This report comprises Appendix A of the forthcoming 8th edition of the *PCI Design Handbook*. The report has been approved by the PCI Blast Resistance and Structural Integrity Committee and the Technical Activities Committee. It is being published in *PCI Journal* for public comment. Please submit your comments to Journal@PCI.org by March 31, 2014. Include "Appendix A" in the subject line of your email.

PCI Design Handbook: Appendix A: Blast-resistant design of precast, prestressed concrete components

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A.1 Introduction

Abating the threat of explosions is crucial in the design of government and high-profile public facilities. The threat from explosive blasts is primarily located on the building exterior; however, for petrochemical and some manufacturing facilities, interior explosions may pose a greater threat. This appendix will focus on the design of precast concrete building components to resist blast loads, with a primary focus on exterior explosive blasts. Additional considerations for buildings designed for blast resistance may include threats of ballistic impact, exterior breach, or forced entry. These topics are not included in the scope of the appendix. Additional information on blast design considerations for precast concrete components can be found in PCI MNL-141-12 *Blast-Resistant Design Manual*[1] and *Blast Considerations*,[2] published by PCI.

A.1.1 Definitions

Ductility: The ratio of the maximum deflection of the component to the yield deflection of the component.

Impulse: The area under the blast pressure–versus–time curve.

Level of protection: The desired performance designated for the building system based on the use, potential threat, and owner needs.

Maximum deflection: The largest deflection that the component reaches when subjected to the defined blast load.

Mechanism: The formation of a stable local yield point in a component that allows for continued rotation without additional loading.

Response limit: The allowable support rotation or ductility of a component for a given level of protection.

Shape function: The displaced shape of the component as a function of its length normalized to the maximum deflection.

Standoff distance: The distance from the center of the detonation to the component of interest.

Support rotation: The peak deflection of the component normalized to the distance from the support.

Yield deflection: The deflection of the component corresponding to formation of a mechanism.

A.2 Blast-resistant design criteria

Blast-resistant design requirements for a given project are typically defined in the project specifications. The specifications generally provide information to define the blast load, an acceptable dynamic analysis methodology, and performance requirements that often refer to response criteria. Response criteria are essentially maximum deflection limits, as discussed in section A5.3. There are many different forms for specifying this information, but often the blast load is defined in terms of a peak pressure and impulse and the response criteria are defined in terms of an acceptable damage level or level of protection as defined in a referenced document. The acceptable analysis methodology may be defined, or the specification may require a minimum number of years of relevant design experience for the blast design engineer, assuming that this will help ensure that an appropriate analysis methodology is used. In almost all cases, a design methodology where the structural components are analyzed as equivalent single-degree-of-freedom (SDOF) systems is acceptable, as discussed in section A5.1. The following documents may be referenced in the specifications:

- *Physical Security Criteria for Federal Facilities*[3] (For Official Use Only)
- United Facilities Criteria (UFC) 3-340-01: *Design and Analysis of Hardened Structures to Conventional Weapons Effects*[4] (for official use only)
- UFC 3-340-02: *Structures to Resist the Effects of Accidental Explosions*[5]
- UFC 4-010-01: *DoD Minimum Antiterrorism Standards for Buildings*[6]
- *Single Degree of Freedom Structural Response Limits for Antiterrorism Design*[7]
- *Design of Blast Resistant Buildings in Petrochemical Facilities*[8]

These criteria contain specific performance requirements. The requirements define the acceptable structural damage in terms of response limits for individual element types based on the defined levels of protection. The response limits are specified as either a maximum support rotation or maximum ductility, which is defined as the ratio of the maximum deflection to the elastic deflection.

For typical precast concrete components, blast-resistant design is primarily focused on the design of the exterior building components and their connections. Blast-resistant building components are designed to provide acceptable performance in the event of an explosion. This is defined in the design specifications or governing blast criteria, which will prescribe the acceptable severity of damage, with moderate or heavy damage usually acceptable as long as the component remains attached to the building and the building remains stable after the explosion. For a given explosion scenario, precast concrete components and their connections to the building should be designed to resist the resulting blast load. Framing members directly supporting precast concrete components need to be designed to resist the dynamic reaction forces from the component. Alternatively, they can be designed to resist the blast load acting over their tributary area.

Unlike seismic and wind loads, blast loads have a short duration that is measured in milliseconds. Therefore, the large mass associated with overall building response often provides enough inertia so that the building's lateral-force-resisting system is adequate to resist blast loads. The fact that a blast wave applies positive pressure on all sides of the building also limits the effect on the lateral framing system, though the pressure is higher on the sides facing the explosive source. The lateral-force-resisting system on smaller one- or two-story buildings should typically be checked to consider the combined effects of overall frame sway and direct blast-load effects on frame members. Conventionally designed foundation systems for large and small buildings almost always have adequate mass and strength to resist the short-duration reaction loads from the building response to the blast load. Therefore, dynamic foundation analysis is not required for most blast-resistant buildings and almost never for larger buildings that are more than two stories.

A.3 Blast loads

An explosion is a release of energy that occurs so rapidly that there is a local accumulation of energy at the site of the explosion.[9] This energy expands as a shock or pressure wave, causing a localized, short-duration rise in air pressure at the wave front followed by a short-duration negative pressure, or suction, with a lower magnitude. The peak pressure may rise anywhere from tenths of a pound per square inch to thousands of pounds per square inch depending on the mass of the explosive and distance from the explosion. In addition, the energy release may cause ground shock, fragmentation, cratering, thermal radiation, or any combination of these effects.

Explosions can be caused by many sources, including solid explosives, dust or flammable vapor clouds, pressure vessel bursts, rapid electric-energy discharge in a spark gap, rapid vaporization of a fine wire or thin metal strip, and molten metal contacting liquid. Most often, structural components are designed for accidental or terrorist explosions of solid explosives (also known as high explosives) and accidental industrial explosions from vapor clouds or pressure vessel bursts. Many factors affect the pressures caused by an explosion, including the type of explosion, the explosive charge's weight or mass, the charge shape and orientation, the distance from the explosive source, the height of the explosion's aboveground surface, surrounding buildings or objects, and the confinement around the explosion.

Simplified blast load prediction methods are available for some explosion scenarios. This includes the common design case of a surface burst of high explosives, where the explosion is near the ground surface at a standoff distance (that is, the distance between the explosive and the structure) from the component of interest. This case is often applicable to a terrorism scenario. The blast loads for this case and other simplified cases can be predicted as described in multiple references.[5,8,9]

Figure A.1 shows the simplified shape of blast-load time histories commonly used for blast design. The blast load rises immediately to its peak pressure and then decays linearly to ambient pressure over a duration t_d. The impulse, which is the shaded area under the pressure-versus-time curve, is a measure of the total energy in the blast load. The blast load must normally be defined in terms of at least two parameters, which are typically the peak pressure and impulse or the peak pressure and equivalent triangular duration t_d. A design blast pressure may be defined in terms of just the peak pressure only if there is an understanding that the blast-load duration is long compared with the response time of the building's structural components. This is the case for some industrial explosions and for nuclear explosions.

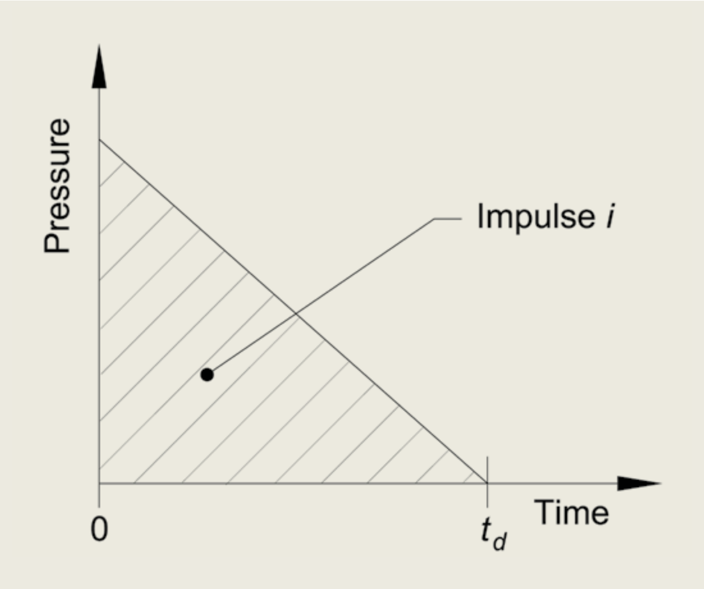

Figure A.1. Idealized blast-pressure history typically used for design. Note: t_d = equivalent triangular duration of pressure.

Figure A.2 shows the actual shape of the blast load, including the negative phase (suction pressure), which has a much lower magnitude but a longer duration than the positive phase (inward pressure). The negative phase (Fig. A.2) is often neglected in design, which is generally conservative. The simplified design blast load (Fig. A.1) always preserves the peak pressure and impulse from the actual positive phase blast load (Fig. A.2), which typically causes the equivalent triangular duration t_d (Fig. A.1) to be less than the actual positive phase blast pressure duration t_o (Fig. A.2). The negative phase can significantly reduce the maximum response of precast concrete components, especially as the span length (and, therefore, natural period) increases.[10] It is typically important to include the negative phase when analyzing test data to determine analysis results that are as close as possible to measured values. The negative phase is typically not included in the design of new components because there is more uncertainty in the prediction of the negative phase blast load compared with the positive phase blast load. The negative phase load can be included for analysis of existing components when a more accurate result is desired to avoid unnecessary structural upgrades and for conservative estimation of rebound connection forces.

Figure A.3 shows the blast load from 100 lb of trinitrotoluene (TNT) on a structural component facing the explosive source at a

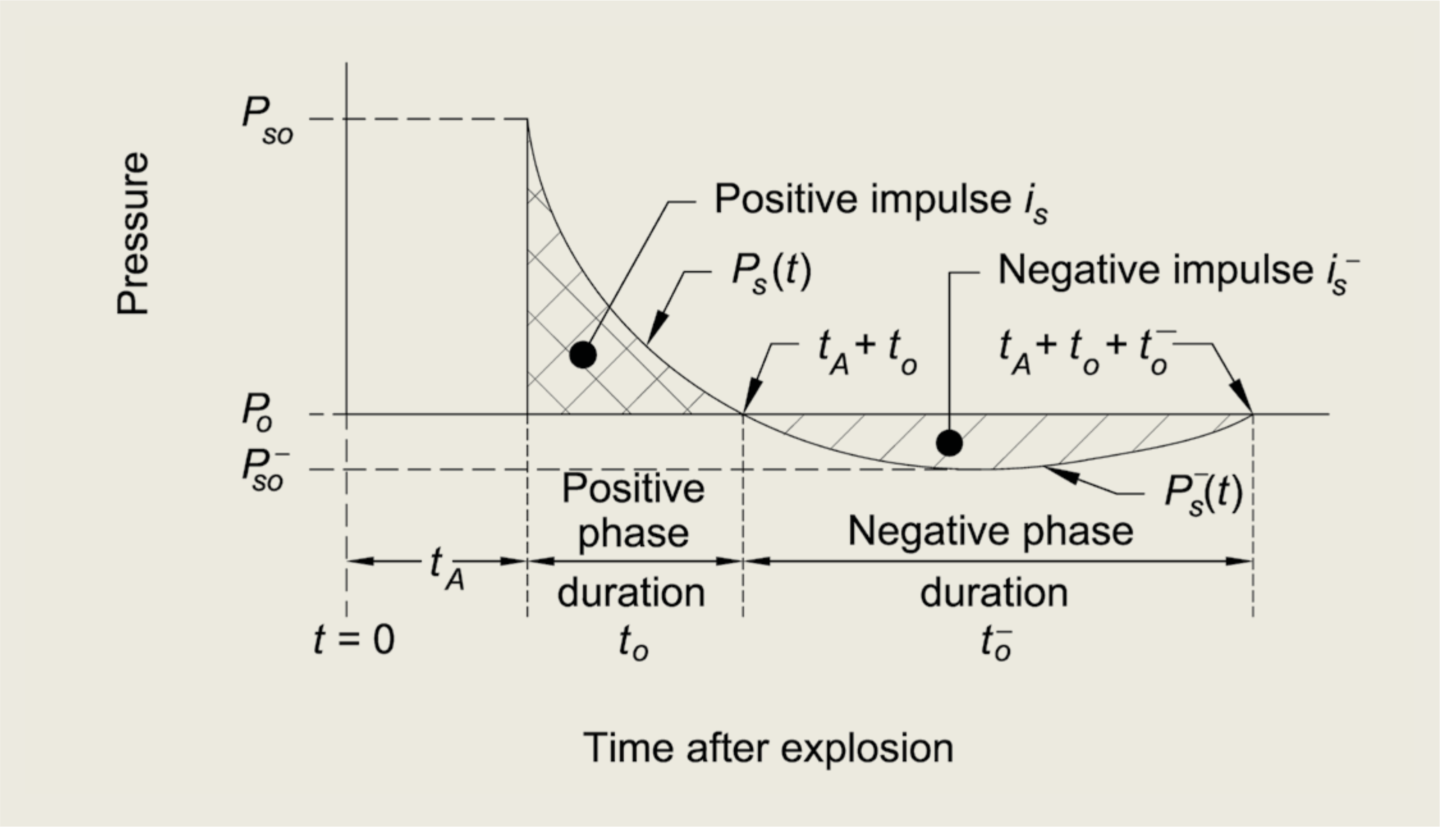

Figure A.2. Actual shape of blast load from a solid, or high explosive, explosion. Note: P_o = ambient pressure; $P_s(t)$ = positive pressure history on component; $P_s^-(t)$ = negative pressure history on component; P_{so} = peak positive pressure; P_{so}^- = peak negative pressure; t = time; t_A = time of arrival of blast pressure at structure.

