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Performance-based framework for evaluating the flexural response of precast concrete wall panels to blast loading

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ABSTRACT

This paper proposes a new performance-based approach to assess the response of precast concrete wall panels to blast loading. Conventional blast-resistant design and analysis methods currently rely on simplified assumptions of component behavior and limit states that are based on visual observations of damage. The proposed methodology allows for the computation of nonlinear moment-rotation resistance functions, component-specific response criteria and deformation-dependent load-mass transformation factors as a function of the panel's section mechanics and material properties. Computational modeling is used to examine direct correlations between constitutive behavior and critical panel response milestones. These milestones are used as the basis for calculating performance-based limit states, which vary as a function of panel geometry and material properties. Resistance functions and flexural response obtained from the performance-based approach are compared to those calculated via conventional methods and obtained from experimental test data. Comparisons are made between the proposed performance limits and existing response criteria for a set of concrete wall panels under experimental blast loading. Parametric studies were conducted to assess the relationships between performance-based limit states and variations of the panel's geometric properties and material behavior. The proposed methodology enables improved blast-resistant design of precast concrete wall panels versus the current state-of-practice without significant sacrifices in analytical efficiency.

1. Introduction

Blast-resistant design considerations are commonly implemented for petrochemical, military, government or other high-risk facilities. The source of blast loading may be intentional (such as acts of terrorism) or accidental. Building envelopes provide the first line of defense against blast loading by shielding both the occupants and the load-bearing structural elements from the blast-induced shock wave. Due to their weight and flexural customizability, precast concrete wall panels are commonly chosen as a design solution for blast-resistant façades. Most blast resistant design procedures in current practice allow the use of simplified methods, such as the generalized single degree-of-freedom (SDOF) approach [1], to determine the response of structural components to blast loading. In order to solve the SDOF equation of motion and calculate blast-induced deformations, a resistance function considering the component's material behavior, cross section geometry, span length, load application, and boundary conditions must first be determined. These resistance functions are typically calculated based on strength limit states using simplified approximations, particularly elastic-perfectly-plastic idealized material behavior.

In current practice, levels of damage are determined by comparing the peak response of the component against a set of prescribed response criteria. Damage levels ranging from superficial damage to a complete loss or blowout of the component are specified in design standards in accordance with observed damage from experimental tests. Response limits define a boundary between successive levels of component damage and are intended to represent significant milestones of component response. The boundaries between damage levels are tied to the component's peak deformation and are specified according to the component type (i.e. column, beam, wall panel, etc.) and construction type (i.e. reinforced or prestressed concrete, structural steel, etc.). Due to the reliance on empirically established damage states, the current state-of-practice approach does not provide a direct correlation between material limit states and the response limits or damage states. This can lead to potentially conservative or unconservative predictions of the actual blast-induced damage. For example, two reinforced concrete panels with different reinforcement ratios will reach yield, nominal, and ultimate strength at different deformation levels. However, both panel designs would currently fall within the same set of response criteria in most of the current blast resistant design specifications.

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This paper proposes a new performance-based framework that allows for the computation of component-specific resistance functions and response criteria for use in evaluating precast concrete panels subjected to blast loading. Significant response milestones are used as the basis for defining performance-based limit states, each of which will be examined as a function of panel constitutive properties. These limit states represent significant transitions on the component resistance functions, which are typically used for simplified blast-resistant design of building components. Computational modeling is utilized to calculate panel responses from which direct correlations between constitutive behavior and overall panel performance are observed. Comparisons are made between resistance functions computed using conventional methods and the proposed framework. Using the proposed framework, discrepancies between calculated panel response milestones and several current prescriptive blast design criteria are presented. A set of parametric studies is included to assess the dependency of the performance-based response metrics on variations of component design parameters. The proposed framework is shown in this paper to be effective for calculating performance-based resistance functions and limit states for precast concrete wall panels and facilitates component level detailing and innovation when targeting specific response milestones.

2. Background

Precast concrete wall panels are commonly used in buildings that must resist accidental or intentional blast pressures. These panels are designed to resist dynamic lateral forces (represented as a short duration pressure time history) resulting from a specified blast event, in addition to conventional design loads such as wind or handling. Selective detailing of connections and reinforcement can enable panels with conventional geometry to resist the blast loading. Conventional wall panels may be solid or insulated, with the latter comprising a layer of expanded or extruded polystyrene, polyisocyanurate or other types of thermally insulating materials sandwiched between two concrete wythes. Precast concrete fabrication environments facilitate the production of structural and architectural components with tighter construction tolerances and enhanced aesthetic value. Numerous combinations of panel thickness, reinforcement layout and type, discrete panel-to-structure connections, insulation properties and material behavior can be used to customize the design for a wide range of structural and architectural applications. This paper focuses on the response of solid precast wall panels to blast – the response of insulated panels is also being explored by the authors [2] and will be the focus of future studies.

