of interest (**Fig. 4**). For

Table 1. Impulse and pressure values of the control component for normalization approach

Impulse, psi-ms	Pressure, psi	Impulse, psi-ms	Pressure, psi	Impulse, psi-ms	Pressure, psi
66.07	500.00	71.77	8.52	343.53	2.65
68.27	318.04	81.47	5.42	428.07	2.58
68.67	202.30	91.76	4.49	533.42	2.53
68.67	128.67	114.34	3.68	664.69	2.49
68.97	81.85	127.37	3.45	1032.11	2.43
68.57	52.06	142.48	3.28	1286.12	2.40
67.77	33.11	177.54	3.03	3864.07	2.34
67.47	21.06	221.24	2.87	n.d.	n.d.
68.47	13.40	275.68	2.74	n.d.	n.d.

Note: n.d. = no data. 1 psi = 6.895 kPa.

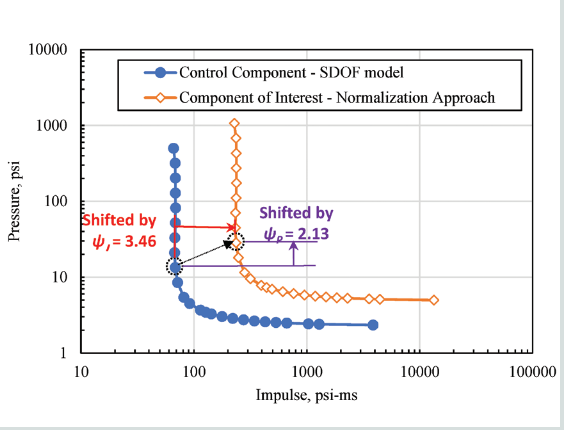


Figure 4. Example of shifting control component. Note: SDOF = single degree of freedom; Ψ_I = ratio of impulse asymptote of component of interest to control component; Ψ_P = ratio of pressure asymptote of component of interest to control component. 1 kPa = 0.1450 psi.

the purposes of this study, the control wall panel component illustrated in **Fig. 5** was selected. **Table 1** summarizes the *P-I* curve data points for the control component. The associated $I_{0,c}$ and $P_{0,c}$ are 58.54 psi-ms and 2.30 psi (403.62 kPa-ms and 15.86 kPa), respectively.

$$I = \psi_I \times I_c \tag{9}$$

$$P = \psi_P \times P_c \tag{10}$$

The accuracy of the normalization approach was evaluated by comparing the resulting shifted *P-I* curves with traditional SDOF analyses. A case study of 9450 wall panel design configurations was performed. Errors between the normalization approach and the SDOF analyses were calculated for each wall panel design and level of protection. For each design, the error was calculated over the three separate regions: impulsive, dynamic, and quasi-static (**Fig. 6**). The total error for each curve was determined using a root mean square calculation. The errors calculated in the impulsive and quasi-static (pressure-governed) regions were determined by simply

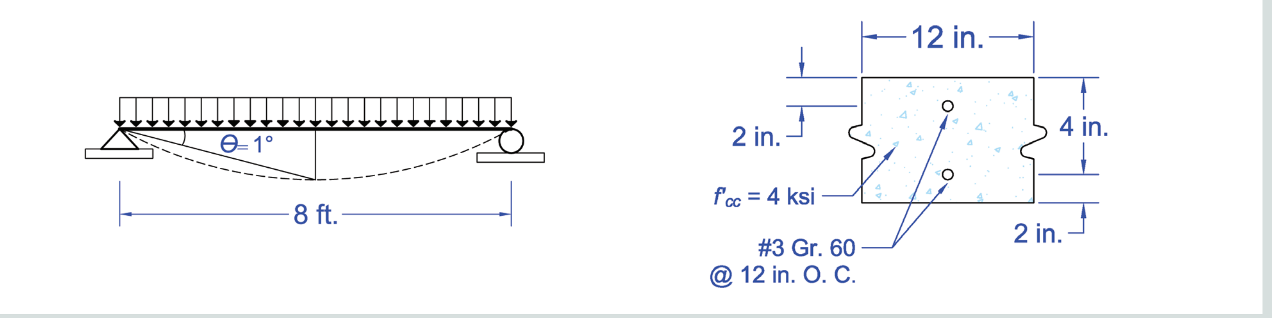


Figure 5. Selected control component data: span and response limit (left) and cross-section properties (right). Note: @ = at; # = number; f'_{cc} = concrete compressive strength; Gr. = grade; O. C. = on center; θ = support rotation response limit. 1 in. = 25.4 mm; 1 ft = 0.305 m; 1 ksi = 6.895 MPa.