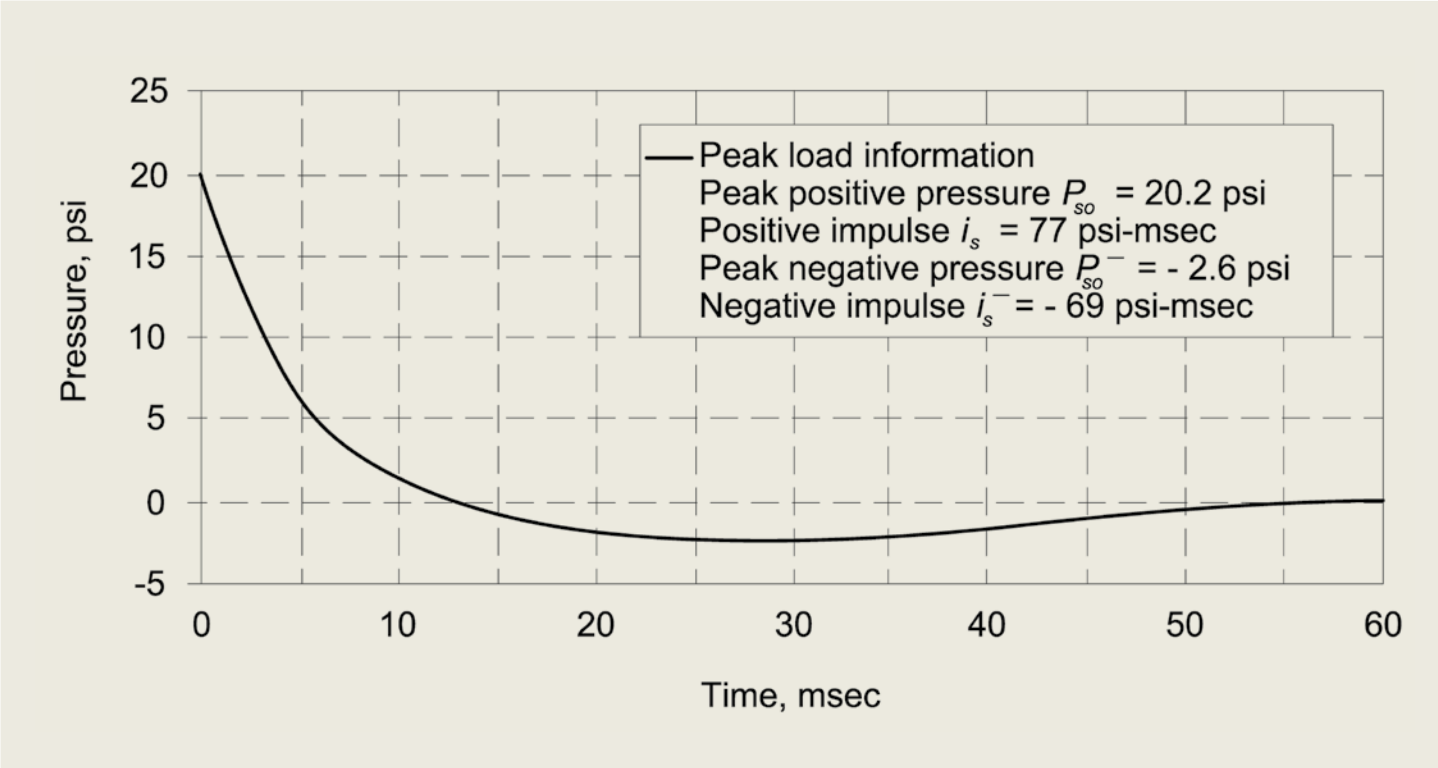

Figure A.3. Reflected blast load for 100 lb of trinitrotoluene (TNT) at 50 ft.

distance of 50 ft. The time of arrival is set to zero for the purposes of the evaluation (that is, the blast wave arrival time t_A is not shown), as is usually the case for a single-source explosive event. This figure illustrates the short duration of blast loads. The positive-phase durations of blast pressures from high explosive explosions are often less than 20 msec, though the duration is a function of the charge weight and standoff and may vary considerably. Even long-duration industrial explosions are almost always less than 300 msec.

In many cases, the design blast load on a structural component is specified for the structural engineer in terms of the peak blast pressure and the impulse. The blast-pressure history can be assumed to have the shape in Fig. A.1. For the case in Fig. A.3, the component would be required to resist a peak reflected pressure of 20.2 psi and an impulse of 77 psi-msec. This could also be specified as a peak pressure of 20.2 psi linearly decreasing over a duration of 7.6 msec.

The design external blast load for industrial explosions is often specified in terms of an overpressure or free-field pressure (that is, the nonreflected peak pressure in open air at the location of interest) and the impulse based on the free-field overpressure. In this case, the applied blast pressure on structural components must be determined based on the orientation of the building surface containing the structural component of interest relative to the explosive source.[5,8,9]

A.4 Dynamic material properties

Blast-load durations are generally on the order of milliseconds, causing similarly short component-response times. Therefore, the effects of dynamic material properties should not be ignored. Under dynamic loads, materials exhibit increased yield strengths due to strain rate effects, which can considerably improve the ultimate load capacities of blast-loaded components.

To account for the strain rate effects, a dynamic increase factor *DIF* is used when computing the strength of components, as shown in Eq. (A.1). **Table A.1**[5,8] shows typical design values of *DIF* for reinforcing steel and concrete subject to different stress conditions. The stress conditions include bending (tension and compression), compression (that is, axial members), diagonal tension (load on stirrups or lacing), direct shear (load on diagonal reinforcement), and bond. A *DIF* of 1.0 is recommended for prestressing steel strands. The value of 1.0 is based on the lack of high strain-rate characterization of strands. The value is also supported by the fact that high strain-rate testing of high-strength steels has shown that *DIF* decreases as material strength increases.[5] In addition to the *DIF* modification, the minimum specified yield strength for reinforcing steel f_y is typically modified by a strength increase factor K_e of 1.1 for blast design to account for the difference between actual reinforcing steel static yield strengths and the minimum specified values obtained from many tensile tests.

Equation (A.1) shows the dynamic yield strength for reinforcement. Equation (A.2) shows the dynamic concrete compressive strength. These values can be used along with standard design equations to determine the dynamic strength of blast-resistant components. The shear strength can be calculated based on the dynamic concrete compressive strength f'_{dc}. Dynamic increase factors should also be used for design of structural steel connections. UFC 3-340-02[5] provides appropriate *DIF* values for structural steel materials.

$$f_{dy} = f_y K_e (DIF) \quad \text{(A.1)}$$

where

f_{dy} = reinforcing steel dynamic yield stress

f_y = minimum specified reinforcing steel static yield stress

K_e = strength increase factor

DIF = applicable dynamic increase factor for reinforcing steel (Table A.1)

$$f'_{dc} = f'_c\,(DIF) \qquad \text{(A.2)}$$

where

f'_{dc} = dynamic concrete compressive strength

f'_c = specified concrete compressive strength

DIF = applicable dynamic increase factor for concrete (Table A.1)

Table A.1. Dynamic increase factors for reinforcing bars and concrete

Stress type	Dynamic increase factor *DIF*		
	Reinforcing bars*		Concrete
	f_{dy}/f_y	f_{du}/f_u	f'_{dc}/f'_{dc}
Flexure	1.17	1.05	1.19
Compression	1.10	1.00	1.12
Diagonal tension	1.00	1.00	1.00
Direct shear	1.10	1.00	1.10
Bond	1.17	1.05	1.00

* Applicable for Grade 40 and Grade 60 reinforcing steel only.
Note: *DIF* for all prestressing steel = 1.0. f'_c = specified concrete compressive strength; f'_{dc} = dynamic concrete compressive strength; f_{du} = dynamic tensile strength of reinforcement; f_{dy} = dynamic yield strength of reinforcement; f_u = specified tensile strength of reinforcement; f_y = specified yield strength of reinforcement.

A.5 General blast-design methods

Rigorous analytical methods, such as dynamic, nonlinear finite element analyses, may be used to obtain the dynamic response of blast-loaded components. These analyses can consider more complex effects, such as dynamic interaction between cladding and framing, multiple modes excited during frame-sway response, higher mode response of the component, and highly nonuniform blast loading over the area of a component.

It is generally conservative to ignore these complex effects because the dynamic interaction and higher mode response do not usually affect maximum deflections of blast-resistant components when there is significant ductile, plastic response and because blast loads are relatively uniform over individual structural components. Furthermore, rigorous finite element analyses tend to be time consuming and require users to understand the sophisticated software packages.

It is generally accepted that blast design of most building components can be performed by modeling the dynamic response of the component with an equivalent single-degree-of-freedom (SDOF) system.[5,8]

A.5.1 Single-degree-of-freedom blast design method

The SDOF method idealizes the structural component into a mass-spring system with stiffness and mass related to those in the blast-loaded structural component (**Fig. A.4**). The equivalent SDOF system is defined such that its deflection at each time step is equal to the maximum deflection of the structural component, assuming its response can be described in terms of a single degree of freedom (that is, the point of maximum deflection) and an assumed shape function that defines motion at all other points on the component relative to the maximum deflection.

Figure A.5 shows the shape functions $\phi(x)$ that are usually assumed for a simply supported beam before and after yielding in the maximum moment region. The shape function before yielding is almost always based on the deflected shape of the component from a static load with the same spatial-load distribution as the blast load (for example, uniformly distributed). The shape function after yielding is based on a hinge at the yielded moment region. Similar shape functions are defined for other boundary conditions and for two-way spanning components.[1,5,8,11]

Equation (A.3) is the equation of motion for the equivalent spring-mass system typically used for blast design. The terms in Eq. (A.3) are related to the mass, stiffness, and load on the structural component by transformation factors (that is, load and mass factors) that are combined into a single load-mass factor. The transformation factors are calculated such that the work energy, strain energy, and kinetic energy of the component expressed in terms of its single degree of freedom (the movement at midspan) and assumed shape functions equal the work, strain, and kinetic energies of the equivalent spring-mass system at each time step.[11,12] **Table A.2** shows load-mass factors for the case of a simply supported beam during elastic and plastic response with different spatial-load distributions. Load-mass factors for beams and one-way spanning slabs with other boundary conditions and for two-way spanning slabs are available in a number of references.[1,5,8,11,12] Damping can also be included in the equation of motion but is often neglected because the peak response almost always occurs during the first response cycle before the cumulative effect of the damping force becomes significant.

$$K_{LM}M[y''(t)] + R(y(t)) = F(t) \qquad \text{(A.3)}$$

where

K_{LM} = load-mass factor (a function of the deflected shape as tabulated in Table A.2)

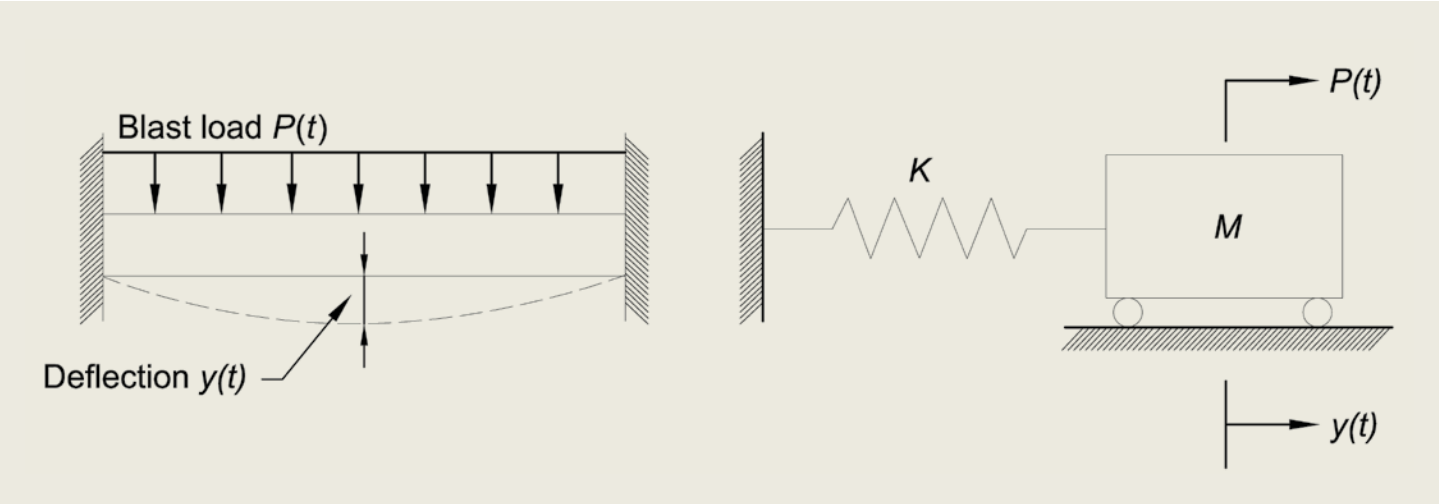

Figure A.4. Equivalent spring-mass system representing dynamic response of beam loaded by blast. Note: K = equivalent stiffness of spring-mass system; M = mass of blast-loaded component.

