BLAST

Resistant Design of Reinforced Concrete Structures

By Dennis M. McCann, Ph.D., P.E. and Steven J. Smith, Ph.D., P.E.

The study of blast effects on structures has been an area of formal technical investigation for over 60 years. There are numerous texts, guides and manuals on the subject, with continuing research and technical reporting occurring at a brisk pace. However, there is limited guidance available in the literature on the direct application of established blast effects principals to structural design. Numerous efforts are under way to develop comprehensive guides and standards to fill this void. This article presents a general overview of key design concepts for reinforced concrete structures.

Blast Resistance and Progressive Collapse

Progressive collapse-resistant design mitigates disproportionately large failures following the loss of one or more structural elements. Progressive collapse-resistant design is *system-focused*, and is often divided into two approaches, direct and indirect. The direct method designs the structural system to respond to a specific threat either by providing an alternate load path in the event of failure of one or more members, or by specific local-resistance improvements of key elements. This method is similar to blast-resistant design. The indirect method provides general systemic improvements to toughness, continuity and redundancy; tension ties are an example of an indirect detailing technique.

Blast-resistant design is *element-focused*. It enhances toughness, ductility, strength and dynamic characteristics of individual structural elements for resistance to air-blast induced loading. This article is devoted to blast-resistant design, though there is overlap with progressive collapse-resistant design.

What's Special About Blast Loading?

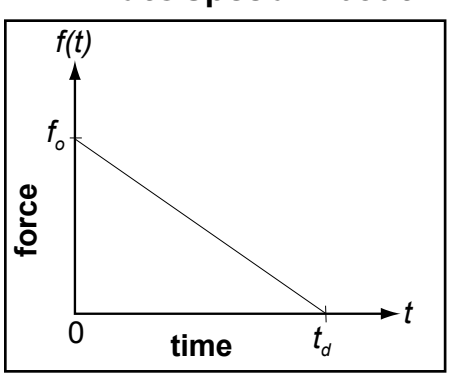

Figure 1: Idealized blast pulse with a peak intensity, f_0 and duration, t_d

This article specifically addresses the affects of shock loading from air-blast. This type of load is applied to the perimeter structural elements of a building due to a high explosive blast event external to the building. The pressure wave applied to the building is characterized by short duration and high intensity (*Figure 1*).

The blast wave duration, t_d, is typically in the range of 0.1 – .001 seconds. This is often much shorter than, or at most on the order of, the natural period, T_n, of typical structural elements. For situations where $t_d < 0.4T_n$ (some sources advise $t_d < 0.1T_n$), the blast wave effectively imparts an initial velocity to a structural element and the element then continues to respond at its natural frequency. The magnitude of that initial velocity, for a single-degree-of-freedom (SDOF) model, is $v = f_0 t_d / 2m$, where f_0 and t_d are shown in *Figure 1* and m is the mass. Thus, in this response regime, the mass of the structural element is the only system parameter that controls the magnitude of the initial motion of the system – the more massive the structural element, the less it will be excited by the impulse from the blast wave. In this regard, the greater mass of concrete structures can be used to great advantage.

