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 underway 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?**

This article specifically addresses the effects 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 – 0.001 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 $v = f_o \phi d/2m$, where $f_o$ 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.

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, $\theta_{max}$, or ductility ratio, $\mu_{max}$, 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:

\[ m_\ast \ddot{x}(t) + R(x,t) = f(t) \]

where \( f(t) \) is the blast load, \( \ddot{x}(t) \) is the acceleration
response, \( m_\ast \) 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: \( R(x,t) = k_\ast x(t) \), where \( k_\ast \)
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:

\[ R(x,t) = R_m - k_\ast (x(t) - x(t)_{\text{max}}) \]

where \( x(t)_{\text{max}} \) 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
than anticipated when designing members for blast response).
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, \( t_p = 50 \) 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 applied blast loads can be resisted by the diaphragms rather than weak-axis beam bending.

### Table 2: Maximum Response Limits for SDOF Analysis of Compression Elements*

<table>
<thead>
<tr>
<th>Element Type</th>
<th>Superficial</th>
<th>Moderate</th>
<th>Heavy</th>
<th>Hazardous</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \mu_{\text{max}} )</td>
<td>( \theta_{\text{max}} )</td>
<td>( \mu_{\text{max}} )</td>
<td>( \theta_{\text{max}} )</td>
</tr>
<tr>
<td>Reinforced Concrete</td>
<td>Single-Reinforced Slab or Beam-Column</td>
<td>1</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>Double-Reinforced Slab or Beam-Column without Shear Reinforcement</td>
<td>1</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>Double-Reinforced Slab or Beam-Column with Shear Reinforcement</td>
<td>1</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Walls and Seismic Columns(^{c,d} )</td>
<td>0.9</td>
<td>–</td>
<td>1</td>
<td>–</td>
</tr>
<tr>
<td>Non-seismic Columns(^{c,d} )</td>
<td>0.7</td>
<td>–</td>
<td>0.8</td>
<td>–</td>
</tr>
<tr>
<td>Masonry</td>
<td>Unreinforced(^b )</td>
<td>1</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td></td>
<td>Reinforced</td>
<td>1</td>
<td>–</td>
<td>–</td>
</tr>
<tr>
<td>Structural Steel (Hot-Rolled)</td>
<td>Beam-Column with Compact Section(^{c,e} )</td>
<td>1</td>
<td>–</td>
<td>3</td>
</tr>
<tr>
<td></td>
<td>Beam-Column with Noncompact Section(^{c,e} )</td>
<td>0.7</td>
<td>–</td>
<td>0.85</td>
</tr>
<tr>
<td></td>
<td>Column (Axial Failure)(^d )</td>
<td>0.9</td>
<td>–</td>
<td>1.3</td>
</tr>
</tbody>
</table>

\(^c\) Where a dash (\(-\)) is shown, the corresponding parameter is not applicable as a flexural response limit.

\(^d\) 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.

\(^e\) 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.

\(^b\) Ductility ratio is based on axial deformation, rather than flexural deformation.

\(^\) 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.

\(^c\) Limiting width-to-thickness ratios for compact and noncompact sections are defined in ANSI/AISC 360.

\(^d\) Use connection shear capacity, rather than element flexural capacity, to calculate ultimate resistance for analysis.

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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

$$T_n = \sqrt{\frac{2\pi m_e}{k_e}} = 0.057 \text{ seconds}.$$  

Since the pulse duration and natural period are similar (i.e. $t_d/T_n = 0.05$ sec/0.057 sec $\approx 1$) 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 $x_{\text{max}} = 3.1$ inches with a permanent deformation after rebound of $x_f = 2.7$ inches and a ductility ratio of $\mu = x_{\text{max}} / (x_{\text{max}} - x_f) = 7.75$. The peak displacement corresponds to rotations at the top and bottom of the wall section of $\theta = \tan^{-1} \left( \frac{x_{\text{max}}}{0.5h_{\text{max}}} \right) = 2.5$ degrees, which exceeds the response limit for flexural members of $\theta_{\text{max}} = 2.0$ 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/in, 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

<table>
<thead>
<tr>
<th>Level of Protection</th>
<th>Potential Overall Structural Damage</th>
<th>Component Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Below AT Standards³ (Blowout)</td>
<td>Severely damaged; frame collapse/massive destruction; little left standing.</td>
<td>The component is overwhelmed by the blast load causing failure and debris with significant velocities.</td>
</tr>
<tr>
<td>Very Low (VLLOP)</td>
<td>Heavily damaged - onset of structural collapse: major deformation or primary and secondary structural members, but progressive collapse is unlikely; collapse of non-structural elements.</td>
<td>A portion of the component has failed, but there are no significant debris velocities.</td>
</tr>
<tr>
<td>Low (LLOP)</td>
<td>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.</td>
<td>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.</td>
</tr>
<tr>
<td>Medium (MLOP)</td>
<td>Building is damaged, but repairable; minor deformations of non-structural elements and secondary structural members and no permanent deformation in primary structural members.</td>
<td>The component has some permanent deflection; 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.</td>
</tr>
<tr>
<td>High (HLOP)</td>
<td>Superficially damaged; no permanent deformation of primary and secondary structural members or non-structural elements.</td>
<td>No visible permanent damage.</td>
</tr>
</tbody>
</table>

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.

References