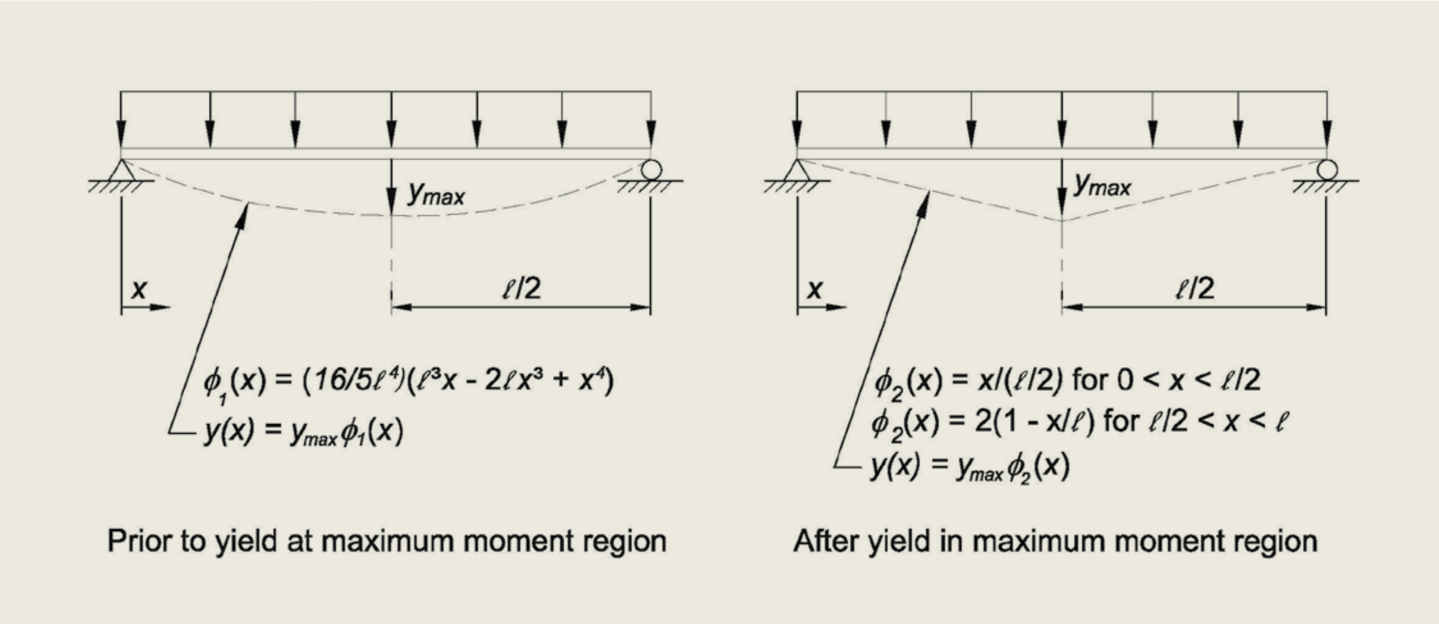

Figure A.5. Deflected shape functions for simply supported beam. Note: ℓ = clear span of component; x = location along component span; y_{max} = maximum deflection of component; $\phi_1(x)$ = shape function of simply supported beam before yielding in maximum moment region; $\phi_2(x)$ = shape function of simply supported beam after yielding in maximum moment region.

Table A.2. Load-mass factors for one-way components with simple supports

Load distribution	Response range	Load-mass factor K_{LM}*
Single point load at midspan	Elastic	0.49
	Plastic	0.33
Two point loads at third points	Elastic	0.60
	Plastic	0.56
Uniformly distributed load	Elastic	0.78
	Plastic	0.66
* For component with uniformly distributed mass		

M = mass of blast-loaded component

$y''(t)$ = acceleration of SDOF system

t = time

$R(y(t))$ = resistance of component based on resistance-versus-deflection curve and deflection at each time

$y(t)$ = deflection of SDOF system

$F(t)$ = blast load on component

Detailed information for calculating the terms in Eq. (A.3) is available in multiple references.[1,5,8,12] These terms are also calculated in example A.1. The equation of motion for the equivalent SDOF system in Eq. (A.3) can be solved in a time-step fashion using any number of available numerical integration techniques to determine the deflection history of the component caused by the blast load. *Introduction to Structural Dynamics*[11] provides background information on time stepping techniques. The load-mass factor is determined at each time step based on the assumed deflected shape, taking into account any yielding that has occurred. The resistance is also determined at each time step based on the deflection and the resistance-deflection relationship of the spring in the equivalent SDOF system section. The component must be designed so that the maximum calculated dynamic deflection of its equivalent SDOF system is within acceptable limits. These limits and the resistance-deflection relationship are described in the following sections. Simplified maximum-response charts have also been created showing the maximum response of the equivalent SDOF system plotted against key parameters from Eq. (A.3) for a blast load with the simplified shape in Fig. A.1.[5,8,11]

A.5.2 Component resistance–deflection function

The resistance at each time in Eq. (A.3) is based on the deflection at the previous time step and the component resistance–versus–deflection relationship. The resistance-deflection curve relates the resisted load to the midspan deflection of the blast-loaded component. The resisted load has the same spatial distribution as the applied blast load (typically a uniformly distributed pressure load). The component resistance is always equal and opposite to a statically applied load based on equilibrium requirements, but a dynamic load causes an additional inertial force so that the resistance generally does not equal the applied load (Eq. [A.3]). The resistance-deflection curve for a structural component can be derived with conventional static calculation methods using applicable dynamic material yield strengths. This is illustrated in example A.1. In a ductile reinforced-concrete component, the resisted load, or resistance, increases approximately linearly with deflection until yielding of the reinforcing steel in the maximum moment regions. Then the resistance remains relatively constant with increasing deflection until failure.

Figure A.6 shows the resistance-deflection curve for a non–load bearing precast concrete component with simple supports. The initial slope is the elastic flexural stiffness of the component k_e using an effective moment of inertia. Typically, for prestressed and nonprestressed concrete components the effective inertia is computed using an average of the gross moment of inertia and the fully cracked moment of inertia. The moment of inertia of a cracked prestressed or nonprestressed section can be approximated using Eq. (A.4).

$$I_{cr} = \left(n_p A_{ps} d_p^2 + n_s A_s d^2\right)\left(1 - 1.6\sqrt{n_p \rho_p + n_s \rho}\right) \quad \text{(A.4)}$$

where

I_{cr} = moment of inertia of cracked section (in.^{4})

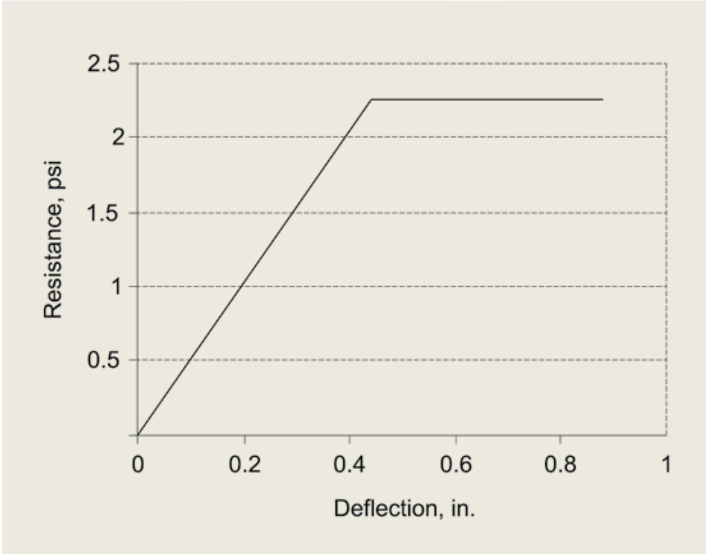

Figure A.6. Resistance-deflection relationship for simply supported precast concrete panel.

n_p = moduli ratio of prestressing steel to concrete

A_{ps} = area of prestressing steel in flexural tension zone (in.^{2})

d_p = distance from extreme compression fiber to centroid of prestressing steel (in.)

n_s = moduli ratio of nonprestressed steel to concrete

A_s = area of nonprestressed longitudinal tension reinforcement (in.^{2})

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement (in.)

ρ_p = ratio of A_{ps} to bd_p

ρ = ratio of A_s to bd

b = width of compression face of member (in.)

The stiffness goes to zero at initial yield deflection of the system y_e, when yielding occurs at the maximum moment region. The resistance does not degrade after yielding in Fig. A.6, implying ductile response. It is assumed that the stress in the reinforcing steel remains constant at f_{dy} after yielding, ignoring the small amount of strain hardening that occurs, so that the resisting moment and resistance of the component remain constant with increasing midspan deflection out to a limit deflection y_{max}.

Yield-line theory is used to determine the ultimate resistance for indeterminate components such as multispan continuous walls and beams. These components have multiple yield loads, where the reinforcing steel yields at different maximum moment regions as the resisted load increases, causing multiple nonzero slopes in the resistance-deflection curve (**Fig. A.7**). The resistance-deflection relationship of an indeterminate component can be simplified (Fig. A.7) using an equivalent stiffness k_E and yield equivalent deflection

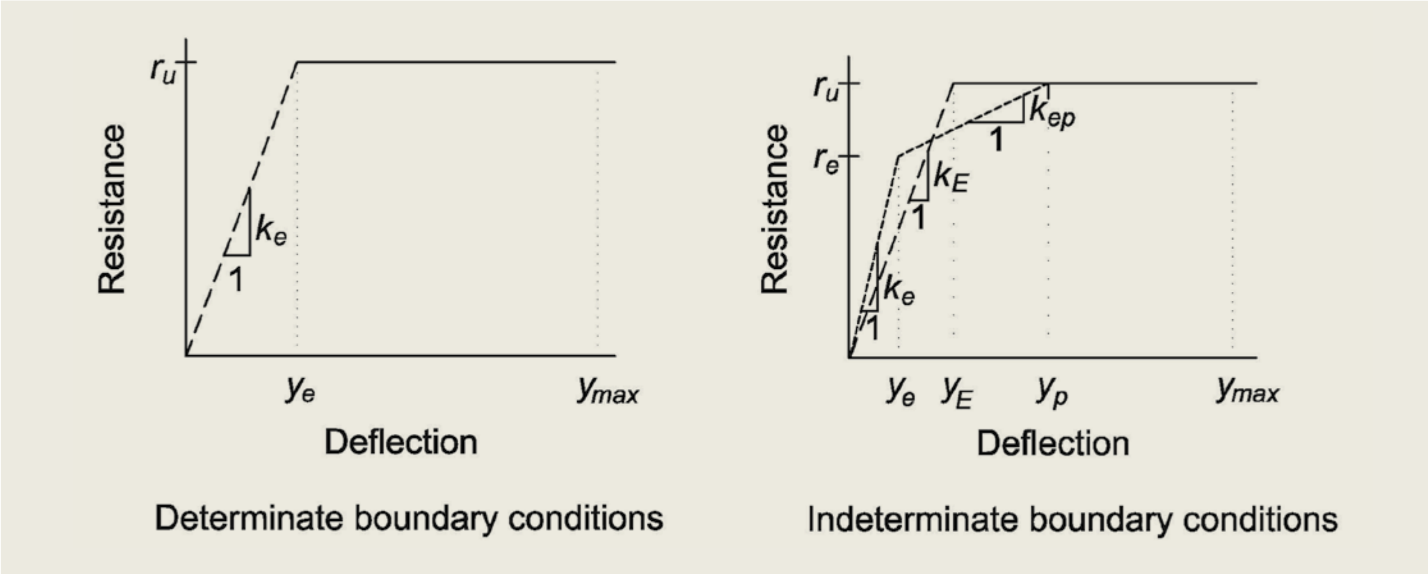

Figure A.7. Resistance-deflection relationships for determinate and indeterminate boundary conditions. Note: k_e = initial elastic stiffness of component; k_E = equivalent stiffness for indeterminate component; k_{ep} = secondary stiffness of indeterminate component; r_e = resistance at first yield of indeterminate blast-loaded component; r_u = ultimate resistance of blast-loaded component; y_e = initial yield deflection of system; y_E = equivalent yield deflection for indeterminate system; y_{max} = maximum deflection of component; y_p = deflection where the indeterminate component becomes a mechanism.

y_E that cause the simplified resistance-deflection curve, with one nonzero slope, to have the same area under its curve as the actual resistance-deflection curve out to the deflection where the component becomes a mechanism y_p. Detailed procedures for determining the resistance-deflection relationships of components are described by ASCE[8] and Departments of the Army, Navy, and Air Force.[5]

The component absorbs strain energy during elastic and plastic response that must equal the energy imparted by the blast load or the component will fail. The absorbed strain energy can be measured as the area under the component resistance–deflection curve out to the maximum calculated component deflection, which must be less than the maximum allowable design deflection. Strain energy must be absorbed with either high resistance (that is, high component moment capacity) or a large component-deflection capacity or by some combination of these two factors. Because the required shear strength and connection capacity for a blast-resistant component is often proportional to the maximum component resistance and not to the maximum deflection, a ductile component with a large deflection capacity is more desirable where possible.

Typically, a well-designed ductile component will absorb most of the blast-load energy with plastic strain energy and will have a maximum deflection below the deflection corresponding to failure. Prestressed concrete components have lower ductility than nonprestressed concrete components. This is because prestressing strands have a lower failure strain limit than nonprestressed reinforcing steel. For example, ASTM A706[13] bars have a strain limit of 10% to 12%, while ASTM A416[14] prestressing steel is required to have a strain capacity of only 3.5%. Existing blast design criteria are based on the assumption that prestressed, precast concrete components are significantly less ductile than conventionally reinforced components, which causes prestressed concrete components to have a much higher required design strength, and correspondingly higher connection loads, to resist a given blast load than a similar nonprestressed concrete component. However, recent blast testing has shown that prestressed concrete wall components with a low prestressing index (that is, 0.03) perform in a significantly more ductile manner than previously thought.[15] Additional blast testing on precast, prestressed concrete components is required to define the available ductility of these components more accurately.

The blast resistance–deflection relationship is altered by the presence of axial load on the component. For load-bearing components, the resistance is affected by secondary moments from the axial load as the panel deflects. There is ongoing research on this topic sponsored by PCI. The SBEDS (single-degree-or-freedom blast effects design spreadsheets) methodology manual[16] describes the use of an equivalent lateral load method in an SDOF analysis to account for secondary moments on components.

A.5.3 Deflection limits

Blast design involves allowing component stresses to exceed yield; therefore, allowable design stresses are not used in blast design. Instead, allowable design limits are set on component deflections that generally include a controlled amount of plastic deflection after the component yields at all maximum moment regions and becomes a mechanism. Allowable maximum dynamic deflections for components subject to blast loads are typically specified in terms of two parameters: the support rotation θ (**Fig. A.8**) and ductility ratio μ (Eq. [A.5]). The ductility ratio μ is the ratio of maximum deflection under the applied blast load to the yield deflection. The value y_E in Fig. A.7 is typically used as the yield deflection for indeterminate components. The support rotation measures the approximate rotation at the supports and essentially relates the maximum deflection to the span of the component for the small angles that are allowed for blast design.

