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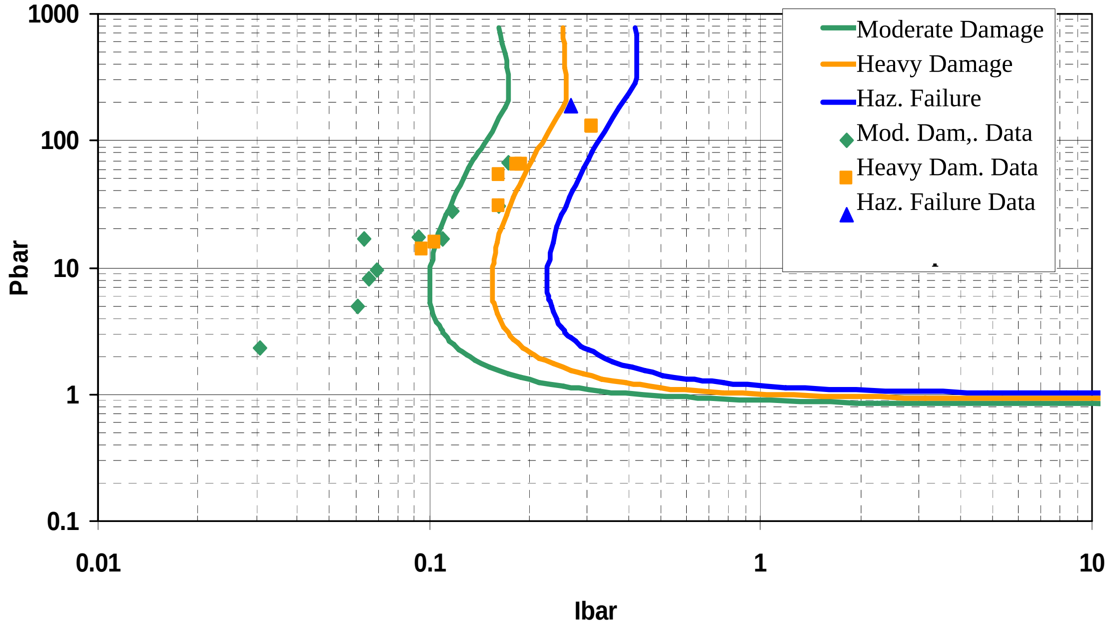
U.S. ARMY CORPS OF ENGINEERS

PROTECTIVE DESIGN CENTER TECHNICAL REPORT

User's Guide for Component Explosive Damage Assessment Workbook (CEDAW v2)



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INTRODUCTION

1) See *Instructions* on the bottom of the Introduction sheet in the CEDAW workbook for general help on using the workbook. 2) Click on the applicable hyperlink from the table below for specific help with input items. 3) Click on the CEDAW.xls application button at bottom of your computer screen to continue input.

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FIGURES

TABLES

Save and Retrieve Files for CEDAW Input/Output and Plots

SAVE and RETRIEVE click buttons	Use SAVE button to save user input for any component type into a text file in a user designated directory on the computer hard drive. The save file information can be read back into CEDAW using the RETRIEVE button on the Input sheets. A saved file with any component type or units can be retrieved from an Input sheet for any component type. User Defined input is saved.
Save Plots to DPLOT File	These options save all curves on the P-i Diagram, Reflected CW-Standoff Chart, and Side-on CW-Standoff Chart to a user-named file that can be read with the DPLOT computer program. Use Option A in the DPLOT program file read menu to read the saved file.

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Input for Steel Beams, Beam-Columns, Corrugated Panels, and Plates

Item	Explanation
Flexural Response	• Flexural response is based on yielding at the maximum moment regions with resistance-deflection curves as shown in Figure 1. Load-mass, stiffness, and ultimate flexural resistance values for all one-way and two-way components are based on Chapter 3, TM 5-1300, except as below. • The elastic resistances for two-way components with fixed supports are based on Table 10-5 in UFC 3-340-01. For components with adjacent fixed supports, the elastic resistance is based only on the negative moment capacity acting on the yield lines pattern for ultimate resistance. • All ultimate resistances for two-way spanning components are calculated based on TM 5-1300 with a 1.08 increase factor to account for conservative approach in TM 5-1300 where only 2/3 maximum moment capacity is assumed in corners.
Response Type	• Flexural response (or Beam flexural response) is used for flexural response of panels, plates, or beams as described in the row above. The full moment capacity is calculated based on the applicable section modulus and dynamic yield strength as described in the row below. • Cold-formed beam flexure with tension membrane response can be used for light, cold-formed steel members that are connected to structural steel framing members with bolted or welded connections or bolted into a concrete slab or beam. It is based on flexural yielding at the maximum moment regions (using K_E in Figure 1 for indeterminate boundary conditions) followed by linear tension membrane response as shown in Figure 2. Tension membrane should not be assumed for typical hot-rolled structural members since it is doubtful that the connections provided in typical construction can add significantly to the higher flexural resistance of these components. Conservatively, tension membrane response can always be ignored. In the unusual case where corrugated panels are connected to supporting members that have enough dynamic flexural capacity to provide significant in-plane tensile restraint to panels and the panel screw spacing is 6 inches or less, the panels can be input as beams with tension membrane using cross sectional properties on a per foot basis and a one foot beam spacing. • Column flexural response is used for ground level columns with connections that are not in shear, due to baseplates embedded in the slab or to column continuity into the basement and connections at the top of the ground floor that are not in shear. Column flexural response is similar to beam flexural response except that a P-i diagram is only constructed for LLOP based on more conservative assumptions than beam flexure. Ground floor perimeter columns can be analyzed in CEDAW to determine possible failure (i.e., a VLLOP) and therefore potential progressive collapse of supported components. Progressive collapse should be assumed unless it can be shown by analysis or test data that progressive collapse is unlikely for a given case of column failure.
Moment Capacity	• Moment capacity for corrugated panels is calculated as f_{dy}* S, where f_{dy} is the dynamic yield strength and S is the applicable section modulus – equal to the average of the positive and negative moment regions for fixed and fixed-simple boundary conditions. • Moment capacity for steel plates is calculated as f_{dy}*(S+Z)/2. • Moment capacity for cold-formed beams is calculated as f_{dy}*S, where S is the effective section modulus. • Moment capacity for hot-rolled beams is calculated as f_{dy}*Z.