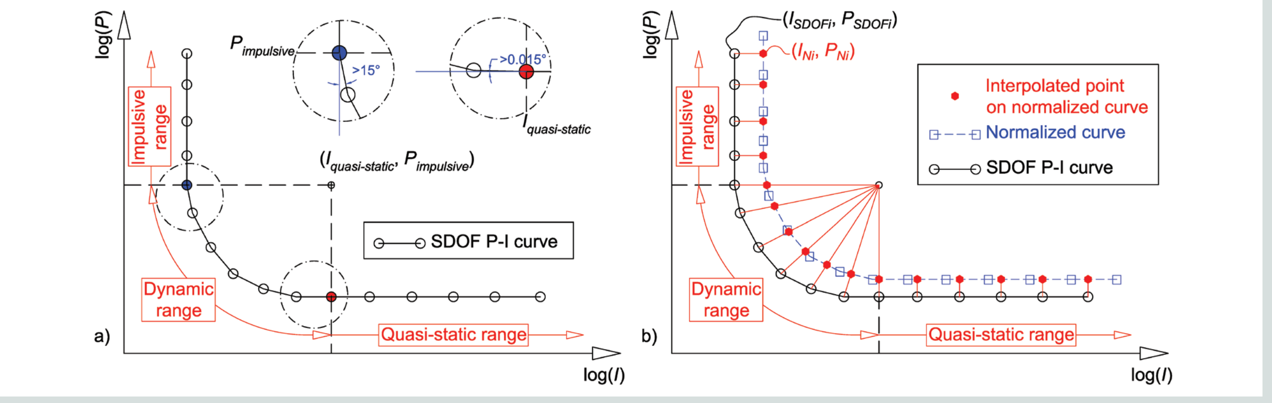


Figure 6. Graphical representation of the pressure-impulse curve error calculation. Note: I = impulse; I_{Ni} = iterated impulse value on the normalized curve; $I_{quasi\text{-}static}$ = point corresponding to the intersection of the minimum impulse value for the quasi-static region; I_{SDOFi} = iterated impulse value on the single degree of freedom curve; P = reflected pressure; $P_{impulsive}$ = point corresponding to the intersection of the minimum pressure value in the impulsive region; P_{Ni} = iterated pressure value on the normalized curve; P_{SDOFi} =iterated pressure value the on the single degree of freedom curve; SDOF = single degree of freedom.

calculating the horizontal or vertical difference, respectively, between the normalization and SDOF *P-I* curves at each discrete point in those regions. For the dynamic region, the differences between the curves were determined using a radial distance approach. To do this, the central point of the radial curve must first be determined. For this evaluation, this point was chosen as the intersection of the minimum pressure value $P_{impulsive}$ in the impulsive region and the minimum impulse value $I_{quasi\text{-}static}$ for the quasi-static region (Fig. 6). $P_{impulsive}$ is located where the slope of the *P-I* curve, moving from the impulsive to dynamic region, exceeds an angle of 15 degrees. In a similar manner, $I_{quasi\text{-}static}$ is located where the slope of the *P-I* curve, moving from the quasi-static to dynamic region, exceeds an angle of 0.015 degrees. A smaller angle change is used for this region because the overall slope of the transition between the quasi-static and dynamic regions is more gradual than for the impulsive to quasi-static regions. Due to the shift, the points on the normalized curve do not align perfectly with their SDOF counterparts along the radial line intersections. To compute the error along the radial lines, the normalized curve is re-discretized relative to points on the SDOF curve (Fig. 6). The error between the normalized curve and the SDOF solution was calculated using Eq. (11), where subscripts *SDOF* and *NM* represent the values for the SDOF and normalization curves, respectively. Total errors are illustrated using a probability density function (**Fig. 7**). Approximately 95% of the examined cases have error percentages ±6%. This simplified approach has acceptable accuracy as a preliminary design tool for estimating purposes for precast concrete wall panels under blast loads. This method is also found to be well suited for computer-based computations and was therefore deployed as a spreadsheet-based design tool, which is presented later in this paper.

$$Error(\%) = \sqrt{\left(\frac{I_{SDOF} - I_{NM}}{I_{SDOF}}\right)^2 + \left(\frac{P_{SDOF} - P_{NM}}{P_{SDOF}}\right)^2} \times 100 \qquad (11)$$

Simplified method 2: Explicit curve-fitting approach

The second approach explicitly links the pressure and impulse asymptotes using a closed-form analytical expression. This explicit curve-fitting approach builds on work previously conducted by Wang et al.,[9] which developed an analytical formula to generate *P-I* curves for one-way reinforced concrete slabs. The original formula by Wang et al. is shown in Eq. (12), where factor of failure mode n is equal to 0.6 and 0.5 for flexural and shear failure modes, respectively. This approach allows reflected pressure P to be defined as a function of impulse I, or vice versa. A plot of this equation directly connects the pressure and impulse asymptotes, thereby forming the dynamic region of the *P-I* diagram (Fig. 6).