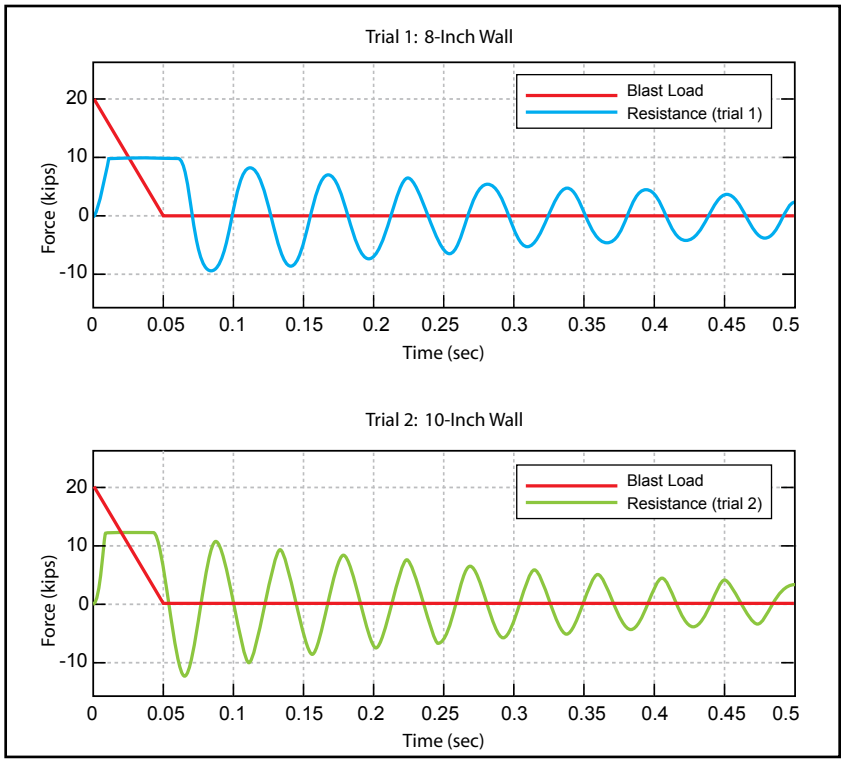

Figure 2: Applied force and internal resistance time histories (using 2% damping).

This load response to a blast is significantly different from the load response to a seismic event, for which the natural frequency of the structure, rather than the mass, is the primary factor in the response.

Response Limits and Member Analysis

The extreme nature of blast loading necessitates the acceptance that members will have some degree of inelastic response in most cases. This allows for reasonable economy in the structural design and provides an efficient mechanism for energy dissipation. This also requires the designer to understand how much inelastic response is appropriate. Greater inelastic response will provide greater dissipation of the blast energy and allow for the sizing of smaller structural elements, but it will also be accompanied by greater damage and, at some point, increased potential for failure of the element.

The U.S. Army Corps of Engineers Protective Design Center (PDC) has developed response criteria for many typical structural elements in terms of maximum allowable support rotation, θ_{max}, or ductility ratio, μ_{max}, as shown in *Tables 1* and *2 (see page 24)*. These limits were developed in conjunction with experts in the field of blast effects and are based on existing criteria and test data. The limits can be correlated to qualitative damage expectations ranging from

no damage with elements responding elastically to severe damage with elements responding far into the inelastic regime. *Table 3 (see page 25)*, provides a sampling of damage expectations for specific structural components, and *Table 4 (see page 26)* provides guidance on overall structural damage that the Department of Defense (DoD) equates with varying levels of protection.

These limits are calibrated to an equivalent single degree of system (SDOF) model of the structural member with lumped mass and stiffness, and should only be compared to responses determined in that manner. The SDOF method assumes the response of the member can be appropriately modeled as a single mode, neglecting contributions from all other modes. The calibration process used for the PDC limits incorporates mapping the idealized SDOF to actual structural response.

The undamped SDOF equation of motion is written:

$m_e \ddot{x}(t) + R(x,t) = f(t)$ where $f(t)$ is the blast load, $\ddot{x}(t)$ is the acceleration response, m_e is the equivalent or activated mass of the structural element, and $R(x,t)$ is the internal resistance as a function of time and displacement. Assuming elasto-plastic material behavior, the resistance is divided into three phases:

1) Elastic response until yield: $R(x,t) = k_e x(t)$, where k_e is the equivalent stiffness and $x(t)$ is the displacement response.
2) Plastic deformation after yield when deformation continues without increase in resistance: $R(x,t) = R_m$, where R_m is the maximum resistance.
3) Elastic rebound after reaching the maximum displacement: $R(x,t) = R_m - k_e[x_m - x(t)]$, where x_m is the maximum displacement.

While closed form solutions exist for some simple load profiles, it is often necessary to solve the SDOF equations of motion numerically. Such methods and a more complete treatment of equivalent SDOF systems can be found in texts on structural dynamics.