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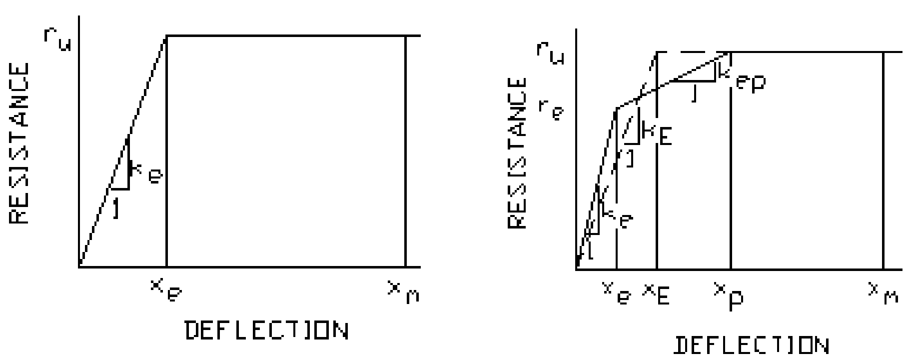
Item	Explanation
Spacing, B	• This input is not used for corrugated steel panel or plate input. For a beam or column, input the width of the building area that effectively loads the beam or column with the applied blast load. Typically, this is the spacing between beams. If wall cladding spans vertically between floors and does not load the columns, input the column flange width or depth depending on the column orientation (i.e., dimension parallel to blast loaded surface). If such cladding is rigid, some engineering judgment may be required to estimate the width of the cladding that will effectively load the column. This is also true when the column or a large girder supports lightweight cladding. In the case of a typical pre-engineered building, approximately 20% of the tributary wall width may be a good assumption for the input spacing. This recognizes that the cladding is much less blast resistant than the column and only transfers a portion of the blast load applied to the full tributary wall area supported by the column before failing when subject to blast loads high enough to cause column failure.
Supported Weight	• Input the supported weight that moves through same deflection as component. This is equal to the total supported weight for panels and closely spaced beams (i.e., up to approximately 7 ft). Conservatively, 20% of the supported weight over the supported width can be input for beams or columns at further spacing. Columns do not have any supported weight unless cladding is directly attached to the column or a portion of the cladding bears against the column under blast load. It is conservative to ignore supported weight.
Static and Dynamic Increase Factors, SIF and DIF	• The steel dynamic yield strength f_{dy}=f_y*SIF*DIF. See Table 1 and Table 2 for information on the SIF and DIF. The minimum specified static yield strength = f_y. The SIF and DIF can be considered functions of f_y.
Support capacity for tension membrane, V_c	• V_c is the in-plane tension strength provided by the connection and supporting framing members to the input component. See Equation 1 for effect of V_c on tension membrane response. • V_c equals the minimum of the in-plane capacity of the component connection or the ultimate dynamic in-plane flexural strength of the framing component supporting the component of interest. Typically the connection capacity controls V_c, which may be controlled directly by the bolt or weld or by the bearing capacity of the component material surrounding the bolt. • V_c can be based on the full in-plane capacity of the connection when the component bears against its support during blast load, otherwise some reduction can be made to account for out-of-plane shear forces on the connection acting with in-plane forces.



Determinate Boundary Conditions

Indeterminate Boundary Conditions
(Solid Curve Used for Flexure Only)
(Dashed Curve for Flexure and Tension Membrane)

Figure 1. Resistance-Deflection Curve For Flexural Response

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Table 1. Recommended Steel Static Increase Factors (SIF) in TM 5-1300

Material	Minimum Yield Strength (f_y) (psi)	Static Increase Factor[1]
A36	36,000	1.1
A588	50,000	1.05[2]
A514	100,000	1.0
Corrugated panels	All	1.2
Note 1: Also referred to as the Average Strength Factor Note 2 Interpolated based on f_y		

Table 2. Recommended Steel Dynamic Increase Factors (DIF) in TM 5-1300

Material	Beam in Bending	
	Low Pressure*	High Pressure*
A36 (f_y=36 ksi)	1.29	1.36
A588 (f_y=50 ksi)	1.19	1.24
A514 (f_y=100 ksi)	1.09	1.12
Corrugated panels	1.1	1.1
** Low pressure can be conservatively assumed in all cases. High pressure can be assumed when the scaled standoff < 3 ft/lb$^{1/3}$. See Chapter 4 in UFC 3-340-01 for DIF information as a function of strain-rate.		