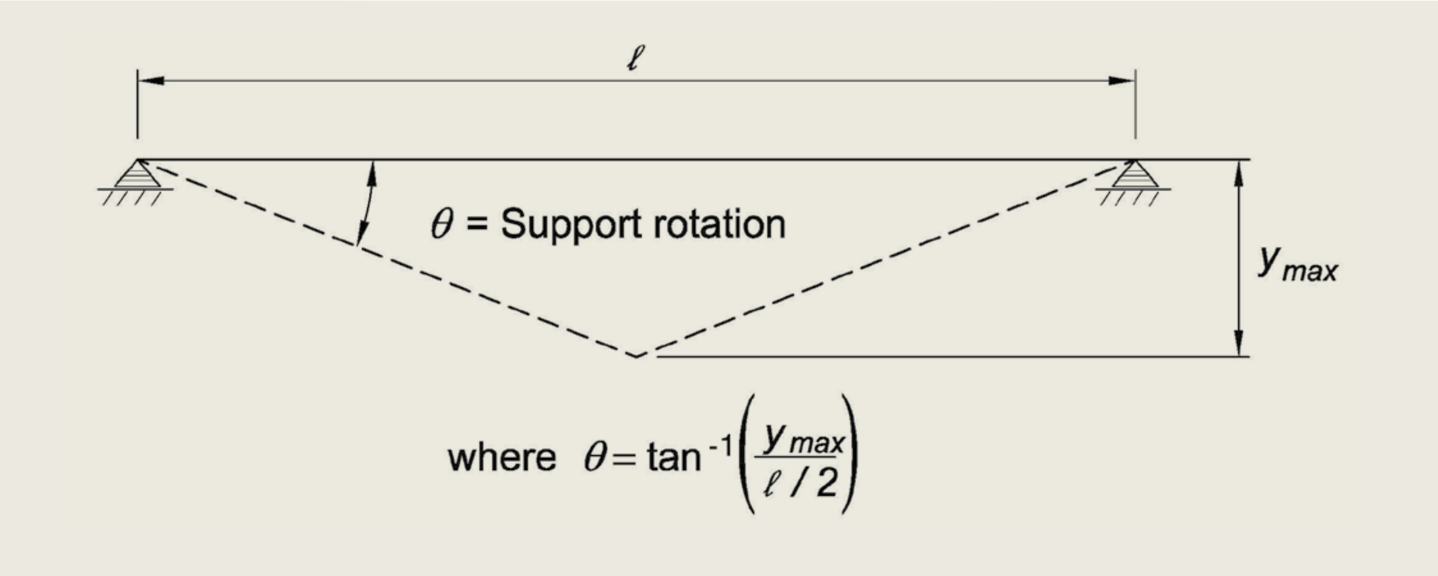

Figure A.8. Component support rotation. Note: ℓ = clear span of component; y_{max} = maximum deflection of component.

$$\mu = y_{max}/y_{yield} \quad \text{(A.5)}$$

where

μ = ductility ratio

y_{max} = maximum component deflection

y_{yield} = deflection causing yield of determinate component y_e or yield of equivalent elastic slope of indeterminate component y_E (Fig. A.7)

Maximum support rotation and ductility ratio values allowed for design are based on guidelines established by government and industry committees that have available blast data showing how these parameters correspond to observed component damage and the desired level of protection for the building. Depending on the protection required and the capability of the blast-resistant component, the maximum component deflection may be limited to the yield deflection or to a larger deflection. Often, the allowable support rotation and ductility ratio are limited to no more than one-half of the values corresponding to failure for design, implying a safety factor against failure of at least 2.0.

The available response criteria generally correlate damage levels with reduced maximum response levels for load-bearing components with significant axial load (for example, more than 20% of the component axial load capacity) compared with non–load bearing components. This is due to the concern that the blast capacity of load bearing components is more likely to be controlled by a nonductile mechanism (for example, flexure controlled by compression rather than tension).

Based on testing and analysis, the support rotation correlates better with failure for nonprestressed concrete, while the ductility ratio correlates better with failure for prestressed concrete. Allowable support rotations for blast design of nonprestressed concrete components are defined by the project requirements; however, for some cases this can range from 2 to 4 degrees. This corresponds to an allowable maximum dynamic deflection between 2.5 in. and 5.0 in. for a 12 ft span. Allowable ductility ratios for blast design of prestressed concrete components are typically in the range of 1 to 3. For a 40 ft, prestressed concrete T-beam, this can correspond to an allowable maximum dynamic deflection between approximately 1.3 in. and 4 in., though this varies depending on the yield deflection.

Example A.1. Blast design example

Given The non–load bearing precast concrete wall panel (**Fig. A.9**) will be designed to resist a given blast load. The 4 ft width of the panel that spans vertically is assumed to carry the blast load from the full 8 ft wide panel, where the opening is covered with a blast-resistant window that transfers the full blast load into the panel on either side. The wall panel is 6 in. thick and reinforced with no. 4 Grade 60 vertical reinforcing bars at 12 in. on center located 1 in. from each face. The design blast pressure will be assumed to have the shape in Fig. A.1 with a peak pressure of 20.2 psi and impulse of 85 psi-msec. The minimum specified compression strength for the concrete is 5000 psi.

Problem Calculate the mass, spring stiffness, ultimate resistance, and resistance-versus-deflection relationship for the equivalent SDOF system in Eq. (A.3) representing the wall panel. Solve the equation of motion for the equivalent SDOF system to determine the deflection history and resistance history for the panel. Determine whether the panel response will satisfy a design requirement that the maximum support rotation must not exceed 3 degrees.

Solution Calculate the mass per unit of blast-loaded area of the SDOF system m based on concrete unit weight w_c, the thickness h, and the acceleration of gravity g. Conservatively, only include the concrete that spans across the fully loaded span and ignore

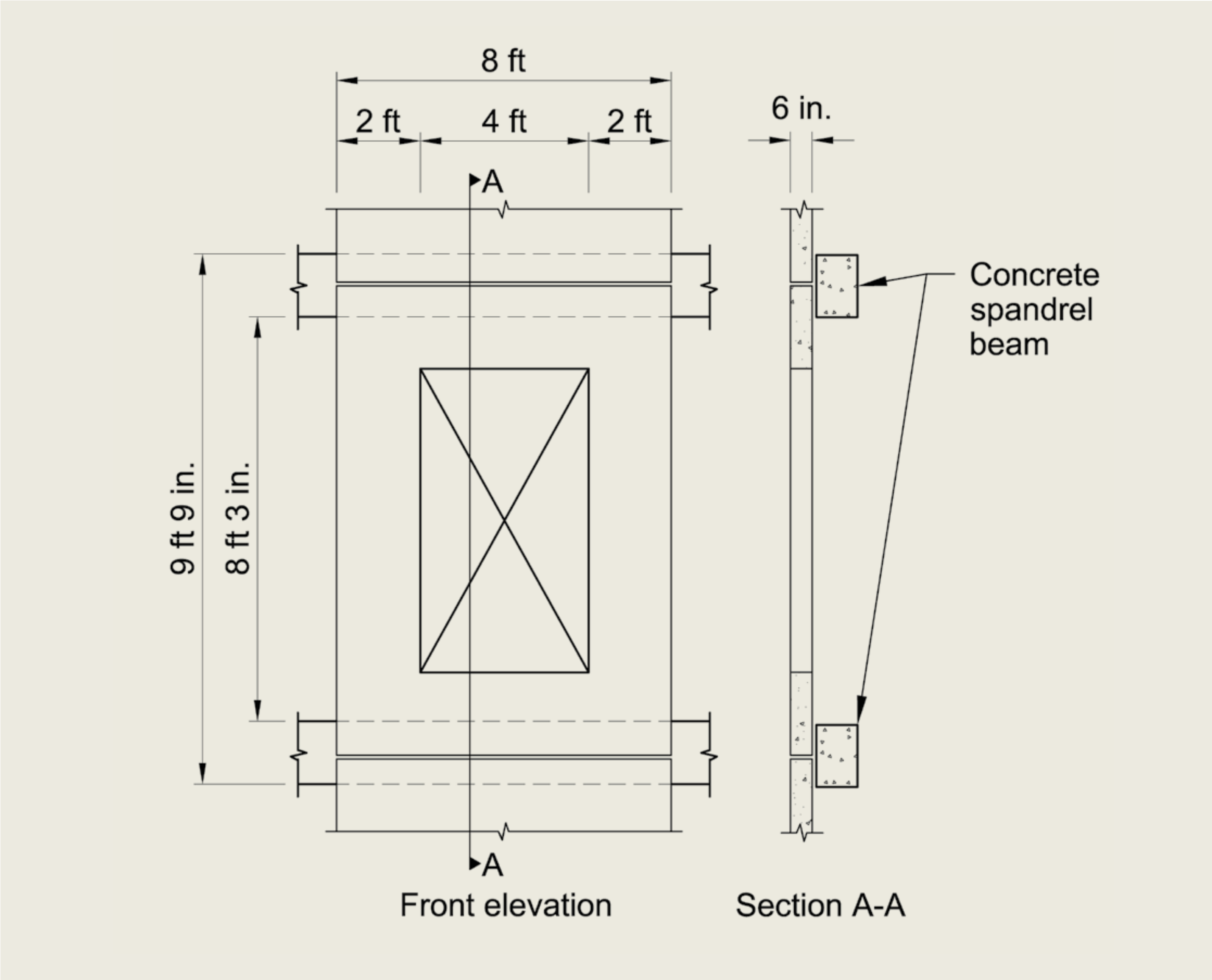

Figure A.9. Wall panel designed to resist blast load.

the weight of the window in the opening. The load-mass factors that are generally available, including those in **Table A.2**, are only applicable for mass that is uniformly distributed over the span length unless otherwise stated.

$$m = \frac{w_c t}{g}\left(\frac{\text{concrete width}}{\text{wall width}}\right) = \left[\frac{\left(150\times\frac{1}{1728}\right)(6.0)}{386\times10^{-6}}\right]\left(\frac{48.0}{96.0}\right)$$

$= 675$ psi-msec^2/in.

Calculate the dynamic moment capacity per unit width M_{du} based on the panel properties and the dynamic yield strengths of the reinforcing steel and concrete from Eq. (A.1) and (A.2). The formula for M_{du} is the same as for nominal flexural strength at section M_n in chapter 5 of this handbook using dynamic material strengths. This is also true for prestressed concrete.

For this example the contribution of the reinforcement located near the compression face is not included. Because the reinforcement is close to the neutral axis, its contribution is minimal and the resulting moment capacity provides a conservative estimate of the panel deformation.

$$f_{dy} = f_y K_e DIF = (60{,}000)(1.10)(1.17) = 77{,}200 \text{ psi}$$

$$f'_{dc} = f'_c\,(DIF) = (5000)(1.19) = 5950 \text{ psi}$$

$$M_{du} = \frac{A_s f_{dy}}{B}\left(d - \frac{A_s f_{dy}}{1.7 b f'_{dc}}\right)$$

$$= \left[\frac{(0.80)(77{,}200)}{96}\right]\left[5.0 - \frac{(0.80)(77{,}200)}{(1.7)(48)(5950)}\right]$$

$$= 3135 \text{ lb-in./in.}$$

where

A_s = area of nonprestressed longitudinal tension reinforcement

f_{dy} = reinforcing steel dynamic yield stress

B = width of loaded component

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

b = width of compression face of member

f'_{dc} = dynamic concrete compressive strength

Calculate the ultimate resistance of blast-loaded component r_u causing the spring to yield. The ultimate resistance of blast-loaded component r_u is equal to the uniform pressure load causing the applied moment in the example panel to equal M_{du}.

$$r_u = \frac{8M_{du}}{\ell^2} = \frac{8(3135)}{99^2} = 2.56 \text{ psi}$$

Calculate the SDOF system spring stiffness per unit width of blast load area k_e during elastic response using an average of the gross I_g and cracked moment of inertia I_{cr}.

Only the 4 ft width of the panel that is continuous over the span length provides stiffness.

Elastic concrete modulus E_c

$$E_c = 33w_c^{1.5}\sqrt{f'_c} = 33(150)^{1.5}\sqrt{5000} = 4287 \text{ ksi}$$

Gross moment of inertia I_g

$$I_g = \frac{1}{12}bh^3 = \frac{1}{12}(48)(6)^3 = 864 \text{ in.}^4$$

Nonprestressed reinforcement modular ratio n_s

$n_s = E_s/E_c = (29{,}000)/(4287) = 6.77$

where

E_s = elastic steel modulus

Cracked moment of inertia I_{cr}

$$I_{cr} = \left(n_p A_{ps} d_p^2 + n_s A_s d^2\right)\left(1 - 1.6\sqrt{n_p\rho_p + n_s\rho}\right)$$

$$= \left[0 + (6.77)(0.80)(5.0)^2\right]\left[1 - 1.6\sqrt{0 + (6.77)(0.00333)}\right]$$

$$= 102.8 \text{ in.}^4$$

Average moment of inertia I_{avg}

$I_{avg} = (I_g + I_{cr})/2 = (864 + 108.2)/2 = 483.4 \text{ in.}^4$

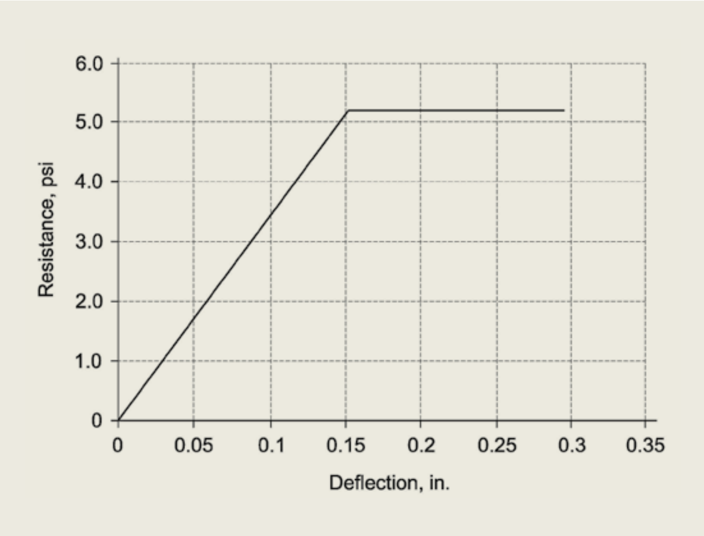

Figure A.10. Resistance-deflection curve for example panel.