$$\left(P - P_0\right)\left(I - I_0\right)^n = 0.33\left(\frac{P_0}{2} + \frac{I_0}{2}\right)^{1.5} \qquad (12)$$

The pressure and impulse asymptotes are calculated using Eq. (5) and (6), as shown previously. The minimum pressure asymptote lies in the semistatic region and therefore shows a consistent strong agreement (that is, with errors generally less than 15%) with that calculated using SDOF methods. However, the minimum impulse asymptote obtained from Eq. (6) can exhibit slightly more error versus that calculated using SDOF methods (**Fig. 8**). Recall that the minimum impulse asymptote is calculated using a K_{LM} that assumes quasi-static deflected shapes for blast-loaded elements (Eq. [5]). As the *P-I* calculations trend toward an increasingly impulsive response at the corresponding minimum asymptote, the assumed quasi-static formulation of K_{LM} may not capture realistic variations in the shape function due to higher-order modal vibration effects. As the response of the component becomes more impulse dominant, the value of K_{LM} is expected to increase, thereby increasing the minimum impulse asymptote (shifting it to the right) and moving closer to the limits of the *P-I* curve. An impulse asymptote modification factor γ is proposed to mitigate

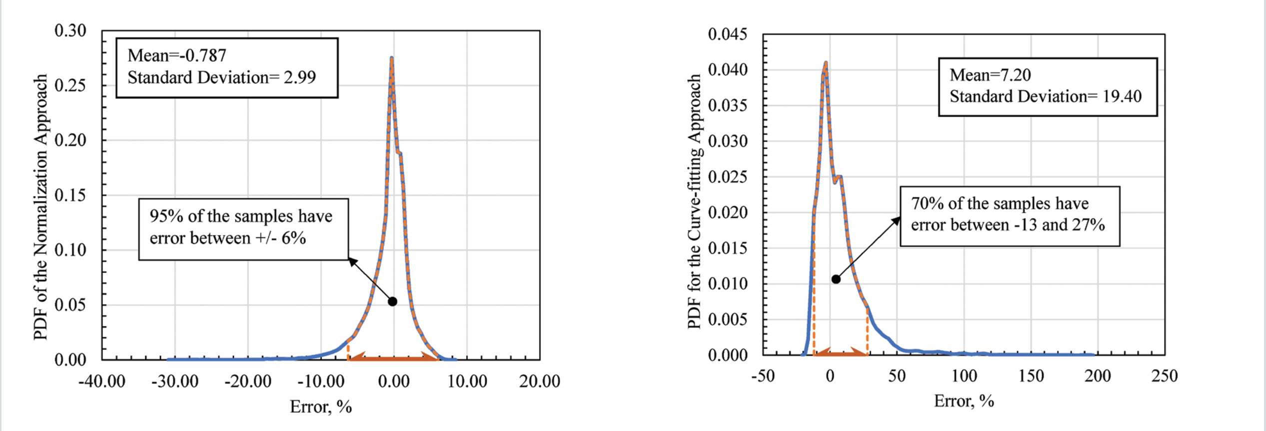


Figure 7. PDF of errors for the normalization approach (left) and the curve-fitting approach (right). Note: PDF = probability density function.

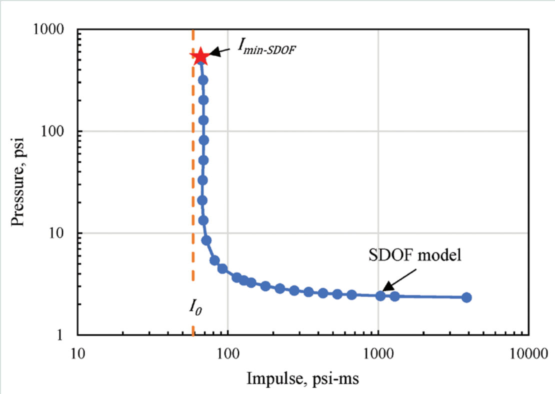


Figure 8. Divergence of pressure-impulse (P-I) curve on impulsive range from impulse asymptote. Note: I = impulse; I_0 = minimum impulse asymptote; I_{min_SDOF} = minimum impulse value of the P-I curve generated using traditional single degree of freedom; P = reflected pressure; SDOF = single degree of freedom. 1 psi = 6.895 kPa.

this effect (Eq. [13]). I_{min_SDOF} represents the minimum impulse value of the P-I curve generated using traditional SDOF methods. An evaluation of precast concrete panels was performed to determine optimal values for this factor. Natural periods T_n were calculated using Eq. (14), where k is the elastic stiffness of the component and K_{LM} corresponds to the elastic range. **Figure 9** plots the factors that were calculated for three different response limits θ for nonprestressed concrete components.[10] From these plots, conservative values of γ can be selected for several ranges of natural periods T_n. For components with low natural periods (below 50 to 100 milliseconds), γ ranges from 1.07 to 1.18. For higher natural periods, γ has a much wider distribution. Conservative floor values for these modification factors are marked with a solid orange line in Fig. 9 and are summarized in **Table 2** for relevant ranges of natural periods and response limits.