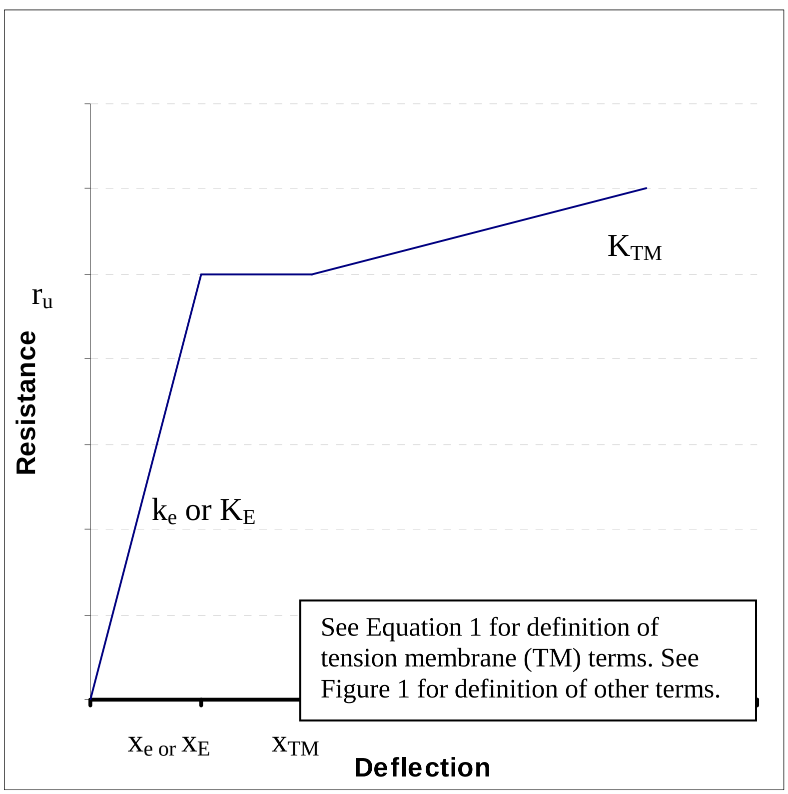

Figure 2. Resistance Deflection Curve for Steel Components with Tension Membrane (See Equation 1 for terms)

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$$K_{TM} = \frac{8T}{bL^2} \quad \text{where} \quad T = \text{Minimum}\left[\left(f_{dy}A\right), V_c\right]$$

Equation 1

where:

x_{TM} = deflection at beginning of simplified tension membrane response curve (assumed equal to 2 degrees support rotation in development of CEDAW P-i diagrams for steel beams and panels)

K_{TM}= linear tension membrane slope

x_E = equivalent elastic yield deflection

f_{dy} = dynamic yield strength

A = minimum net component cross sectional area (within width b for panels)

V_c = support force available for resisting component tension (Based on lesser of connection capacity or in-plane flexural capacity of supporting framing member)

L = span length (least span length for two-way components)

b = beam spacing (typically a unit width for panels)

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Input for Metal Stud Walls

Item	Explanation
Response Type	• Studs with sliding connection – Studs are not screwed to a horizontal channel section at both the top and bottom support that is attached to the building. Usually, the studs sit within and bear against a channel at the top so that they can expand vertically without inducing any stresses. The flexural response of studs is assumed to only reach a portion of the yield strength depending on the LOP before the stud pushes through the channel leg at the unscrewed support. This is a typical situation for infill stud walls in a frame building. • Studs connected top and bottom - Studs are screwed into a horizontal channel section at both the top and bottom support that is attached to the building, or otherwise positively connected to both top and bottom supports. The studs are assumed to yield at the maximum moment regions with resistance-deflection curves as shown in Figure 5. • In all cases the load-mass, stiffness, and resistance based on UFC 3-340-01 and TM 5-1300 are used to calculate P-i curve values. The moment capacity is equal to the product of the effective section modulus and dynamic yield strength. • If the studs are bolted to supports so as to allow tension membrane, analyze studs using the *Steel Beams or Columns* component type.
Metal Stud Shapes	• Structural metal stud shapes are shown in the drop down table as designated by the Steel Stud Manufacturer's Association (SSMA) for structural and non-structural studs. The stud shape designations refer to member dimensions as shown in Figure 4 and Table 3. Also, see www.ssma.com/ssmatechcatalog.pdf for additional information. Manufacturers with their own shape designations typically have a correlation to the SSMA shape designations available on their website or from their technical representatives. User defined cross sectional information can also be input.
Supported Weight	• Input the supported weight that moves through same deflection as component. This is equal to the total supported cladding weight for the typical case of closely spaced studs (i.e.,up to approximately 7 ft). Conservatively, 20% of the supported weight over the supported width can be input for studs at further spacing.
Static and Dynamic Increase Factors, SIF and DIF	• The steel dynamic yield strength $f_{dy}=f_y$*SIF*DIF. Recommended values for SIF and DIF for cold-formed components in TM 5-1300 are 1.21 and 1.1, respectively, for all typical yield strengths.
Moment Capacity	• Moment capacity for the cold-formed studs is calculated as f_{dy}*S, where S is the effective elastic section modulus.