2.1. SDOF analysis and KLM factors

In current practice, the flexural performance of precast concrete wall panels under blast loading is commonly assessed using elastic-plastic resistance functions [3] and a generalized single degree of freedom (SDOF) analysis approach as discussed in Biggs [1]. Many precast panels are single span and do not have moment resisting connections, and they are regularly modeled as simply supported in SDOF blast analyses. Conventional approaches for these elements specify resistance functions that remain perfectly plastic after the nominal strength is reached. The panel is equated to a mass-spring system with one translational degree of freedom associated with the deformation at midspan. A normalized deflected shape is used to generate an equivalent SDOF system which accounts for the component's stiffness, strength, mass, applied load pattern, and boundary conditions. That deflected shape, which is derived from a static flexural response, is used to determine load-mass transformation factors (*KLM*) that are calculated for the elastic and plastic ranges. The elastic shape makes the simplifying assumption that the moment of inertia is uniform along the span and the plastic shape assumes a formation of a discrete plastic hinge at midspan. The elastic assumptions ignore the variation in cracked and uncracked moment of inertia along the span and the plastic assumption ignores the distributed plasticity that is present in flexural reinforced concrete components.

Previous studies have highlighted the limitations of these shape functions and transformation factors for SDOF analyses of components subjected to blast loading. Variation in the flexural deformed shape of components under blast versus those under static load can adversely affect the validity of the current empirically derived performance limits, which are calibrated to the static deformed shape. Yokoyama [4] investigated the accuracy of equivalent SDOF transformation factors from Biggs [1] via comparison to a multi-degree of freedom (MDOF) model. Good comparisons between the MDOF and traditional SDOF approaches were observed for components in the elastic range for small to moderate blast loads. However, the observed deflected shapes diverged once the element responded inelastically. The findings also suggest that the assumed static plastic deflected shapes become invalid once the magnitude and velocity of the blast's shock wave become very large. Yokoyama [5] used MDOF finite element analyses to calculate component-specific shape functions for varying levels of applied blast loading. The results showed that, depending on the magnitude of the applied blast load, the conventional SDOF approach may be unable to fully capture the realistic deflected shape. These outcomes further suggested that deflected shape functions incorporating both detailed effects of component behavior and characteristics of the applied blast load may facilitate a more accurate prediction of component response. These studies highlight the need for developing component-specific transformation factors that can more accurately calculate the dynamic response of wall panels subjected to blast loading. The proposed approach introduces deformation-dependent *KLM* factors that are progressively calculated at each load increment and therefore allow for the inclusion of nonlinear behavior and distributed component plasticity into the equivalent SDOF system.

2.2. Damage limits and response criteria

Current prescriptive response criteria for blast-resistant design are specified for antiterrorism standards [6], petrochemical facilities [7], physical security [8] and explosive safety requirements [3]. Prescriptive definitions of component damage levels specified in these standards are summarized in Table 1. Correlations between observed damage and component response measurements are based on available test data, from sources including Wright [9] and Forsen [10] and as summarized by Oswald and Bazan [11]. Response criteria for antiterrorism design in the United States [6] considers five levels of component damage, ranging from superficial (little or no damage) to blowout (when the component is completely overwhelmed). A current petrochemical standard [7] employs a simplified spectrum with three levels of response: low, medium and high. The American Society of Civil Engineers (ASCE) physical security standard [8] uses a similar three-level classification system of light, moderate and severe. Department of Defense (DoD) protective design standards [3] specify numeric damage levels increasing in severity from 1 to 4. Corresponding allowable deformation limits are typically presented in terms of component ductility ratio, μ, and equivalent support rotation, θ. Limits for reinforced and prestressed concrete wall panels, according to these standards, are summarized in Table 2. Ductility quantifies the ratio of the maximum component deformation to its yield deformation, while support rotation normalizes the maximum deformation as a function of the span length, as shown in Eq. (1) where Δ is the lateral deflection of the panel (assuming that maximum deflection occurs at the midspan, as is the case for a simply supported element) and L is the span length. The prestressed reinforcement index for prestressed concrete panels, ω_p, is calculated using Eq. (2), where A_{ps} is the area of prestressed reinforcement in tension, f_{ps} is the prestressing steel stress at nominal component strength, b is the width of the component, d_p is the depth