Initial elastic stiffness of component k_e

$$k_e = \frac{384E_c I_{avg}}{5B\ell^4} = \frac{384(4{,}287{,}000)(483.4)}{5(96)(99)^4} = 17.26 \text{ psi/in.}$$

Determine the resistance versus deflection relationship for the spring of the equivalent SDOF system representing the panel (**Fig. A.10**). The resistance versus deflection relationship is assumed to be the same in rebound due to the symmetric reinforcement used in the panel. The deflection of the panel at yield is computed.

$y_e = r_u/k_e = (2.56)/(17.26) = 0.148$ in.

Calculate the natural period for this panel T_n based on the initial elastic stiffness k_e, mass m, and load-mass factor for elastic response K_{LM}. The natural period T_n and the equivalent triangular duration of the positive phase of the blast pressure t_d are used to determine the time step for the time-stepping numerical solution to the equation of motion. Generally, the time step is no larger than 10% of T_n or t_d, whichever is smaller.

$$T_n = 2\pi\sqrt{\frac{K_{LM}m}{k_e}} = 2\pi\sqrt{\frac{0.78(675)}{17.26}} = 35 \text{ msec}$$

Use a time-stepping numerical solution to the equation of motion (for example, Cramsey and Naito[10] or U.S. Army Corps of Engineers[16]) for the equivalent SDOF system representing the panel to the blast load based on the SDOF spring and mass properties calculated previously. The elastic and plastic load-mass factors are 0.78 and 0.66, respectively, based on **Table A.2** and the fact that the panel has simple supports and a uniformly applied blast load. **Figure A.11** shows the calculated deflection history of the panel.

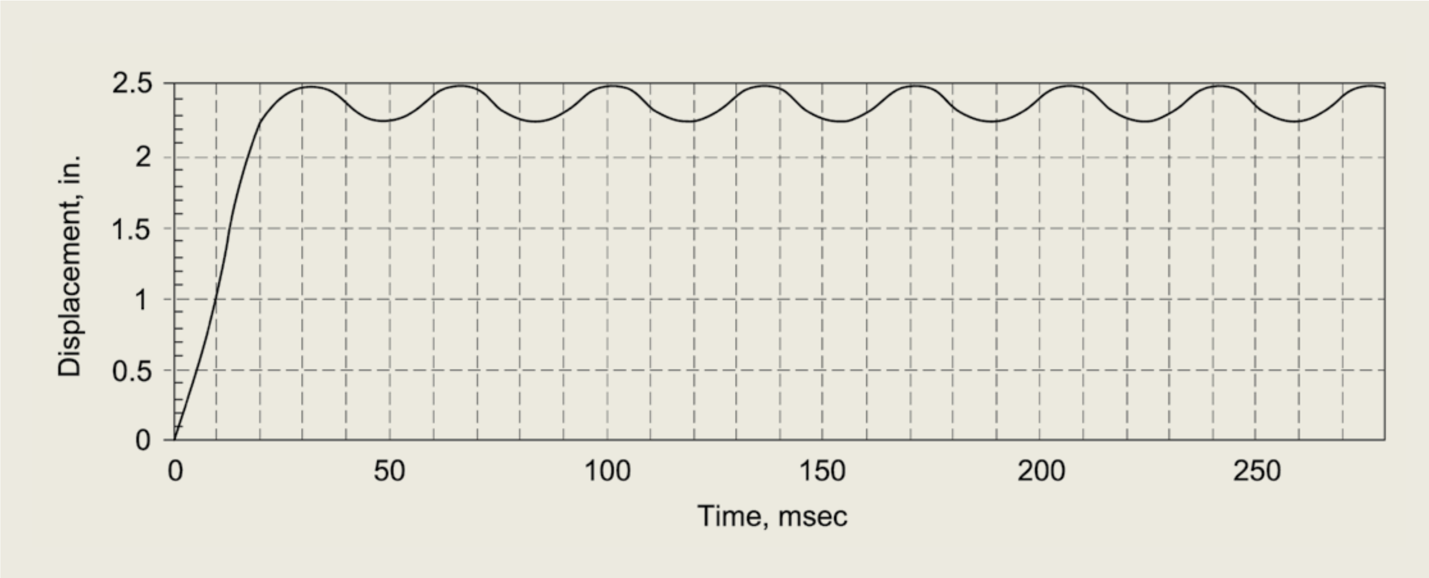

Figure A.11. Calculated deflection history for example panel.

Calculate the maximum support rotation θ based on the maximum component dynamic deflection y_{max} from the deflection history (**Figure A.11**) and the span length. As illustrated, the maximum deflection from the SDOF analysis is 2.47 in.

$$\theta = \tan^{-1}\frac{2y_{max}}{\ell} = \tan^{-1}\frac{2(2.47)}{99} = 2.86$$

The resulting maximum support rotation under the blast load of 2.86 is less than the support rotation limit of 3.00 degrees, and the design is acceptable.

A.6 Connection demands due to blast loads

In almost all cases, the allowable deflection limits are based on an assumed ductile flexural response for the component. Inherent in this requirement is that the shear capacity of the component and connections to the structure exceed the reaction forces from the dynamic flexural load capacity. A conservative approach is to ensure that the shear capacity and connection capacity be designed to exceed the ultimate resistance of the component based on flexural capacity. This can be achieved if the component shear and connection capacities exceed the design reaction loads from a static load equal to the ultimate resistance of the component (2.56 psi in Fig. A.10). In most cases this procedure results in required shear and connection capacities higher than those required for conventional loads. The required shear capacity may necessitate shear reinforcement, though this is typically not the case for precast, prestressed concrete slabs and walls. Some blast design specifications state that shear strength of the component must exceed its design reaction load by 20%; otherwise a prescribed minimum amount of shear reinforcement must be provided.

Typically, the component shear capacity is based on the static shear strength with no dynamic impact factor and no strength-reduction factor. This is a conservative assumption because there is some increase in the dynamic factor for shear.[4] The capacity of connections for blast-loaded components is generally based on load- and resistance-factor design (LRFD), including a small *DIF* for the connection material strength, on the order of 1.05 because connections typically have high-strength steel and the *DIF* decreases with increasing steel strength.[5,8] However, this *DIF* is often neglected. A strength reduction factor as required by LRFD is typically used for the connection design. The connection capacity for blast loading can also be based on 1.7 times the allowable connection capacity from allowable strength design, which should be nearly equivalent to the previously discussed load-resistant factor force-based design.

These requirements for shear and connection capacity imply that the flexural resistance of blast-loaded components should not be overdesigned such that the required shear and connection capacities are expensive or difficult to construct. The more the component relies on ductility or deflection past yield, within the allowable deflection limits to develop the required strain energy capacity to resist the blast load, the more economical and constructible the resulting design will generally be. In cases where the required shear or connection capacity cannot be provided, the ultimate resistance of the component must be based on the lower maximum applied load that can be resisted by the shear strength of the component or the connection. For this condition, a significantly lower maximum dynamic deflection is allowed based on assumed brittle failure of components with ultimate resistance controlled by their shear strength (for example, a maximum ductility ratio of 1.0).

Structural components initially respond inward (into the building) from the positive-phase blast load and then rebound outward, in a similar manner to a spring rebounding from a short-duration applied load. The negative phase of the blast load can accentuate the rebound deflection if it is in phase with the response of the component. During rebound, there is stress reversal at all maximum stress regions of the component. All compressive stress areas during inbound response become tensile stress regions during rebound and require sufficient reinforcing steel to prevent rebound failure. Also, the connections must have the capacity to resist reaction forces from component rebound response. Blast tests conducted on nonstructural prestressed and nonprestressed, precast concrete cladding panels have shown that rebound reaction forces can range from 20% to over 150% of the inbound value.[15] For design purposes, the upper limit on the rebound design reaction force is typically 100% of the peak inbound reaction force, though higher values are possible if the negative phase blast load is in phase with the rebound response of the component. Panels that responded elastically on inbound were found to have the highest rebound forces, while panels that had significant yielding on inbound had significantly lower rebound reaction loads. The localized resistance of the component in which the connection is embedded should also be carefully considered in the design. For example, intermediate wall panel connections on multispan panels may be subjected to high compression loads on inbound response. Appropriate detailing should be included to prevent punching shear failure of the connection into the panel. On rebound, an embedded connector in a multispan panel may be located in the tension zone of the component due to its flexural response. This region will be cracked and will need to be detailed to maintain the required tension force needed of the connection. Proper connection design should consider these effects. In addition to strength requirements, the designer should ensure that the selected connection type is able to accommodate the plastic deformations calculated for the component without failing.

It is typical for blast design to provide equal reinforcing steel (symmetrical reinforcing at each face) and connection capacity for inbound and rebound responses. Testing shows that components with reinforcement at each face are significantly more ductile than components with one layer of reinforcement. A single layer of reinforcement at midthickness is acceptable for components that are too thin for multiple layers of reinforcement. In all cases, there must be adequate reinforcement to resist both inbound and rebound responses of blast-loaded components. This must be determined based on a dynamic analysis of component response to the blast load that models both inbound and rebound component responses with a resistance-deflection relationship that has differing ultimate resistances for inbound and rebound responses based on the corresponding dynamic moment capacities.

An approach for connection design is to determine the fully factored design load for LRFD-based design of connections based on the lesser of the maximum dynamic resistance R_{max} and the ultimate strength of the panel r_u. This load is affected by an overstrength factor to be applied to the ultimate resistance, to account for limited ductility response, unaccounted effects of negative phase loading, and potential overloads. This overstrength can vary from 1.0 to 2.0 based on how well the expected load is known. An overstrength factor greater than 1.0 will only affect the connection design load in Eq. (A.6) for the case of elastic response of the panel, when R_{max} is significantly less than r_u. The design of the connection can then be conducted using conventional LRFD approaches. This connection and shear load U are computed in accordance with Eq. (A.6).

$$U = [\min(\Omega_r R_{max},\ r_u)]C_s \ell \qquad \text{(A.6)}$$

where

U = fully factored connection load for LRFD-based design of connection per unit width of panel

Ω_r = overstrength factor on maximum resistance

R_{max} = maximum resistance from SDOF analysis of component = $\max(R(t))$

$R(t)$ = calculated resistance history of component

r_u = ultimate resistance of blast-loaded component

C_s = factor defining ratio of span used to calculate connection load. Use 0.5 for components with simple or fixed supports on each end or 1.0 for cantilever beams. Use structural engineering principles to determine factors for other boundary condition cases.

ℓ = clear span of component

The resistance of the panel or connection can be based on standard design practice approaches using LRFD concepts. Appropriate methods of ACI 318-11,[18] AISC specifications,[19] or the PCI connections manual[20] could be used. The connection design strength can be computed as the nominal capacity multiplied by the appropriate strength reduction factor ϕ.

Example A.2. Reaction design example

Given The wall system and demands were defined in example A.1. The project requirements for this example define that the wall connections must support the inbound dynamic loads generated by the blast demand. For this specific project, the rebound reaction loads are defined to be 50% of the inbound forces and an overstrength factor of 1.5 should be applied to the maximum resistance for calculating the design connection loads if these loads are not based on the ultimate resistance of the panel.

Problem Determine the support reaction loads.

Solution To determine the support loads, the ultimate resistance and the maximum resistance of the panel must be computed. The ultimate resistance r_u was computed in example A.1 to be 2.56 psi. The maximum resistance and the maximum dynamic reaction of the panel are determined from the SDOF analysis of the panel response. **Figure A.12** illustrates the calculated resistance of the panel as a function of time $R(t)$. These computations are part of a standard SDOF analysis and are described in detail in PCI's *Blast-Resistant Design Manual*.[1]

Based on the SDOF analysis, the component yields; therefore R_{max} is equal to the ultimate resistance of the panel, 2.56 psi. The resulting connection load can be calculated.