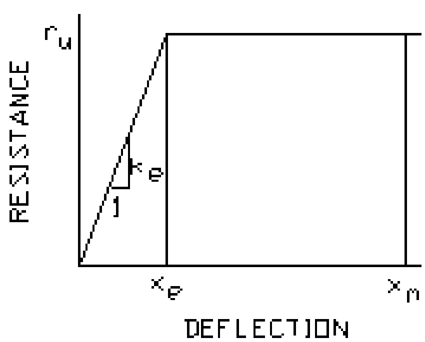

Figure 3. Resistance-Deflection Curve For Flexural Response

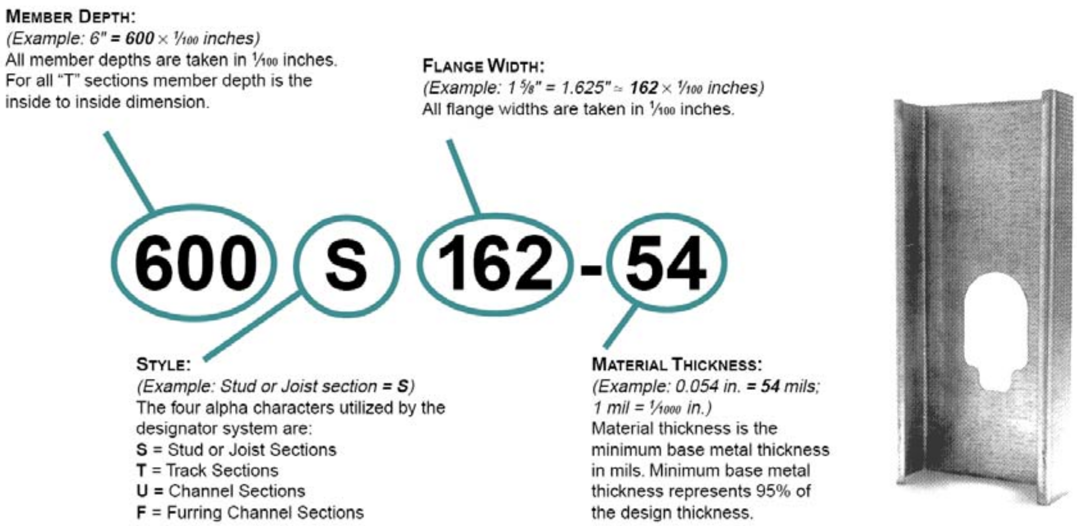

Figure 4. Metal Stud Size Designation and Cross Section of Typical S-Section Stud with Punch-Out (See Table 3 Below for Information on Mils)

Table 3. Correlation Between Metal Stud Gage Thickness and Mils

Gage	Minimum Thickness (1/1000 of an inch= mil)	Minimum Thickness (mm)
25	18	0.455
22	27	0.683
21	30	0.752
20	33	0.836
18	43	1.09
16	54	1.37
14	68	1.72
13	88	2.24
12	97	2.45
10	118	3.00

Input for Open Web Steel Joists

Item	Explanation
Tension membrane force, T_{tm}	• T_{tm} is the lesser of ultimate dynamic tensile capacity of top chord or the ultimate dynamic shear capacity of connection of top chord to its support. Typically, T_{tm} can be input as 120% of ultimate static capacity from 2 inches of 1/8 inch weld on an H or K Type joist (9 kips) or 100% of ultimate static capacity from 4 inches of 1/4 inch weld on LH or DLH Type joist (30 kips). These are minimum weld lengths required by the Steel Joist Institute. Tension membrane force is added to flexural resistance as shown in Figure 5. T_{tm} can be set equal to zero when the integrity of the connection is not assured, the wall cannot dynamically resist the force T_{tm} in the plane of the roof, or a more conservative approach is desired. • An input of T_{tm}=0 will cause P-i curves that are only based on the flexural response of the joist.
Allowable joist * design load, w_{design}	• Allowable load for given span from load tables in manufacturer's and Steel Joist Institute literature. CEDAW backs out a constant moment capacity for each joist from load tables and uses this to calculate w_{design} except that a constant w_{design} is used for when the span length is less the critical value causing constant w_{design} in the load tables. • Ultimate dynamic flexural resistance is based on 2.12*w_{design}, where the 2.12 factor includes a safety factor of 1.7 and static and dynamic increase factors of 1.05 and 1.19, respectively. • If the ultimate capacity is controlled by shear response rather than flexural yielding in the chords, lower maximum response criteria should be used for design. Shear control can be indicated by spans where the change in allowable load capacity is not proportional to the square of the change in span length in the manufacturer's literature.
Load causing * L/360 deflection, w_{LL}	• Load causing L/360 deflection for given span from load tables in manufacturer's and Steel Joist Institute literature. The literature has values of w_{LL} for shorter spans, where w_{design} becomes constant, that probably underestimate the actual load causing L/360 deflection. CEDAW backs out a constant EI for each joist from load tables and uses this to calculate w_{LL} for all span lengths. • The elastic stiffness for SDOF calculations is based on w_{LL}/(L/360)
*Above values are not normally required inputs – information is provided for background knowledge or for cases where "User Defined" input is required.	