$$\gamma = \frac{I_{min,SDOF}}{I_0} \tag{13}$$

Table 2. Recommended impulse asymptote modification factor

Support rotation response limit θ, degrees	Natural period of component T_n, ms	Impulse modification factor γ
1	$T_n \le 53$	1.11
	$T_n > 53$	1.00
2	$0 < T_n \le 72$	1.09
	$72 < T_n \le 188$	1.03
	$T_n > 188$	1.00
5	$T_n \le 98$	1.07
	$T_n > 98$	1.03

$$T_n = \frac{2\pi}{\sqrt{\dfrac{k}{K_{LM} \times M}}} \tag{14}$$

To better capture wall panel response, Eq. (12) is modified such that two new parameters, a and b, replace the numeric coefficients of the equation. The new formulation in Eq. (15) also includes the impulse modification factor γ, which is multiplied by the impulse asymptote I_0. Optimal values for a and b were determined by examining 630 different panel configurations. P-I curves that are generated using the curve-fitting approach for each trial combination of a and b were investigated and compared with curves generated using traditional SDOF methods. **Figure 10** shows a surface plot of the average P-I curve error for a single panel configuration over relevant ranges of a and b values. The combination of a and b that resulted in the lowest average error for each panel configuration was selected and added to a frequency histogram in **Fig. 11**. Recommended values of $a = 0.35$ and $b = 0.80$ are the combination that most often resulted in the lowest error in Fig. 11 (right) across the 630 panel configurations.

$$(P - P_0)(I - \gamma I_0)^b = a(P_0 + \gamma I_0) \tag{15}$$

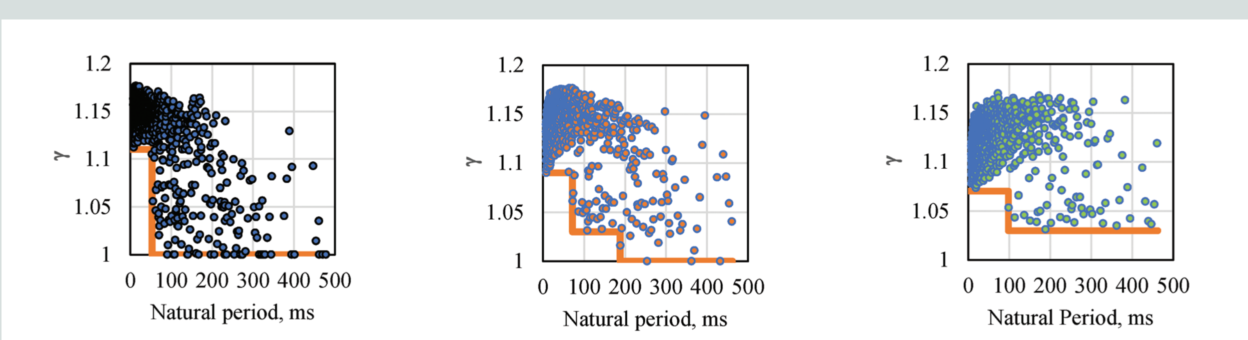


Figure 9. Impulse asymptote modification factor γ versus natural period T_n for support rotation response limit θ of 1 degree (left), 2 degrees (middle), and 5 degrees (right).