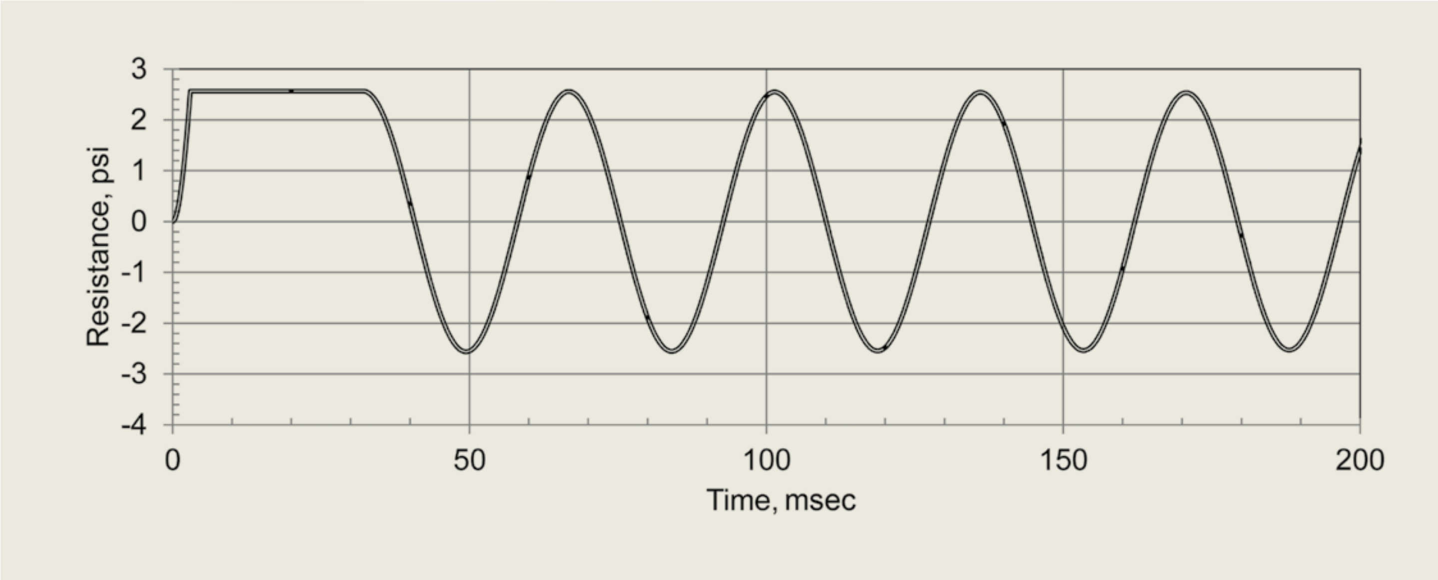

Figure A.12. Calculated resistance history for example panel.

$$U = \left[\min\left(\Omega_r R_{max}, r_u\right)\right]\frac{\ell}{2}$$

$$= \left\{\min\left[(1.5)(2.56), 2.56\right]\right\}\left(\frac{99}{2}\right) = 190 \text{ lb/in.}$$

For the 8 ft wide panel, the top and bottom would each need connectors capable of resisting 18.2 kip on inbound and 9.1 kip on rebound.

A.7 Energy methods to determine approximate displacement

Under specific conditions the response of the structural component can be approximated using energy methods. With this approach, the kinetic energy of the blast event is equated with the strain energy provided by the component as it deforms elastically or inelastically. Using this energy balance technique, an estimated deflection can be computed. This methodology is limited to cases where the blast pressure loads can be considered impulsive relative to the fundamental period of the structural component. Typically this is assumed to occur when the duration of the blast load is less than 10% of the fundamental period of the component. For these cases, this approach can be used to give an order-of-magnitude estimate of the response of the component.

Due to the approximate nature of this approach, the results should be used only as a guide and not for design/analysis. For design purposes, the SDOF evaluation methods of section A.5.1 should be conducted to provide an estimate of the expected response.

A.8 Special considerations for insulated non–load bearing wall panels

Insulated precast concrete wall panels provide a cost-effective, energy-efficient means of cladding building structures. Naito et al.[21] determined that non–load bearing insulated panels can provide resilience against blast loads. Design recommendations are provided for the blast-resistant design of these panels.

A.8.1 Flexural response

The panels should be designed to resist the blast loads through formation of a flexural mechanism. The resistance-deflection response should be determined using the recommendations of section A.5.2. Panels may be designed to be fully composite or treated as noncomposite. Partially composite panels are often used for traditional service load applications such as wind-resistant applications. However, the use of partially composite panels for blast applications is not recommended until proper tools can be developed for accurate determination of the maximum deformation response of these panels.

Response limits for prestressed and nonprestressed insulated concrete panels must follow the requirements of the project and are often defined by the owner. Resources on response limits are summarized in section A.5.3. Research has shown that response limits commonly used for solid prestressed and nonprestressed flexural components are appropriate for non–load bearing insulated wall panels.[21]

Sandwich panels can be designed to resist blast load with the SDOF analysis method described in section A.5. The flexural response determination requires the calculation of the elastic and inelastic properties. For these panels the elastic response may be based on the average of the gross and cracked moment of inertia, Eq. (A.4). For multiwythe insulated prestressed and nonprestressed concrete panels, the inertia is dependent on the level of composite action. For composite insulated panels, the moment of inertia should be computed based on the effective moment of inertia of the gross section. For noncomposite insulated wall panels, the moment of inertia can be assumed to be equal to the sum of the moment of inertia of each structural wythe.

To account for insulation in dynamic SDOF approaches, both the reduction in mass and stiffness due to the insulation must be accounted for. To accomplish this with analysis programs developed for solid concrete sections (that is, SBEDS),[16] appropriate adjustments are necessary. A common approach is to use a solid section and apply a negative mass to account for the foam insulation. For example, for a concrete panel with a unit weight of 150 lb/ft^3 and a 3 in. layer of insulation, a negative weight of 37.5 lb/ft^2 should be applied. To reduce the stiffness from that of a solid panel to that of a sandwich panel, an appropriate reduction in the moment of inertia or elastic modulus of concrete can be used. For example if a 9 in. solid wall is used to model a 3-3-3 insulated wall, a 4% reduction in the elastic modulus would be necessary to match the elastic stiffness. This reduction is equivalent to the decrease in gross moment of inertia from a 9 in. solid panel to the 3-3-3 insulated panel.

Flexural strength of fully composite panels can be based on standard flexural capacity approaches for prestressed and nonprestressed flexural concrete elements.[22] This approach is appropriate if the depth of the compression zone is contained entirely in the compression wythe. For noncomposite panels, the flexural strength of the individual solid concrete sections should be added. For panels with superficial exterior wythes, the flexural strength should be based on the interior section alone. For these sections, the mass of the exterior can be included in the evaluation. Flexural and shear strength can be computed in accordance with the approaches of the PCI sandwich panel design guide[22] using dynamic strengths for the concrete and reinforcing steel as stated in section A.4. For new construction, the ties between wythes should be designed to cause the panel to be fully composite for static response. Blast testing has demonstrated that this will cause the panels to respond in a fully composite manner under blast loading.

A.8.2 Composite action

Composite action in insulated panels is achieved through the use of solid zones or shear ties. For composite panel performance under blast loads, full composite action should be achieved. For blast applications, composite action for sandwich panels should be based on the ultimate flexural strength of the panel M_{du}. The ties or solid zones should be distributed in accordance with the corresponding lateral shear load. **Figure A.13** illustrates a single-span panel subject to a uniform pressure. The interface shear load V should be transferred over a length of interface shear transfer from maximum moment region to location of zero moment d_L. The value of the shear force should be based on the tension compression couple generated by the flexural strength. To achieve composite action on a simply supported panel, a greater amount of interface ties are required at the ends of the panel and none are required in the middle.

- Shear loads are computed based on the requirements of the project. Shear is often prescribed in one of two ways.

- The shear load should be based on the dynamic reactions developed in the component as a result of the prescribed dynamic blast pressures.

The shear load should be based on the ultimate flexural capacity of the panel.

The second approach is commonly used to determine shear requirements for the tie design. For short panels, however, this approach can result in an overly conservative design. For those cases, the first approach is often used. To provide some factor of safety against failure, an overstrength of 1.0 to 2.0 is applied to the first approach based on how well the expected load is known.

In the second approach, the shear loads must be computed from the dynamic flexural capacity of the panel. The shear loads are used to design both the component itself and, for precast concrete components, the connections to the structure. Connection demands must consider both inbound and rebound response. Inbound forces are computed as defined previously. Rebound design forces can vary from 100% to 0% of the inbound depending on the requirements of the project.

The shear strength of the panel should consider only the concrete contribution of the wythe that is in compression. This is a conservative approach in that it does not rely on the shear transfer between wythes. The concrete contribution to shear strength V_c may be calculated using Eq. (A.9). A less conservative approach sometimes used by designers computes the shear strength based on the full flexural depth of the panel minus the insulation thickness. This method is illustrated in Eq. (A.10). For noncomposite sections, the flexural depth of each wythe can be used for determination of the effective shear area.

$$V_c = 2\sqrt{f_c'}\,(b)(\text{compression wythe thickness}) \qquad \text{(A.9)}$$

$$V_c = 2\sqrt{f_c'}\,(b)(d - t_f) \qquad \text{(A.10)}$$

where

V_c = concrete contribution to shear strength

f_c' = specified concrete compressive strength

b = width of compression face of member

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

t_f = thickness of insulation

For noncomposite panels, ties between the layers are designed for erection and handling in accordance with PCI recommendations.[22] These panels are often detailed with a structural interior wythe and a nonstructural exterior wythe. For blast applications, noncomposite panel strength should be assessed by summation of the dynamic flexural resistance of each wythe. The mass should also account

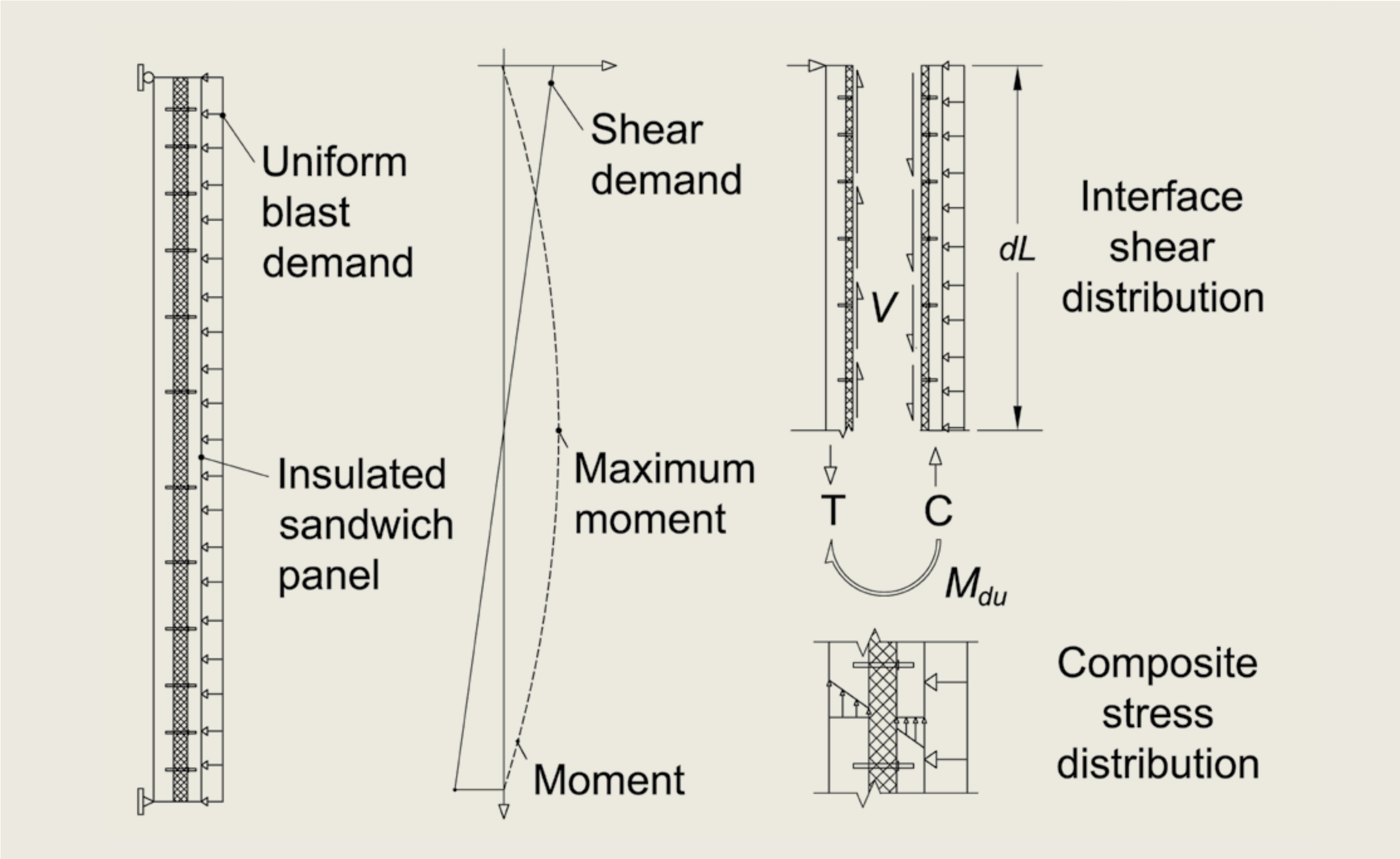

Figure A.13. Shear and moment distribution. Note: C = compression; d_L = length of interface shear transfer from maximum moment region to location of zero moment; M_{du} = dynamic flexural resistance of blast-loaded component per unit width; T = tension; V = interface shear load.

for the mass of the combined interior and exterior wythes. Research has shown that panels detailed as noncomposite achieve partial composite action.[21] To be conservative, the connection design of these panels should be based on the assumption of full composite flexural action in accordance with section A.6.

Example A.3. Sandwich panel resistance example

Given A fully composite insulated sandwich wall panel is to be used for a blast-resistant application. The panel is a non–load bearing multispan **(Fig. A.14)**. The panel is two stories tall with equal spans and is continuous over the intermediate floor. The panel has a 3 in. exterior wythe of concrete, a 4 in. insulation layer of expanded polystyrene, and a 3 in. interior wythe of concrete. The concrete has a density of 150 lb/ft^3 and has a compressive strength of 5000 psi. The panel is prestressed with four $^3/_8$ in. Grade 270 low-relaxation strands, with an initial prestress of 189 ksi, in the center of each wythe as illustrated in **Fig. A.15**. The wall is subjected to a reflected pressure-impulse load of 7 psi and 40 psi-msec (Fig. A.1). The panel is to be reinforced with semicontinuous transverse shear ties that provide a 200 lb/in. shear resistance or discrete ties with shear strength of 2.0 kip per tie.

Problem The wall has a response criterion of 2 degrees of support rotation. Does the panel exceed the response limit? Determine the amount and distribution of shear ties for the panel. Also determine the reaction forces for which the panel-to-structure connections should be designed. For this example, assume that the rebound reaction will be limited to 50% of the inbound reaction.