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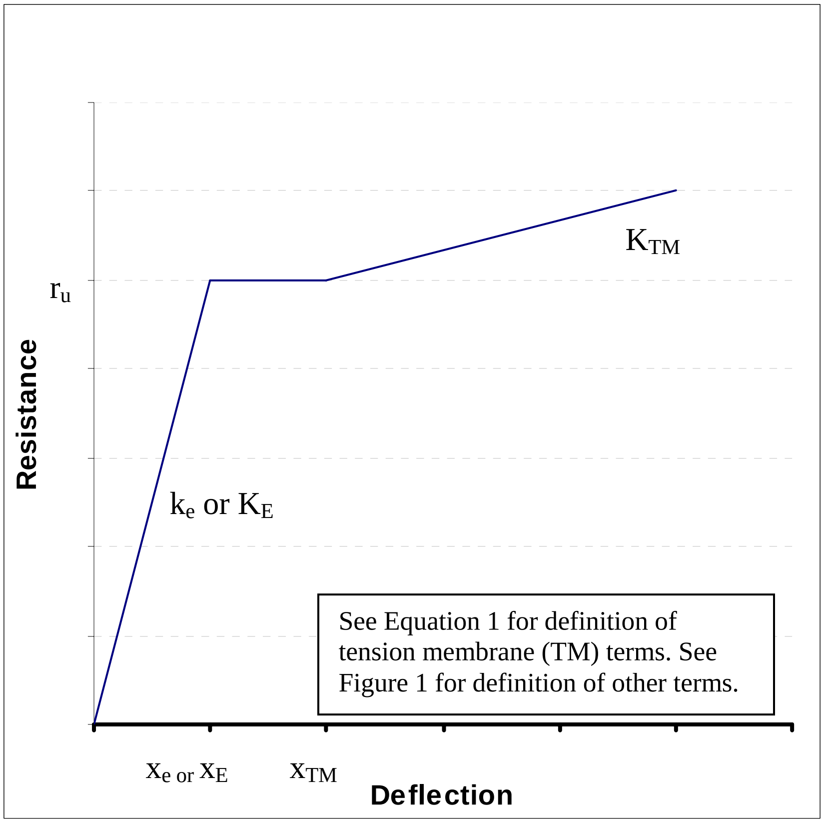

Figure 5. Resistance Deflection Curve for Steel Components with Tension Membrane (See Equation 2 for terms)

$$T = \text{Minimum}\left[\left(f_{dy}A\right), V_c\right]$$

$$K_{TM} = \frac{8T}{bL^2}$$

Equation 2

where:

x_{TM}= deflection at beginning of simplified tension membrane response curve (assumed equal to 2 degrees support rotation in development of CEDAW P-i diagrams for open web steel joists)

K_{TM}= linear tension membrane slope

x_E = equivalent elastic yield deflection

f_{dy} = dynamic yield strength for top chord = f_y(SIF)(DIF). See Table 4 and Table 5 for information on SIF and DIF (typically f_y=50,000 psi).

A = cross sectional area of top chord

V_c = weld strength of connection to supporting member. See table above for guidance on V_c.

L = span length (least span length for two-way components)

b = joist spacing

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Table 4. Recommended Steel Static Increase Factors (SIF) in TM 5-1300

Material	Minimum Yield Strength (f_y) (psi)	Static Increase Factor[1]
A36	36,000	1.1
A588	50,000	1.05[2]
A514	100,000	1.0
Corrugated panels	All	1.2
Note 1: Also referred to as the Average Strength Factor Note 2 Interpolated based on f_y		

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Material	Beam in Bending	
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A588 (f_y=50 ksi)	1.19	1.24
A514 (f_y=100 ksi)	1.09	1.12
Corrugated panels	1.1	1.1

Input for Reinforced Concrete Slabs

Item	Explanation
Flexural Response	• Flexural response is based on yielding at the maximum moment regions with resistance-deflection curves as shown in Figure 6. Load-mass, stiffness, and ultimate flexural resistance values for all one-way and two-way components are based on Chapter 3, TM 5-1300, except as below. • The elastic resistances for two-way components with fixed supports are based on Table 10-5 in UFC 3-340-01. For components with adjacent fixed supports, the elastic resistance is based only on the negative moment capacity acting on the yield lines pattern for ultimate resistance. • All ultimate resistances for two-way spanning components are calculated based on TM 5-1300 with a 1.08 increase factor to account for conservative approach in TM 5-1300 where only 2/3 maximum moment capacity is assumed in corners.
Loaded Width/Slab Width	• Ratio of loaded wall width divided by the slab width resisting blast load. This ratio is never less than 1.0. It equals 1.0 for a solid slab. It is typically > 1.0 for a concrete wall slab with window openings, where the window is assumed to transfer blast load into the wall. In this case, the loaded width is the center-to-center distance between windows and the slab width is the width of slab between windows. When loaded width/slab width (Bw) > 1, the weight of the slab is divided Bw, but the supported weight is not. Thus, the supported weight is independent of Bw.
Reinforcing Steel Areas and Bar Spacing	• See Figure 7 for definition of terms used for input of reinforcing steel areas and bar spacing.
Distance of Cover to Center of Bars, d_c	• See Figure 7 for definition of terms used for input cover distance (d_c) over reinforcing bars. Input average value of d_c for loaded and unloaded face in given direction L or H if the component has fixed supports in either direction. Otherwise, input d_c for the unloaded face.
Supported Weight	• Input any supported weight that moves through same deflection as component. Do not include self-weight of slab.
Reinforcing Steel Yield Strength Dynamic and Static Strength Increase Factors	• Default values of 1.1 for the static increase factor and 1.17 for the dynamic increase factor are used in CEDAW as recommended in TM 5-1300. See UFC 3-340-01 Chapter 4 for more detailed information as a dynamic increase factor as a function of strain-rate. The increase factors can be modified using the "User Defined" input for the reinforcing steel.
Moment of Inertia (I)	• I is calculated as the average cracked and uncracked moment of inertia conservatively assuming the cracked moment of inertia is insignificant compared to the uncracked moment of inertia. I is not an input.