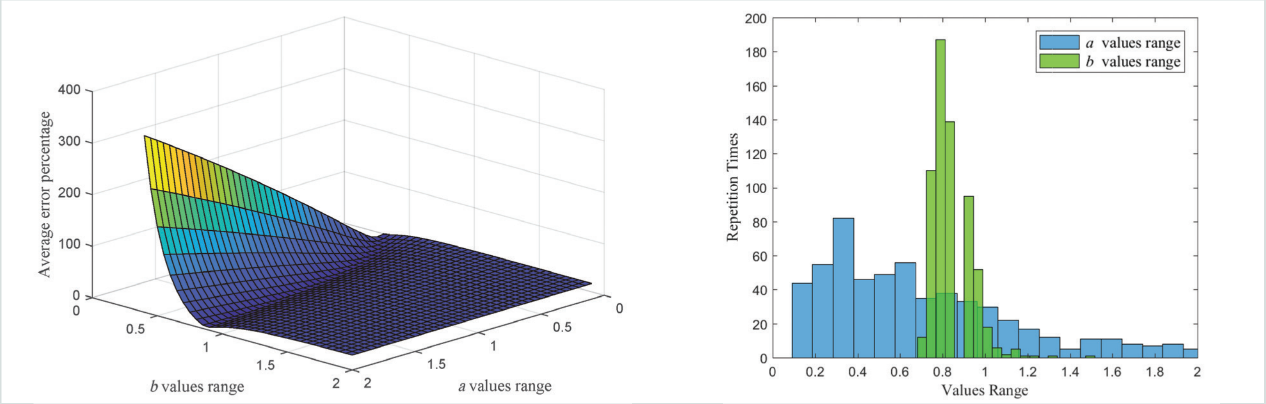


Figure 10. Average error percentage for *a* and *b* values for a representative panel (left). Frequency histograms of *a* and *b* values that result in minimum error for all panel configurations (right). Note: *a* = curve-fitting parameter 1; *b* = curve-fitting parameter 2.

For verification, *P-I* curves that are generated using Eq. (15) with the recommended values of *a*, *b*, and γ were compared with SDOF solutions for the 630 panel configurations. Figure 7 summarizes the error percentage of each point along the *P-I* curves across all panel configurations as a probability density function. The probability density function shows that error percentages range from approximately -20% to 200%. Approximately 70% of the examined cases have error percentages between -13% and 27%. Although this approach results in a wider range of potential error than the normalization method, the curve-fitting method still enables an efficient and reasonably accurate generation of *P-I* curves for preliminary design of blast-resistant precast concrete panels using a closed-form equation compared with SDOF analyses, which have higher computational expense. Because it is closed form, the curve-fitting method is well suited for implementation in design handbooks for estimating purposes.

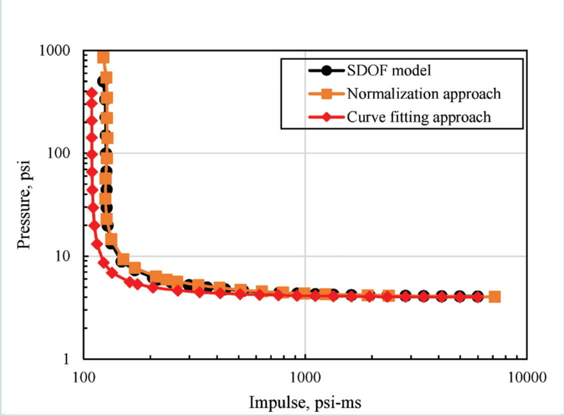


Figure 11. Targeted component pressure-impulse curves obtained using the normalization and curve-fitting approaches as well as single degree of freedom analysis. Note: SDOF = single degree of freedom. 1 psi = 6.895 kPa.

P-I curve development example

The following example shows the implementation of the two proposed approaches.

Required: Develop a *P-I* curve for the wall component outlined here with a support rotation limit of 1 degree using the normalization approach and the curve-fitting approach.

Given: A simply supported 12 ft (3.65 m) tall and 8 in. (203.2 mm) thick wall panel with no. 5 (16M) reinforcing bars at 12 in. (304.8 mm) on center. The tension reinforcement is located at a depth of 6 in. (152.4 mm) with a 2 in. (50.8 mm) cover to center of bars. The concrete has a compressive strength of 4 ksi (27.58 MPa), and grade 60 (420 MPa) reinforcement is used. The concrete density is 150 lb/ft^3 (2402 kg/m^3). Static and dynamic increase factors for the steel reinforcement are 1.10 and 1.17, respectively. The dynamic increase factor for concrete is 1.19.

Note: A unit width will be analyzed, and top reinforcement is neglected.

Procedure

Normalization approach

Step 1. Obtain given parameters of the control component.

Step 2. Establish given parameters of the targeted component.

Step 3. Determine dynamic moment capacity of the targeted component.

Step 4. Compute elastic stiffness and ultimate resistance at midspan for the targeted component.

Step 5. Compute the impulse and pressure asymptotes of the targeted component using Eq. (5) and (6), respectively.

Step 6. Compute impulse and pressure normalization factors using Eq. (7) and (8), respectively.

Step 7. Develop the impulse and pressure curve for the targeted component using Eq. (9) and (10), respectively.

Step 8. Compute the reaction for the targeted component.

Curve-fitting approach

Step 1. Same as in step 2 of the normalization approach.

Step 2. Same as in step 3 of the normalization approach.

Step 3. Same as in step 4 of the normalization approach.

Step 4. Same as in step 5 of the normalization approach.

Step 5. Determine the impulse asymptote modification factor from Table 2.

Step 6. Develop the impulse and pressure curve for the targeted component using Eq. (15).