Design

The design procedure includes:

1) Blast load definition
2) Response limit selection
3) Trial member sizing and reinforcing
4) Nonlinear dynamic SDOF analysis of the member
5) Comparing the calculated SDOF response with the response limit and adjusting the trial member as necessary

As noted above, some amount of inelastic response is generally anticipated when designing members for blast response. Economy of design is achieved by selecting smaller members and allowing greater inelasticity. Where greater protection is warranted, larger members are selected, potentially even such that the response to the design blast threat remains elastic. While member sizes can be scaled to match the desired level of protection, proper detailing of joints, connections and reinforcing should always be provided so that the members can achieve large, inelastic deformations even if the intent is for elastic response (thus providing greater margins against an actual blast that is larger

Table 1: Maximum Response Limits for SDOR Analysis of Flexural Elements[a]

Element Type	Expected Element Damage							
	Superficial		Moderate		Heavy		Hazardous	
	μ_{max}	θ_{max}	μ_{max}	θ_{max}	μ_{max}	θ_{max}	μ_{max}	θ_{max}
Reinforced Concrete								
Single-Reinforced Slab or Beam	1	–	–	2°	–	5°		10°
Double-Reinforced Slab or Beam without Shear Reinforcement[b]	1	–	–	2°	–	5°	–	10°
Double-Reinforced Slab or Beam with Shear Reinforcement[b]	1	–	–	4°	–	6°	–	10°
Slab or Beam with Tension Membrane[c] (Normal Proportions[d])	1	–	–	6°	–	12°	–	20°
Slab or Beam with Tension Membrane[c] (Deep Elements[d])	1	–	–	6°	–	7°	–	12°
Prestressed Concrete[e]								
Slab or Beam with $\omega_p > 0.30$	0.7	–	0.8	–	0.9	–	1	–
Slab or Beam with $0.15 \le \omega_p \le 0.30$	0.8	–	$0.25/\omega_p$	1°	$0.29/\omega_p$	1.5°	$0.33/\omega_p$	2°
Slab or Beam with $\omega_p \le 0.15$ and Shear Reinforcement[b]	0.8	–	$0.25/\omega_p$	1°	$0.29/\omega_p$	1.5°	$0.33/\omega_p$	2°
Slab or Beam with $\omega_p < 0.15$ and Shear Reinforcement[b]	1	–	–	1°	–	2°	–	3°
Slab or Beam with Tension Membrane[c,f] (Normal Proportions[d])	1	–	–	1°	–	6°	–	10°
Masonry								
Unreinforced[g]	1	–	–	1.5°	–	4°	–	8°
Reinforced	1	–	–	2°	–	8°	–	15°
Structural Steel (Hot-Rolled)								
Beam with Compact Section[h]	1	–	3	3°	12	10°	25	20°
Beam with Noncompact Section[h]	0.7	–	0.85	3°	1	10°	1.2	20°
Plate Bent about Weak Axis	4	1°	8	2°	20	6°	40	12°

[a]Where a dash (–) is shown, the corresponding parameter is not applicable as a flexural response limit
[b]Stirrups or ties that satisfy the minimum requirements of Section 11.5.6 of ACI 318 and enclose both layers of flexural reinforcement throughout the span length
[c]Tension membrane forces shall be restrained by a member capable of resisting the corresponding loads and typically cannot be developed along a slab free edge
[d]Elements with normal proportions have a span-to-depth ratio greater than or equal to 4; deep elements have a span-to-depth ratio less than 4
[e]Reinforcement index $\omega_p = (A_{ps}/bd)(f_{ps}/f'_c)$
[f]Values assume bonded tendons, draped strands and continuous slabs or beams
[g]Values assume wall resistance controlled by brittle flexural response or axial load arching with no plastic deformation; for load-bearing walls, use Superficial or Moderate damage limits to preclude collapse
[h]Limiting width-to-thickness ratios for compact and noncompact sections are defined in ANSI/AISC 360

Developed from PDC-TR-06-08, ***Single Degree of Freedom Response Limits for Antiterrorism Design****, Protective Design Center, U.S. Army Corps of Engineers, October 2006.*

than the design blast). Without proper detailing, it is uncertain whether a structure intended for blast resistance will achieve the design intent. The January, 2007 STRUCTURE® article *Concrete Detailing for Blast* provides effective recommendations for concrete detailing. In addition to that article, general design and detailing considerations include:

Beams

1) Balanced design often leads to a strong column – weak beam approach, with the intent that beam failure is preferable to column failure.
2) Provide sufficient shear transfer to floor slabs so that directly applied blast loads can be resisted by the diaphragms rather than weak-axis beam bending.
3) Transfer girders should be avoided in regions identified as having a high blast threat.