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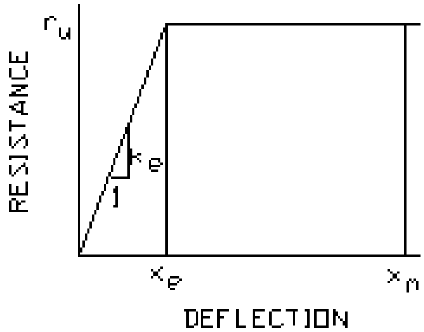
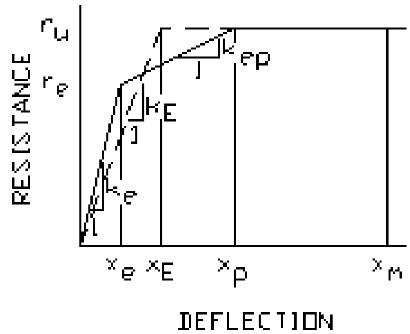


Determinate Boundary Conditions



Indeterminate Boundary Conditions
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(Dashed Curve for Flexure and Tension Membrane)

Figure 6. Resistance-Deflection Curve For Flexural Response

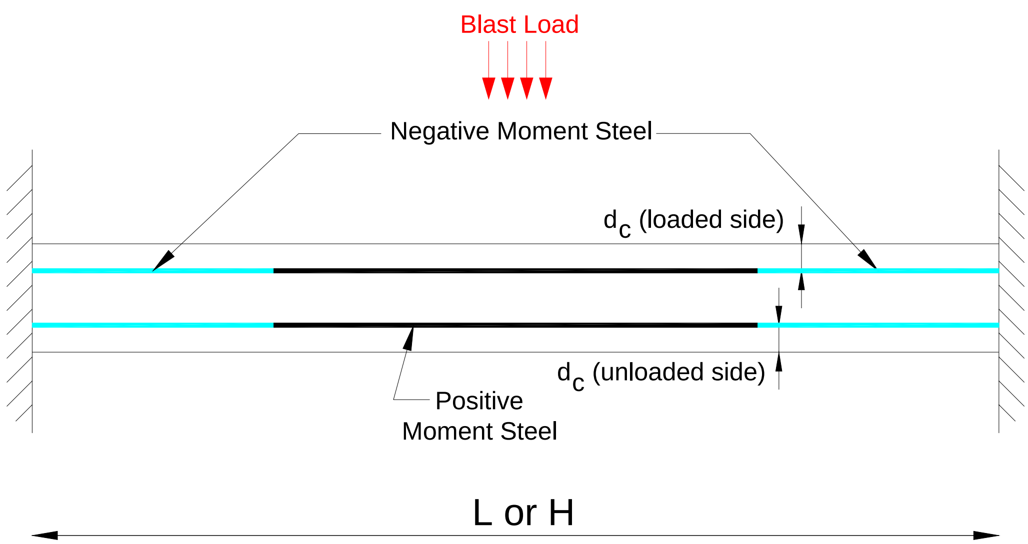

Figure 7. Information for Input of Steel Area and Distance of Cover Depth (d_c)

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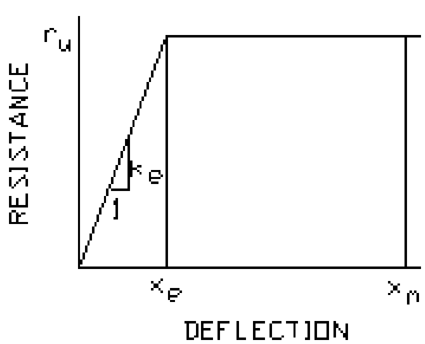
Input for Reinforced Concrete Beams

Item	Explanation
Flexural Response	• Flexural response is based on yielding at the maximum moment regions with resistance-deflection curves as shown in Figure 8. Load-mass, stiffness, and resistance values used to calculate the P-i curves are based on UFC 3-340-01 and TM 5-1300..
Reinforcing Steel Areas	• See Figure 9 for definition of terms used for input of reinforcing steel areas
Distance of Cover to Center of Bars	• See Figure 9 for definition of terms used for input cover distance (d_c) over reinforcing bars.
Supported Weight	• Input the supported weight that moves through same deflection as component. This is equal to the total supported weight for closely spaced beams (i.e., up to approximately 7 ft). Conservatively, 20% of the supported weight over supported width area can be input for beams at further spacing.
Reinforcing Steel Yield Strength Dynamic and Static Strength Increase Factors	• Default values of 1.1 for the static increase factor and 1.17 for the dynamic increase factor are used in CEDAW as recommended in TM 5-1300. See UFC 3-340-01 Chapter 4 for more detailed information as a dynamic increase factor as a function of strain-rate. The increase factors can be modified using the "User Defined" input for the reinforcing steel.
Moment of Inertia (I)	• I is the average cracked and uncracked moment of inertia calculated according to UFC 3-340-01 Eq. 10-31. Figure 10-5 for a singly reinforced cross section is used to determine the cracked section coefficient, F, for $I_{cracked}$ for reinforced concrete. This is slightly conservative for a doubly reinforced cross section. I is not an input.