Solution

Normalization approach

Step 1. Given parameters of the control component (Fig. 5)

Boundary conditions	Simply supported
Steel area of control component	A_{sc} = 0.11 in.^{2} (70.97 mm^2)
Span length of control component	L_c = 8 ft (2.4 m)
Depth of reinforcement of control component	d_c = 4 in. (101.6 mm)
Concrete compressive strength of control component	f'_{cc} = 4 ksi (27.58 MPa)
Thickness of control component	h_c = 6 in. (152.4 mm)
Concrete density	γ_c = 150 lb/ft^3 (2402 kg/m^3)
Minimum impulse asymptote of control component	$I_{0,c}$ = 58.54 psi-ms (403.63 kPa-ms)
Minimum pressure asymptote of control component	$P_{0,c}$ = 2.30 psi (15.86 kPa)
Impulse and pressure for response limit 1 degree	See Table 2

Step 2. Given parameters of the targeted component

Boundary conditions	Simply supported
Steel area of targeted component	A_{sT} = 0.31 in.^{2} (200 mm^2)
Span length of targeted component	L_T = 12 ft (3.65 m)
Depth of reinforcement of targeted component	d_T = 6 in. (152.4 mm)
Concrete compressive strength of targeted component	f'_{cT} = 4 ksi (27.58 MPa)
Thickness of targeted component	h_T = 8 in. (203.2 mm)
Unit width of targeted component	b_T = 12 in. (304.8 mm)
Concrete density	γ_c = 150 lb/ft^3 (2402 kg/m^3)
Concrete elastic modulus	$E_c = 33\times 150^{1.5}\times\sqrt{4000}$ = 3834 ksi (26,435 MPa)
Dynamic steel tensile strength of targeted component	f_{dyT} = 1.17 × (1.10 × 60) = 77.22 ksi (532.4 MPa)
Dynamic concrete compressive strength of targeted component	f_{dcT} = 1.19 × 4 = 4.76 ksi (32.8 MPa)

Step 3. Determine dynamic moment capacity of the targeted component.

Dynamic moment capacity

$$M_{du} = A_{sT}\times f_{dyT}\times\left(d_T - 0.5\times\frac{A_{sT}\times f_{dyT}}{0.85\times f_{dcT}\times b_T}\right)$$

$$= 0.31\times 77.22\times\left(6 - 0.5\times\frac{0.31\times 77.22}{0.85\times 4.76\times 12}\right)$$

$$= 137.73 \text{ kip-in. } (15.56 \text{ kN-m})$$

Step 4. Compute elastic stiffness and ultimate resistance at midspan of the targeted component.

Average moment of inertia $I_{av} = \frac{I_g + I_{cr}}{2} = \frac{512 + 60.54}{2}$

$= 286.27$ in.^{4} (119×10^6 mm^4)

where

I_g = gross moment of inertia

I_{cr} = cracked moment of inertia

Elastic stiffness $k = \frac{384 \times E_c \times I_{av}}{5 \times L_T^4 \times b_T}$

$= \frac{384 \times 3834000 \times 286.27}{5 \times 144^4 \times 12}$

$= 16.34$ psi/in.

Ultimate resistance $r_u = \frac{8 \times M_{du}}{L_T^2 \times b_T}$

$= \frac{8 \times 137730}{144^2 \times 12}$

$= 4.43$ psi (30.54 kPa)

Step 5. Compute the impulse and pressure asymptotes of the targeted component.

Load mass factor $K_{LM} = 0.78$ (elastic), 0.66 (plastic)

Mass $M = \frac{h_T \times \gamma_c}{g} = \frac{8 \times 0.08681 \times 1000^2}{386}$

$= 1799$ psi-ms^2/in.

where

g = gravitational acceleration

Midspan displacement at 1 degree $y_{limit} = \tan(\theta) \times L_T \times 0.5$
$= 1.26$ in. (32 mm) (larger than displacement at ductility of one y_e)

Strain energy of the resistance function $E = 0.5 \times r_u \times y_e + r_u \times (y_{limit} - y_e)$

$= 4.97$ lb/in. (0.87 kN/m)

Minimum impulse asymptote $I_0 = \sqrt{2 \times E \times K_{LM} \times M}$

$= \sqrt{2 \times 4.97 \times 0.66 \times 1799} =$
108.57 psi-ms
(748.6 kPa-ms)

Minimum pressure asymptote $P_0 = E/y_{limit} = 4.97/1.26$
$= 3.95$ (27.2 kPa)

Step 6. Compute impulse and pressure normalization factors.

Ratio of asymptotes for impulse $\Psi_I = I_0/I_{0,c} = 108.57/58.54 = 1.86$

Ratio of asymptotes for pressure $\Psi_P = P_0/P_{0,c} = 3.95/2.30 = 1.72$

Step 7. Develop the impulse and pressure curve for the targeted component.