Columns

Design critical columns to be able to span two stories, in the event that lateral bracing is lost, particularly when using a weak beam approach.

Detailing and Connections

1) Use special seismic moment frame details.
2) Avoid splices at plastic hinge locations.
3) Provide continuous reinforcing through joints.
4) Used hooked bars where continuous reinforcing is not possible (particularly at corners).

Example

Consider an exterior panel wall measuring 12 feet tall by 30 feet long, attached to the primary structural framing system at its top and bottom. The wall is to be designed to resist the effects of a high explosive blast resulting in a 12 pounds per square inch (psi) peak reflected pressure and a positive phase pulse duration, t_d = 50 milliseconds.

Since the wall is attached at its top and bottom, the vertical reinforcement will provide the primary load-path and blast resistance; as such this example will be limited to design of the vertical reinforcement. As an initial trial, an 8-inch thick wall with #4 reinforcing bars spaced every 6 inches at each face will be considered. For each trial section, the bending and shear (yield) strength of a unit strip are computed, applying strength increase factors (SIF) to account for the actual (rather than code minimum) strength of materials and dynamic increase factors (DIF) to account for the increased strength of materials exhibited under

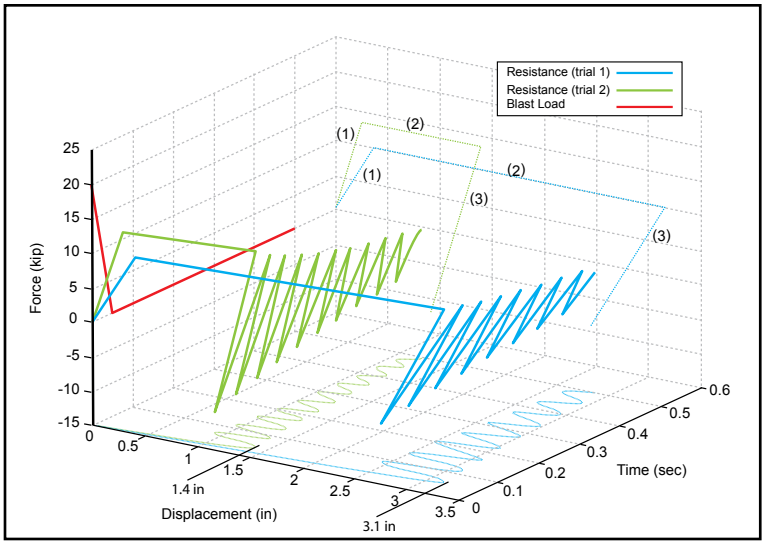

Figure 3: Three dimensional SDOF response histories for each trial section (using 2% damping). Two dimensional resistance-displacement and displacement-time projections are also shown. Regions of (1) initial elastic deformation, (2) plastic deformation, and (3) elastic rebound are indicated on the resistance-displacement projections.

Table 2: Maximum Response Limits for SDOF Analysis of Compression Elements[a]

Element Type	Expected Element Damage							
	Superficial		Moderate		Heavy		Hazardous	
	μ_{max}	θ_{max}	μ_{max}	θ_{max}	μ_{max}	θ_{max}	μ_{max}	θ_{max}
Reinforced Concrete								
Single-Reinforced Slab or Beam-Column	1	–	–	2°	–	2°	–	2°
Double-Reinforced Slab or Beam-Column without Shear Reinforcement[b]	1	–	–	2°	–	2°	–	2°
Double-Reinforced Slab or Beam-Column with Shear Reinforcement[b]	1	–	–	4°	–	4°	–	4°
Walls and Seismic Columns[c,d]	0.9	–	1	–	2	–	3	–
Non-seismic Columns[c,d]	0.7	–	0.8	–	0.9	–	1	–
Masonry								
Unreinforced[c]	1	–	–	1.5°	–	1.5°	–	1.5°
Reinforced	1	–	–	2°	–	2°	–	2°
Structural Steel (Hot-Rolled)								
Beam-Column with Compact Section[f,g]	1	–	3	3°	3	3°	3	3°
Beam-Column with Noncompact Section[f,g]	0.7	–	0.85	3°	0.85	3°	0.85	3°
Column (Axial Failure)[d]	0.9	–	1.3	–	2	–	3	–