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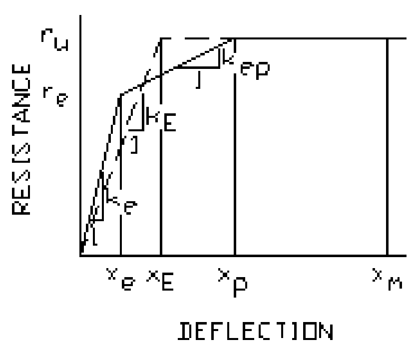

Indeterminate Boundary Conditions
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(Dashed Curve for Flexure and Tension Membrane)

Figure 8. Resistance-Deflection Curve For Flexural Response

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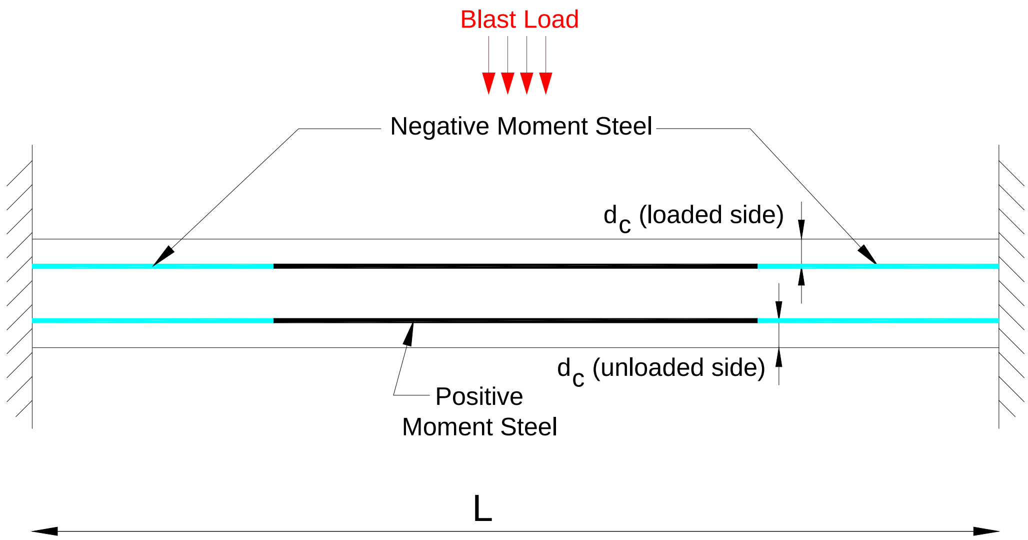

Figure 9. Information for Input of Steel Area and Distance of Cover Depth (d_c)

Input for Reinforced Masonry Walls

Item	Explanation
Flexural Response	• Flexural response is based on yielding at the maximum moment regions with resistance-deflection curves as shown in Figure 10. Load-mass, stiffness, and resistance values used to calculate the P-i curves are based on UFC 3-340-01 and TM 5-1300.
Loaded Width/Slab Width	• Ratio of loaded wall width divided by the wall width resisting blast load. This ratio is never less than 1.0. It equals 1.0 for a solid slab. It is typically > 1.0 for a concrete wall slab with window openings, where the window is assumed to transfer blast load into the wall. In this case, the loaded width is the center-to-center distance between windows and the slab width is the width of slab between windows. When loaded width/slab width (Bw) > 1, the weight of the slab is divided Bw, but the supported weight is not. Thus, the supported weight is independent of Bw.
Reinforcing Steel Areas	• See Figure 11 for definition of terms used for input of reinforcing steel areas. If a masonry wall with a fixed support has a single layer of reinforcing steel at midthickness, enter the reinforcing bar area and spacing as both “Positive Moment” and “Negative Moment” steel.
Distance of Cover to Center of Bars, d_c	• See Figure 11 for definition of terms used for input cover distance (d_c) over reinforcing bars. Typically reinforcement is located at the center of the masonry thickness of CMU walls.
Masonry Type	• See Table 6 and Table 7 for assumptions for each available masonry type. The user can also choose the User Defined option.
Percent of Void Spaced Grouted	• This input is only applicable for CMU walls. It is the percentage of the cells that are grouted, so that a fully grouted wall would have an input of 100%. Typically, this input equals 8 inches divided by the reinforcing bar spacing expressed as a percentage, since usually only the reinforced cells are grouted and the cells are typically spaced at 8 inches in most CMU block.
Masonry Static Compressive Strength, f'_m	• Input masonry prism strength. If no measured values are available, see conservative recommended values in Table 8 from TM 5-1300.
Reinforcing Steel Yield Strength Dynamic and Static Strength Increase Factors	• Default values of 1.1 for the static increase factor and 1.17 for the dynamic increase factor are used in CEDAW as recommended in TM 5-1300. See UFC 3-340-01 Chapter 4 for more detailed information as a dynamic increase factor as a function of strain-rate. The increase factors can be modified using the “User Defined” input for the reinforcing steel.
Supported Weight	• Input supported weight that moves through same deflection as component. Do not include wall self-weight.
Moment of Inertia (I)	• I is the average cracked and uncracked moment of inertia calculated according to TM5-1300, Equation 6-6. Equation 6-7 in TM 5-1300 is used to calculate $I_{cracked}$ for reinforced masonry. I is not an input.