Table 3. Impulse and pressure values of the targeted component using normalization approach

Impulse, psi-ms	Pressure, psi	Impulse, psi-ms	Pressure, psi	Impulse, psi-ms	Pressure, psi
122.49	859.50	133.06	14.65	636.90	4.56
126.57	546.71	151.04	9.32	793.64	4.44
127.31	347.75	170.12	7.72	988.96	4.34
127.31	221.19	211.99	6.33	1232.34	4.27
127.87	140.69	236.14	5.93	1913.54	4.17
127.12	89.49	264.16	5.64	2384.46	4.13
125.64	56.92	329.16	5.22	7163.99	4.03
125.08	36.21	410.17	4.93	n.d.	n.d.
126.94	23.03	511.11	4.71	n.d.	n.d.

Note: n.d. = no data. 1 psi = 6.895 kPa.

Table 4. Impulse and pressure values of the targeted component using curve-fitting approach					
Impulse, psi-ms	Pressure, psi	Impulse, psi-ms	Pressure, psi	Impulse, psi-ms	Pressure, psi
122.48	500.00	170.12	7.33	1109.62	4.28
125.18	334.09	205.21	6.21	1338.49	4.23
125.58	223.23	216.18	5.93	1614.57	4.19
125.88	149.15	247.53	5.61	1947.60	4.16
126.18	99.66	298.59	5.23	2349.31	4.13
126.58	66.59	360.18	4.96	2833.89	4.12
126.98	44.49	434.47	4.77	3418.42	4.10
126.98	29.73	524.09	4.62	4123.51	4.08
128.18	19.86	632.19	4.50	4974.04	4.07
133.08	13.27	762.58	4.41	6000.00	4.06
148.18	8.87	919.88	4.34	n.d.	n.d.

Note: n.d. = no data. 1 psi = 6.895 kPa.

Impulse $I = \Psi_I \times I_c$ (see **Table 3**)

Pressure $P = \Psi_P \times P_c$ (see Table 3)

Step 8. Compute the support reaction for the targeted component.

Reaction of targeted component $R_T = \dfrac{r_u \times L_T}{2} = \dfrac{4.43 \times 144}{2}$

$= 318.96$ lb/in. (55.86 kN/m)

Curve-fitting approach

Step 1 to 4. Same solution as step 2 to step 5 in the normalization approach.

Step 5. Determine impulse asymptote modification factor.

Natural period $T_n = 2 \times \pi \times \sqrt{K_{LM} \times M / k}$

$= 2 \times \pi \times \sqrt{0.66 \times 1799 / 16.34}$

$= 53.56$ ms

Impulse asymptote modification factor (from Table 2) $\gamma = 1$

Step 6. Develop the impulse and pressure curve of the targeted component.

By assuming values of impulse, use Eq. (15) to calculate pressure $P = \dfrac{a\left(P_0 + \gamma I_0\right)}{\left(I - \gamma I_0\right)^b} + P_0$

$$= \frac{0.35 \times \left(3.95 + 1 \times 108.57\right)}{\left(I - 1 \times 108.57\right)^{0.8}} + 3.95$$

(The solution is shown in **Table 4.**)

Summary

The *P-I* curves that were obtained for the targeted component using the normalization and curve-fitting approaches are compared against a *P-I* curve obtained from SDOF analysis of the same component (**Fig. 11**).

Simplified *P-I* generating tool

To further facilitate the implementation of the proposed methods into conventional engineering practices for preliminary design purposes, a spreadsheet-based tool was developed to enable users to easily generate the *P-I* curves based on a set of input parameters that correspond to the wall panel of interest. Due to its overall lower error distribution, the normalization approach was selected as the featured method for this tool. The main purpose of this tool is to provide a rapid evaluation of component damage when subjected to a blast load. To use this tool, the user must first obtain the blast pressure and impulse demands (that is, the design-basis threat) and select the desired response limit. The constitutive properties of the component of interest can then be input to the spreadsheet, after which the tool determines whether the component is able to satisfy the desired level of protection for the given blast demands. If the initial design does not satisfy the response limit (that is, the blast demand point falls above and to the right of the *P-I* curve), the designer can easily change the design parameters until the component meets the desired level of protection. The tool also provides the plastic moment capacity of the wall and the reaction forces, which allows the designer to rationally estimate the connection types needed to satisfy the given blast demands. The reactions are calculated following the equivalent static reaction approach for a one-way element, as defined in UFC 3-340-02.[1] The simplified tool is provided on the PCI website at https://www.pci.org/2019July-Appx. For situations where a blast-resistant opening is present in the panel, the pressure demand should be increased by the ratio of the tributary panel width to the structural width. It should be noted that this tool is intended only for preliminary design and should not be used for final design evaluation.