[a]Where a dash (–) is shown, the corresponding parameter is not applicable as a flexural response limit
[b]Stirrups or ties that satisfy the minimum requirements of Section 11.5.6 of ACI 318 and enclose both layers of flexural reinforcement throughout the span length
[c]Seismic columns have ties or spirals that satisfy, at a minimum, the requirements of Section 21.12.5 of ACI 318; see Chapter 9 for complete detailing requirements
[d]Ductility ratio is based on axial deformation, rather than flexural deformation
[e]Values assume wall resistance controlled by brittle flexural response or axial load arching with no plastic deformation; for load-bearing walls, use Superficial or Moderate damage limits to preclude collapse
[f]Limiting width-to-thickness ratios for compact and noncompact sections are defined in ANSI/AISC 360
[g]Use connection shear capacity, rather than element flexural capacity, to calculate ultimate resistance for analysis

*Developed from PDC-TR-06-08, **Single Degree of Freedom Response Limits for Antiterrorism Design**, Protective Design Center, U.S. Army Corps of Engineers, October 2006.*

Table 3: Qualitative Damage Expectations for Reinforced Concrete Elements

Element - Issue	Superficial	Moderate	Heavy	Hazardous
Beam and Column - Reinforcement	No damage	No damage	Local buckling of longitudinal reinforcement	Fracture of longitudinal and transverse reinforcement
Beam and Column - Core Concrete	No visible, permanent structural damage	Minor cracking (repairable by injection grouting)	Substantial damage	Rubble
Beam and Column - Cover	No visible, permanent structural damage	Substantial spalling	Lost	Lost
Beam and Column - Stability	None	None	Local buckling of longitudinal reinforcement	Global buckling
Connection - Reinforcement	None	None	Limited fracture and compromised anchorage at joint (load transfer maintained)	Fracture and loss of anchorage at joint
Connection - Concrete	No visible, permanent structural damage	Minor spalling and cracking (repairable)	Substantial damage	Rubble at core
Slab - Diaphragm Action	Hair line cracking in the vicinity of the blast; concrete and reinforcement essentially undamaged; diaphragm action uncompromised for the lateral force and gravity force resistance	Spalling of concrete cover limited to the immediate vicinity of blast; connection to supporting beam intact except in the immediate vicinity of blast where localized separation is likely; diaphragm action uncompromised for lateral force and gravity resistance.	Minor damage concrete and reinforcement; connection to supporting beam yields but fracture is likely in vicinity of blast resulting in localized separation	Significant damage to concrete and reinforcement; diaphragm action compromised for lateral force resistance but provides stability for gravity force resistance

fast load application rates. SIF and DIF values for reinforced concrete design are suggested in *Design of Blast Resistant Buildings in Petrochemical Facilities* (ASCE 1997) and TM5-1300, *Structures to Resist the Effects of Accidental Explosions* (USACE 1990). The lesser of the computed bending or shear strengths is used as the maximum resistance, R_m, in the elasto-plastic resistance function. R_m = 10 kips for the 8-inch thick unit strip trial section.

The equivalent SDOF is then computed. The effective stiffness in this case would be computed based on the center deflection of a simply supported beam. Since both elastic and plastic response is anticipated, the moment of inertia used for the stiffness calculation is taken as the average of the gross and cracked moments of inertia. Load (stiffness) and mass transformation factors may be applied to compute the effective mass of the trial section. The effective mass can be thought of as the portion of the total mass of the section that participates in the SDOF response. A more complete treatment of mass participation and load-mass factors used to compute the effective mass can be found in *Introduction to Structural Dynamics* (Biggs 1964). The 8-inch thick unit strip trial section has an equivalent stiffness, k_e = 27.7 kip/in, and an equivalent mass, m_e = 2.24 pounds-seconds^2/inch, giving a natural period of vibration of the equivalent SDOF of