Solution Determine the mass of the panel per surface area.

$$m = \frac{(t_1 + t_2)w_c}{g} = \frac{(3.0+3.0)(0.0868)}{3.861\times10^{-4}} = 1349 \text{ psi-msec}^2\text{/in.}$$

where

m = mass per unit area of blast-loaded component

t_1 = thickness of interior wythe

t_2 = thickness of exterior wythe

w_c = unit weight (density) of concrete

g = acceleration of gravity

Determine the resistance deflection response of the wall panel.

Compute the dynamic flexural strength of the wall.

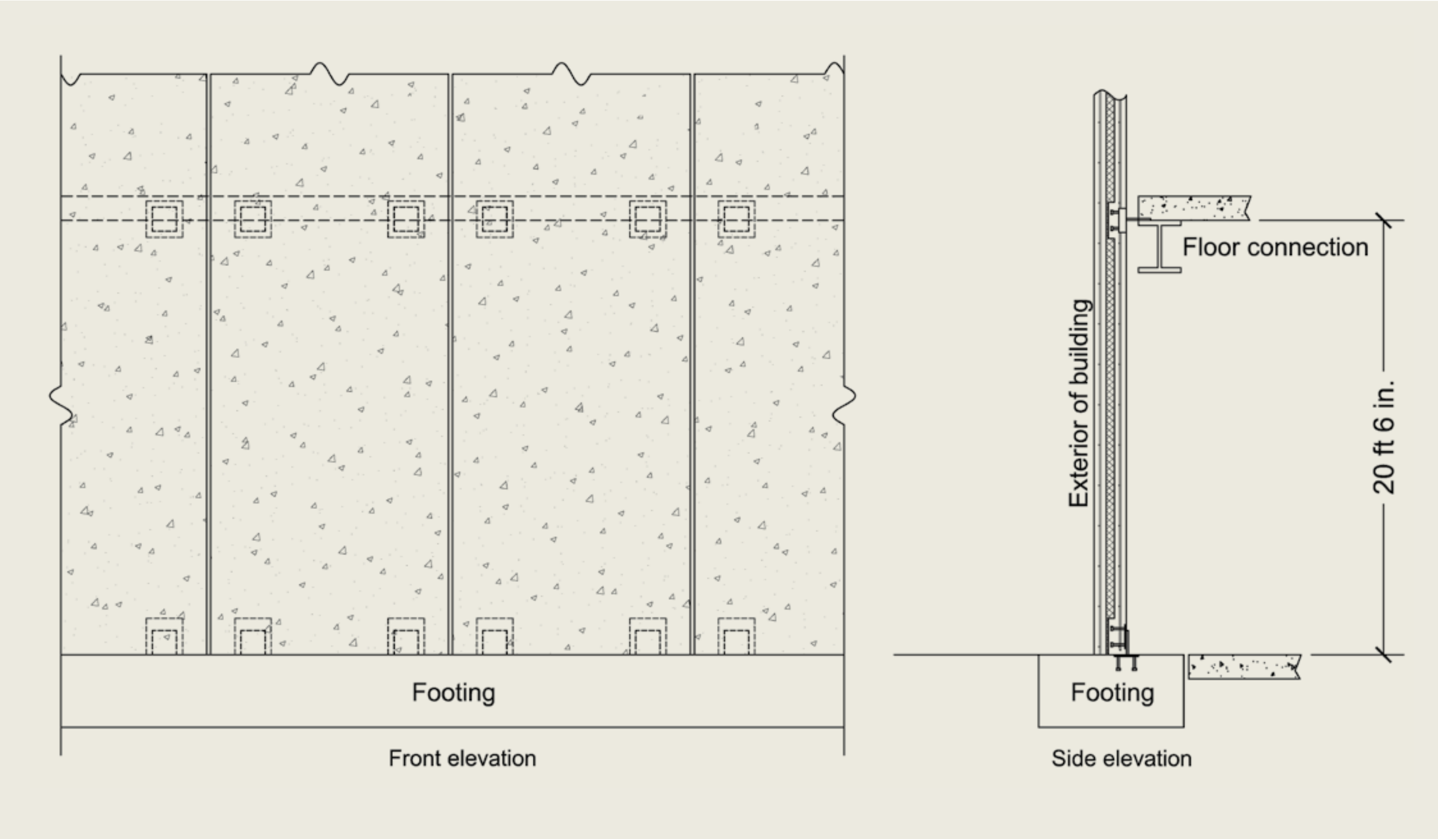

Figure A.14. Wall configuration.

Determine the dynamic material strengths.

$f_{dpu} = f_{pu}DIF$ = (270,000)(1.00) = 270,000 psi

where

f_{dpu} = dynamic tensile strength of prestressing steel

f_{pu} = specified tensile strength of prestressing steel

$f'_{dc} = f'_c\, DIF$ = (5000)(1.19) = 5950 psi

Determine the stress in the strands at the flexural strength of the panel. Neglect the compression steel.

$$f_{pe} = 0.80 f_{pi} = 0.80\left(\frac{16{,}100}{0.085}\right) = 151.5 \text{ ksi} = 0.56 f_{pu}$$

where

f_{ps} = stress in prestressing steel at nominal flexural strength

f_{pi} = ratio of initial prestress force to area of prestressing steel

f_{pu} = specified tensile strength of prestressing steel

Because $f_{pe} > 0.50 f_{pu}$ the simplified approach for f_{ps} can be used.

$$f_{ps} = f_{dpu}\left[1 - \frac{\gamma_p}{\beta_1}\left(\rho_p \frac{f_{dpu}}{f'_{dc}}\right)\right]$$

$$= 270{,}000\left\{1 - \frac{0.28}{0.80}\left[\frac{0.34}{(59)(8.5)}\left(\frac{270{,}000}{5950}\right)\right]\right\} = 267.1 \text{ ksi}$$

where

f_{dpu} = dynamic tensile strength of prestressing steel

γ_p = factor for type of prestressing steel (see ACI 318-11[18] section 18.7.2 for values)

β_1 = factor relating depth of equivalent rectangular compressive stress block to neutral axis depth (see ACI 318-11[18] section 10.2.7.3 for values)

ρ_p = ratio of A_{ps} to bd_p

f'_{dc} = dynamic compressive strength of concrete

A_{ps} = area of prestressing steel in flexural tension zone

b = width of compression face of member

d_p = distance from extreme compression fiber to centroid of prestressing steel

Depth of equivalent rectangular stress block a

$$a = \frac{A_{ps} f_{ps}}{0.85 f'_{dc} b} = \frac{\left[(4)(0.085)\right](267{,}100)}{0.85(5950)(59)} = 0.304 \text{ in.} < t_2$$

The compression block is within compression wythe.

The dynamic flexural capacity of the section can be found using standard flexural strength equations. The negative and positive flexural strength is equal due to the symmetric section. The compression steel is conservatively ignored.

$$M_{du} = \frac{A_{ps} f_{ps}}{b}\left(d_p - \frac{A_{ps} f_{ps}}{1.7 b f'_{dc}}\right)$$

$$= \left[\frac{(4\times 0.085)(267{,}100)}{59}\right]\left[8.5 - \frac{(4\times 0.085)(267{,}100)}{(1.7)(59)(5950)}\right]$$

$$= 12{,}849 \text{ lb-in./in.}$$

Based on the structural configuration of the wall, the component is assumed to have a simple boundary condition at the support and a fixed boundary condition at the intermediate floor connection. Fixity is assumed due to the continuity of the wall into the second floor of the building. The component forms a negative flexural mechanism at the fixed support followed by a positive flexural mechanism at midheight. The pressure-deflection response can be computed as follows:

Resistance at first mechanism

$$r_e = \frac{8 M_{du-}}{\ell^2} = \frac{8(12{,}849)}{246^2} = 1.70 \text{ psi}$$

where

r_e = resistance at first yield of indeterminate blast-loaded component

M_{du-} = dynamic negative flexural resistance of blast-loaded component per unit width

ℓ = clear span of component

Resistance at second mechanism

$$r_u = \frac{4(M_{du-} + 2M_{du+})}{\ell^2} = \frac{4\left[12{,}849 + (2)(12{,}849)\right]}{246^2} = 2.55 \text{ psi}$$

where

r_u = ultimate resistance of blast-loaded component

M_{du+} = dynamic positive flexural resistance of blast-loaded component per unit width

ℓ = clear span of component

Compute the average moment of inertia of the component.

Gross moment of inertia I_g

$$I_g = \frac{1}{12} b h_1^3 + b h_1 y_1^2 + \frac{1}{12} b h_2^3 + b h_2 y_2^2$$

where

b = width of compression face of member

h_1 = thickness of tension wythe of sandwich panel component

y_1 = distance from center of gravity of gross section to center of gravity of tension wythe

h_2 = thickness of compression wythe of sandwich panel component

y_2 = distance from center of gravity of gross section to center of gravity of compression wythe

$$I_g = \frac{1}{12}(59)(3)^3 + (59)(3)(3.5)^2$$

$$+ \frac{1}{12}(59)(3)^3 + (59)(3)(3.5)^2 = 4602 \text{ in.}^4$$

Compute transformed cracked moment of inertia; neglect welded wire reinforcement contribution for simplicity.

$$n_p = \frac{E_{ps}}{E_c} = \frac{28{,}500}{4287} = 6.65$$

Σ(area)(distance from cracked centroid) = 0

$(bc)(c/2) + (n_p A_{ps(top)})(c - d') + (n_p A_{ps(bot)})(c - d) = 0$

where

b = width of compression face of member

c = distance from compression face to centroid of cracked transformed section

n_p = moduli ratio of prestressing steel to concrete

$A_{ps(top)}$ = area of prestressing steel in tension wythe

d' = distance from extreme compression fiber to centroid of compression reinforcement

$A_{ps(bot)}$ = area of prestressing steel in compression wythe

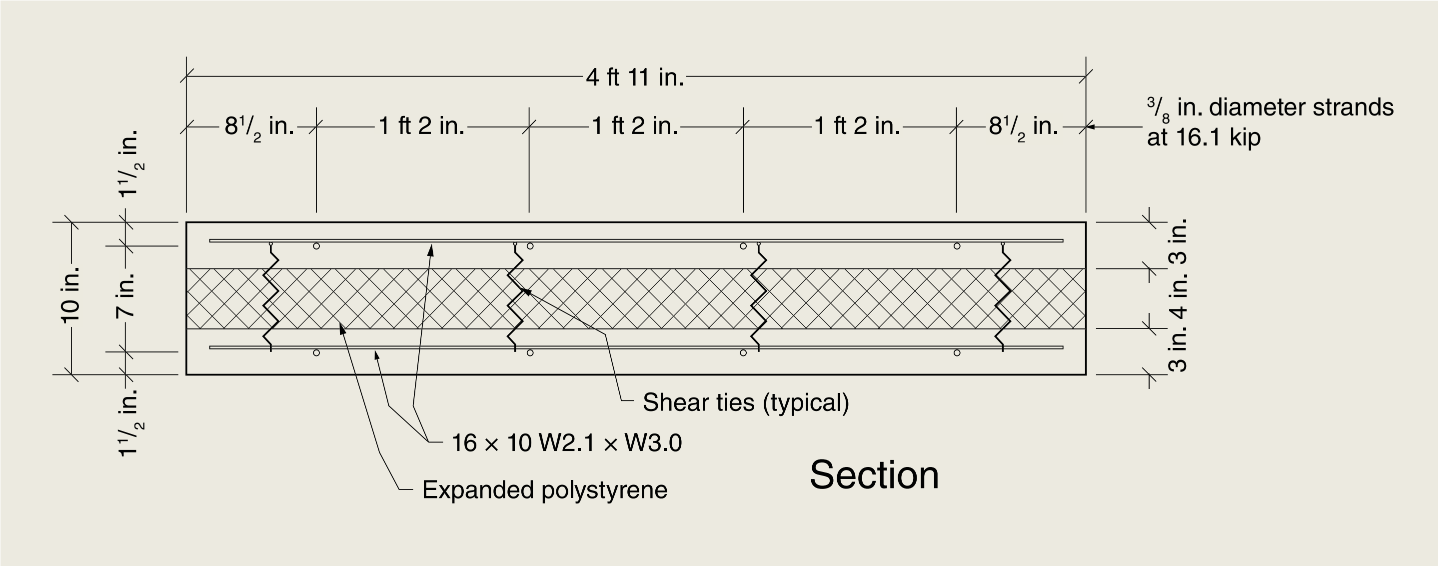

Figure A.15. Insulated sandwich wall cross section.

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

$(59c)(c/2) + [(6.65)(0.34)](c - 1.5) + [(6.65)(0.34)](c - 7.5) = 0$

Solving the equation, c = 0.80 in.

$$I_{cr} = \frac{1}{12}bc^3 + (bc)(c/2)^2 + \left(n_p A_{ps(top)}\right)(c - d')^2 + \left(n_p A_{ps(bot)}\right)(c - d)^2$$

$$I_{cr} = \frac{1}{12}(59)(0.80)^3 + (59)(0.80)(0.80/2)^2 + (6.65)(0.34)(0.80 - 1.5)^2 + (6.65)(0.34)(0.80 - 8.5)^2 = 145 \text{ in.}^4$$

$$I_{avg} = \frac{I_g + I_{cr}}{2} = \frac{4602 + 145}{2} = 2373 \text{ in.}^4$$

Compute the elastic stiffness of the wall.

Initial stiffness $k_e = \dfrac{185 E_c I_{avg}}{b\ell^4} = \dfrac{185(4,287,000)(2373)}{(59)(246)^4}$ = 8.71 psi/in.

Secondary stiffness $k_{ep} = \dfrac{384 E_c I_g}{5b\ell^4} = \dfrac{384(4,287,000)(2373)}{5(59)(246)^4}$ = 3.62 psi/in.