Conclusion

Two approaches were developed to facilitate rapid generation of blast-resistant capacity curves (pressure-impulse or *P-I* curves) for precast concrete wall panels: a normalization approach and a curve-fitting method. The *P-I* curves obtained from these methods are intended as a preliminary design approach to assess the blast resistance of non-load-bearing solid precast concrete wall panels with conventional reinforcement. The approaches are also limited to far-field blast loads. The normalization approach was compared with the traditional SDOF model for a large sample of precast concrete panel configurations and was found to have low error percentages, with 95% of the examined cases having error percentages being ±6%. *P-I* curves of 630 precast concrete panel configurations were computed with the curve-fitting approach and also compared with conventional SDOF estimates. The curve-fitting approach resulted in a wider spread of error, with approximately 70% of the cases having error percentages between -13% and +27%. Due to the higher error, the curve-fitting approach was found to be better suited for simplified hand calculations or implementation in a handbook. A spreadsheet-based tool was developed using the normalization approach to further facilitate its deployment by practitioners. Both methods are intended for preliminary design calculations that would be conducted during the bidding phase. These methods can rapidly determine whether a panel design of interest meets the desired level of protection and can assist the designer in estimating rational connection types by providing support reaction forces caused by the response to blast loading.

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Notation

a = curve-fitting parameter 1

A_{sc} = steel area of control component

A_{sT} = steel area of targeted component

b = curve-fitting parameter 2

b_T = unit width of targeted component

d_c = depth of reinforcement of control component

d_T = depth of reinforcement of targeted component

E = strain energy of the resistance function

E_c = concrete elastic modulus

f'_{cc} = concrete compressive strength of control component

f'_{cT} = concrete compressive strength of targeted component

f_{dyT} = dynamic steel tensile strength of targeted component

f_{dcT} = dynamic concrete compressive strength of targeted component

$F(t)$ = blast pressure versus time history

h_c = thickness of control component

h_T = thickness of targeted component

I = impulse

I_0 = minimum impulse asymptote

$I_{0,c}$ = minimum impulse asymptote of control component

I_{av} = average moment of inertia

I_c = control component impulse vector

I_{cr} = cracked moment of inertia

I_g = gross moment of inertia

I_{min_SDOF} = minimum impulse value of the *P-I* curve generated using traditional single degree of freedom methods

I_{Ni} = iterated impulse value on the normalized curve

I_{NM} = impulse values for the normalization curve

$I_{quasi\text{-}static}$ = point corresponding to the intersection of the minimum impulse value for the quasi-static region

I_r = positive phase impulse

I_{SDOF} = impulse values for the single degree of freedom curve

I_{SDOFi} = iterated impulse value on the single degree of freedom curve

k = elastic stiffness

K_L = load factor

K_{LM} = load-mass transformation factor

K_M = mass factor

L = span length

L_c = span length of control component

L_T = span length of targeted component

M = lumped mass of the system

M_{du} = dynamic moment capacity

n = factor of failure mode

P = reflected pressure

P_0 = minimum pressure asymptote

$P_{0,c}$ = minimum pressure asymptote of control component

P_c = control component pressure vector

$P_{impulsive}$ = point corresponding to the intersection of the minimum pressure value in the impulsive region

P_{NM} = pressure values for the normalization curve

P_{Ni} = iterated pressure value on the normalized curve

P_r = peak reflected pressure

P_{SDOF} = pressure values for the single degree of freedom curve

P_{SDOFi} = iterated pressure value on the single degree of freedom curve

r_u = ultimate resistance

R = resistance function of the component

R_T = reaction of targeted component

t = time

t_d = positive phase duration

T_n = natural period

y_e = displacement at ductility of one

y_{limit} = the deformation corresponding to the level of protection

$y(t)$ = midspan displacement of the panel as a function of time

γ = impulse asymptote modification factor

γ_c = concrete density

θ = support rotation response limit

$\Phi(x)$ = shape function for the actual system

Ψ_I = ratio of asymptotes for impulse

Ψ_P = ratio of asymptotes for pressure

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Abstract

This paper presents two simplified blast-resistant design methodologies for rapid generation of pressure-impulse (*P-I*) curves for solid precast concrete wall panels with conventional reinforcement: a normalization approach and a curve-fitting methodology. Both methods are primarily intended for preliminary design calculations that would be conducted during the bidding phase. The normalization approach involves shifting a control *P-I* curve to determine the respective curve for any panel configuration based on its constitutive properties. This method generally results in low error and, due to its streamlined computational efficiency, was used to develop a spreadsheet-based design tool. The curve-fitting approach, which uses an analytical formula to calculate the dynamic region of the *P-I* curve, is suited for simplified hand calculations or design handbooks. Both approaches exhibit superior computational efficiency relative to traditional single degree of freedom analyses and are well suited for rapid assessments of panel performance during the preliminary design phase.

Keywords

Bid process, blast design, design tool, estimating, pressure-impulse curve, wall panel.

Review policy

This paper was reviewed in accordance with the Precast/Prestressed Concrete Institute's peer-review process.

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