$$T_n = \sqrt{2\pi\, m_e / k_e} = 0.057 \text{ seconds (sec.)}.$$

Since the pulse duration and natural period are sim-ilar (i.e. td / Tn = 0.05 sec/0.057 sec ≈ 1) in this case, the assessment of the response requires solution of the SDOF equation of motion. Numerical solution of the SDOF equation of motion gives a peak displacement response of x_m = 3.1 inches with a permanent deformation after rebound of x_p = 2.7 inches and a ductility ratio of $\mu = x_m / (x_m - x_p) = 7.75$. The peak displacement corresponds to rotations at the top and bottom of the wall section of $\theta = \tan^{-1}(x_m / 0.5h_{wall})$ = 2.5 degrees, which exceeds the response limit for flexural members of θ_{max} = 2.0 degrees. Hence, the analysis must be conducted again with a new trial section.

Using the same reinforcing steel spacing, but increasing the wall thickness to 10 inches, increases the maximum resistance to 13.4 kips, the equivalent stiffness to 53.5 kip/inch, and the effective mass to 2.8 pounds-seconds^2/inch. This results in a natural period of 0.045 seconds for the new trial section. Numerical solution of the equivalent SDOF with these parameters gives a peak displacement response of 1.4 inches with a permanent deformation of 1.1 inches, or a ductility demand just over 4.5 times the elastic limit. Rotations at the top and bottom of the wall are reduced to 1.1 degrees, which is now within the response limit. *Figure 2 (see page 22)* shows the applied force and internal resistance time histories for each of the trial sections. *Figure 3 (page 24)* shows the SDOF response for each trial in three dimensions, with two-dimensional projections of the resistance-displacement curves and the displacement time history.

Summary

Reinforced concrete can provide substantial protection from even extreme blast loading. The relatively large mass of concrete elements provides an inherent resistance to impulsive loads. Structural design considerations include sizing members to provide an expected degree of deformation and associated damage and optimizing the structure to resist and transfer blast loads in a reliable manner. Proper detailing is the final critical component of the design process to ensure that the structural elements have sufficient toughness to achieve the desired inelastic deformations.▪

Table 4 and References on next page

Table 4: Department Of Defense Damage Descriptions

Level of Protection	Potential Overall Structural Damage[1]	Component Damage[2]
Below AT Standards[3] (Blowout)	Severely damaged; frame collapse/massive destruction; little left standing.	The component is overwhelmed by the blast load causing failure and debris with significant velocities.
Very Low (VLLOP)	Heavily damaged - onset of structural collapse: major deformation or primary and secondary structural members, but progressive collapse is unlikely; collapse of non-structural elements.	A portion of the component has failed, but there are no significant debris velocities.
Low (LLOP)	Building is damaged beyond repair; major deformation of non-structural elements and secondary structural members and minor deformation of primary structural members; but progressive collapse is unlikely.	The component has not failed, but it has significant permanent deflections causing it to be unrepairable; the component is not expected to withstand the same blast load again without failing.
Medium (MLOP)	Building is damaged, but repairable; minor deformations of non-structural elements and secondary structural members and no permanent deformation in primary structural members.	The component has some permanent defection; it is generally repairable, if necessary, although replacement may be more economical and aesthetic; the component is expected to withstand the same blast load again without failing.
High (HLOP)	Superficially damaged; no permanent deformation of primary and secondary structural members or non-structural elements.	No visible permanent damage.

Note 1: Department of Defense definition in terms of overall building damage. Shown only for reference.
Note 2: Definitions developed for CEDAW components. Components at each LOP do not necessarily cause the overall building to have the same LOP. A separate correlation between component LOP and building LOP based in part on component type is necessary, but is outside the scope of this report.
Note 3: This is not an official level of protection. It only defines a realm of more severe structural response that can provide additional useful information in some cases.

BakerRisk Project No. 02-0752-001, ***Component Explosive Damage Assessment Workbook (CEDAW) Methodology Manual V1.0,*** *prepared for Protective Design Center, U.S. Army Corps of Engineers, June 2005.*

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