Compute the deflections at the limit states.

Initial yield displacement $y_e = r_e/k_e$ = 1.70/8.71 = 0.195 in.

Secondary yield displacement

$$y_p = y_e + \frac{r_u - r_e}{k_{ep}} = 0.195 + \frac{2.55 - 1.70}{3.62} = 0.430 \text{ in.}$$

Compute the equivalent stiffness and yield displacement.

Equivalent stiffness $k_E = \dfrac{r_u}{y_e + \frac{1}{3}y_p} = \dfrac{2.55}{0.195 + \frac{1}{3}(0.430)}$ = 7.53 psi/in.

Alternatively $k_E = \dfrac{160 E_c I_{avg}}{b\ell^4} = \dfrac{160(4,287,000)(2373)}{(59)(246)^4}$ = 7.53 psi/in.

Equivalent yield displacement $y_E = r_u/k_E$ = 2.55/7.53 = 0.338 in.

Figure A.7 illustrates the computed terms of the resistance deflection response of the indeterminate panel. **Figure A.16** illustrates the resulting SDOF response. The maximum deformation is 0.45 in.

$$\theta = \tan^{-1}\frac{2y_{max}}{\ell} = \tan^{-1}\frac{2(0.45)}{246} = 0.21$$

The panel meets the response limit with less than 2 degrees of support rotation.

Determine the amount and location of shear ties required to achieve composite action under the blast loads. The shear transfer force between the two wythes is equal to the magnitude of the tension or compression force at the dynamic moment capacity. The interface shear force V_{int} can be computed:

$V_{int} = A_{ps}f_{ps}$ = (0.34)(267,100) = 90.81 kip

Length of distributed shear ties required $L_{req} = \dfrac{90,810}{200}$ = 454.1 in.

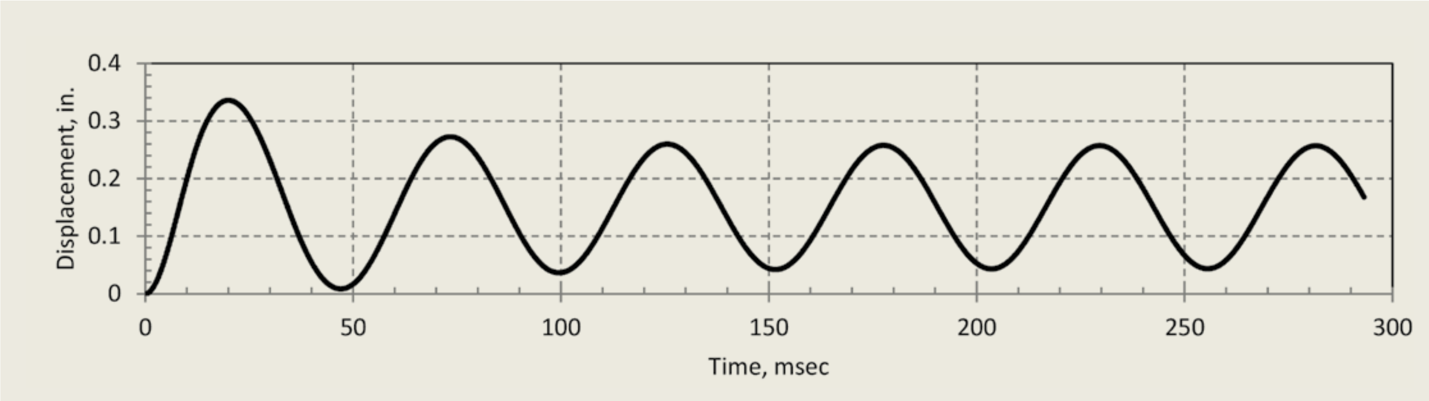

Figure A.16. Calculated deflection history for example sandwich panel.

Number of discrete ties required $N_{ties} = \dfrac{90,810}{2000} = 45.4$ (use 46 ties)

The maximum moment occurs when the second flexural hinge is formed. At this phase of response the fixed support can be equated to a hinge. Under this condition the panel is essentially simply supported. Consequently the required ties should be distributed over half the wall span. The distribution of ties can be uniform over the panel length or can be varied in accordance with a linearly varying shear load distribution. **Figure A.17** illustrates examples of a uniform discrete tie, uniform distributed tie, and varying discrete tie layout. The distribution shown is used to support the applied blast loads. Handling and erection loads may require an additional amount or distribution of ties.

Determine the transverse shear forces and the connection forces for the panel. Check the panel shear strength. For this example the connection and shear forces are computed from the ultimate flexural strength of the panel. When the ultimate strength of the panel is achieved, the component boundary conditions are simple-simple. The maximum shear load can be estimated from the ultimate resistance of the blast-loaded component r_u.

Shear load in the panel $V_u = r_u \dfrac{\ell}{2} b = (2.548)\left(\dfrac{246}{2}\right)(59) = 18.49$ kip

Shear capacity V_{cap} of the panel assuming only compression wythe active in shear resistance

$$V_{cap} = 2\sqrt{f_c'}bh_2 = 2\sqrt{5000}(59)(3) = 25.03 \text{ kip}$$

Shear capacity V_{cap} of the panel assuming full flexural depth is active

$$V_{cap} = 2\sqrt{f_c'}b(d - t_f) = 2\sqrt{5000}(59)(8.5 - 4.0) = 37.55 \text{ kip}$$

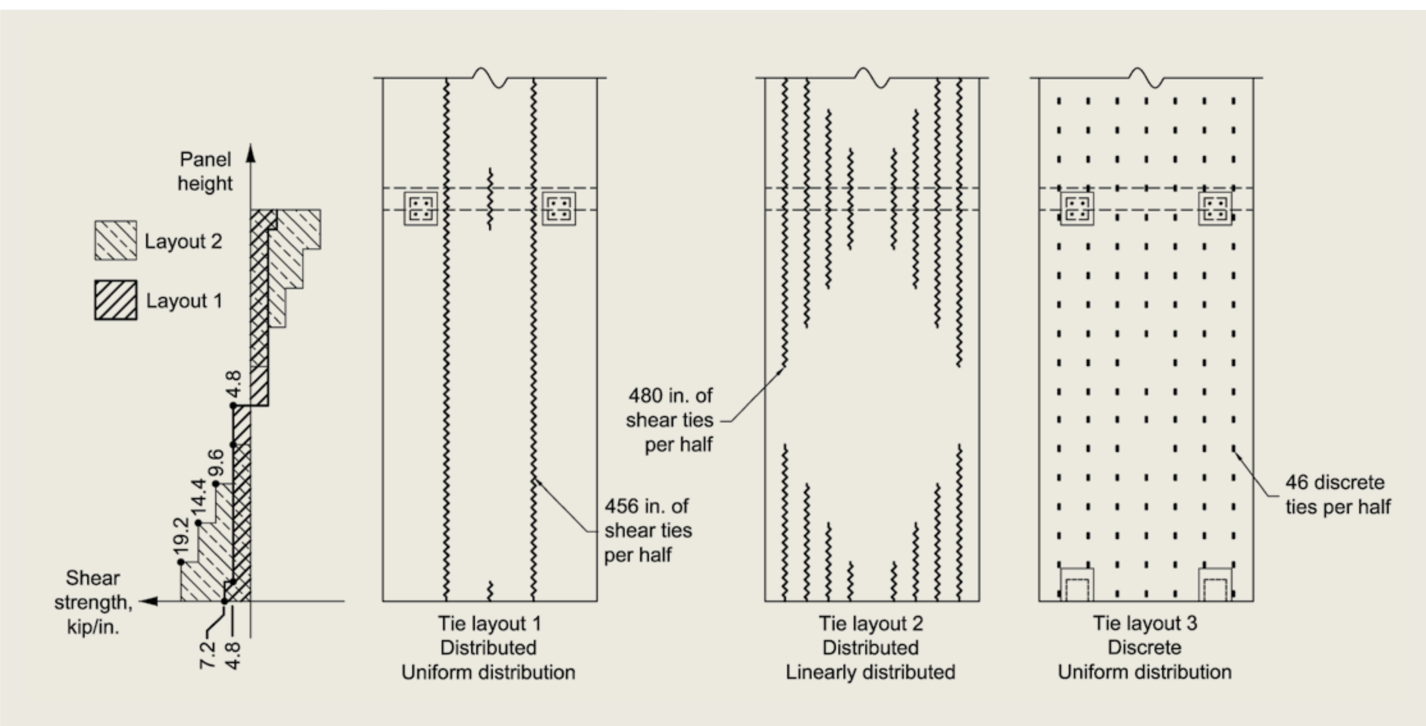

Figure A.17. Possible shear tie layouts.

Shear capacity of the panel is greater than the demand. The panel is safe against shear failure when the maximum flexural strength is achieved. The connections to the structure should be designed for the same load. Consequently, if two connections are used at the top of the panel each should resist 9.25 kip.

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A.10 Notation

a = depth of equivalent rectangular stress block

A_{ps} = area of prestressing steel in flexural tension zone

$A_{ps(bot)}$ = area of prestressing steel in compression wythe

$A_{ps(top)}$ = area of prestressing steel in tension wythe

A_s = area of nonprestressed longitudinal tension reinforcement

b = width of compression face of member

B = width of loaded component

c = distance from compression face to centroid of cracked transformed section

C_s = factor defining ratio of span used to calculate connection load

d = distance from extreme compression fiber to centroid of longitudinal tension reinforcement

d' = distance from extreme compression fiber to centroid of compression reinforcement

d_L = length of interface shear transfer from maximum moment region to location of zero moment

d_p = distance from extreme compression fiber to centroid of prestressing steel

DIF = dynamic increase factor

E_c = modulus of elasticity of concrete

E_s = modulus of elasticity of steel

$F(t)$ = blast load on component

f_c' = specified compressive strength of concrete

f_{dc}' = dynamic compressive strength of concrete

f_{du} = dynamic tensile strength of reinforcement

f_{dpu} = dynamic tensile strength of prestressing steel

f_{dy} = dynamic yield strength of reinforcement

f_{pi} = ratio of initial prestress force to area of prestressing steel

f_{ps} = stress in prestressing steel at nominal flexural strength of component

f_{pu} = specified tensile strength of prestressing steel

f_u = specified tensile strength of reinforcement

f_y = specified yield strength of reinforcement

g = acceleration of gravity

h = thickness of component

h_1 = thickness of tension wythe of sandwich panel component

h_2 = thickness of compression wythe of sandwich panel component

i = impulse

i_s = positive impulse

i_s^- = negative impulse

I_{avg} = average moment of inertia of section

I_{cr} = moment of inertia of cracked section

I_g = gross moment of inertia of uncracked section

k_e = initial elastic stiffness of component

k_E = equivalent stiffness for indeterminate component

k_{ep} = secondary stiffness of indeterminate component

K = equivalent stiffness of spring-mass system

K_e = strength increase factor

K_{LM} = load-mass factor (a function of the deflected shape)

ℓ = clear span of component

L_{req} = length of distributed shear ties required

m = mass per unit area of blast-loaded component

M = mass of blast-loaded component

M_{du} = dynamic flexural resistance of blast-loaded component per unit width

M_{du-} = dynamic negative flexural resistance of blast-loaded component per unit width

M_{du+} = dynamic positive flexural resistance of blast-loaded component per unit width

M_n = nominal flexural strength at section (ACI)

n_p = ratio of moduli of prestressing steel to concrete

n_s = ratio of moduli of nonprestressed steel to concrete

N_{ties} = number of discrete ties

P_o = ambient pressure

$P_s(t)$ = positive pressure history on component

$P_s^-(t)$ = negative pressure history on component

P_{so} = peak positive pressure

P_{so}^- = peak negative pressure

$P(t)$ = blast load of single-degree-of-freedom system

r_e = resistance at first yield of indeterminate blast-loaded component

r_u = ultimate resistance of blast-loaded component

$(R(t))$ = calculated resistance history of component

$R(y(t))$ = resistance of component based on resistance-versus-deflection curve and deflection at each time

R_{max} = maximum resistance from single-degree-of-freedom analysis of component

t = time

t_1 = thickness of interior concrete wythe

t_2 = thickness of exterior concrete wythe

t_A = time of arrival of blast pressure at structure

t_d = equivalent triangular duration of pressure

t_f = thickness of insulation

t_o = positive phase duration

t_o^- = negative phase duration

T_n = natural period of structural component

U = fully factored connection load for load- and restance-factor design of connection per unit width of panel

V = interface shear load

V_c = concrete contribution to shear strength

V_{cap} = shear capacity of component

V_{int} = interface shear demand for blast-loaded sandwich wall panel

V_u = shear demand on the component

w_c = unit weight (density) of concrete

x = location along component span

$y(t)$ = deflection of single-degree-of-freedom system

$y''(t)$ = acceleration of single-degree-of-freedom system

y_1 = distance from center of gravity of gross section to center of gravity of tension wythe

y_2 = distance from center of gravity of gross section to center of gravity of compression wythe

y_e = initial yield deflection of system

y_E = equivalent yield deflection for indeterminate system

y_{max} = maximum deflection of component

y_p = deflection where the indeterminate component becomes a mechanism

y_{yield} = deflection causing yield of determinate component y_e or yield of equivalent elastic slope of indeterminate component y_E

β_1 = factor relating depth of equivalent rectangular compressive stress block to neutral axis depth (see ACI 318-11[19] section 10.2.7.3 for values)

γ_p = factor for type of prestressing steel (see ACI 318-11[19] section 18.7.2 for values)

θ = support rotation

μ = ductility ratio

ρ = ratio of A_s to bd

ρ_p = ratio of A_{ps} to bd_p

ϕ = strength reduction factor

$\phi(x)$ = shape function of simply supported beam

$\phi_1(x)$ = shape function of simply supported beam before yielding in maximum moment region

$\phi_2(x)$ = shape function of simply supported beam after yielding in maximum moment region

Ω_r = overstrength factor on maximum resistance