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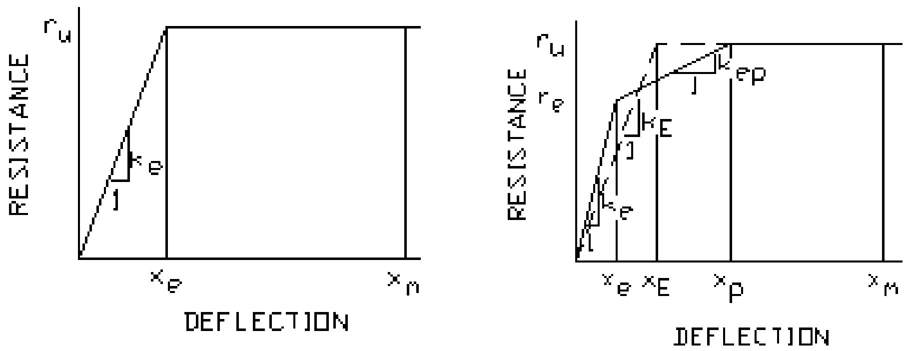

Determinate Boundary Conditions

Indeterminate Boundary Conditions
(Solid Curve Used for Flexure Only)
(Dashed Curve for Flexure and Tension Membrane)

Figure 10. Resistance-Deflection Curve For Flexural Response

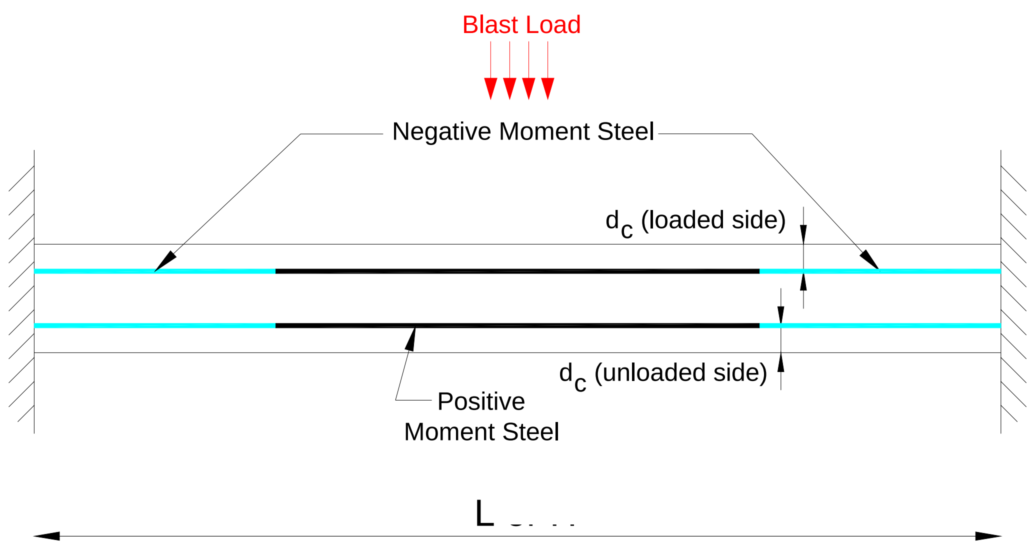

Figure 11. Information for Input of Steel Area and Distance of Cover Depth (d_c)

Table 6. Masonry Block Information Assumed In CEDAW

Type	Density (pcf)	Solid Ratio (%)
Brick	120	100
European Insulated Block (see Figure 12)	60 – 120*	50**
Heavy Weight CMU	135	See Table 7
Light Weight CMU	95	
Medium Weight CMU	120	

* 87 pcf assumed in CEDAW calculations – use User Defined option if incorrect.
** An approximate, generally conservative value for blocks shown in Figure 12.

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Table 7. Assumed Dimensions and Area Ratios for CMU Blocks

Block Thickness (in)	Face Shell Thickness (in)	Web Thickness (in)	Solid Ratio (%)
4	0.75	0.75	50
6	1.25	1.0	55
8	1.25	1.25	49
10	1.25	1.25	43
12	1.25	1.25	40

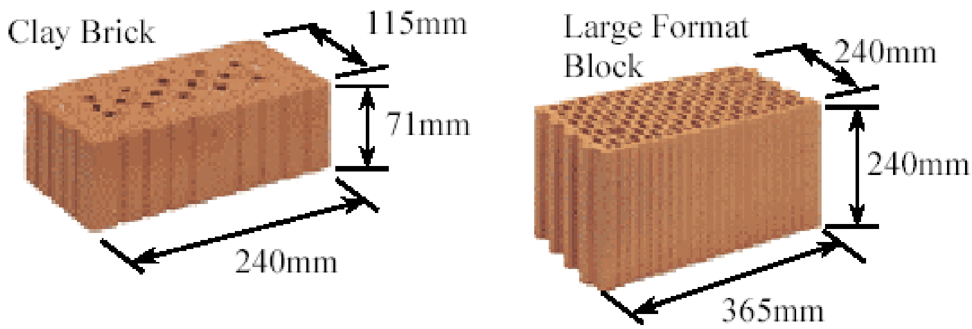


Figure 12. Small and Large European Insulated Blocks

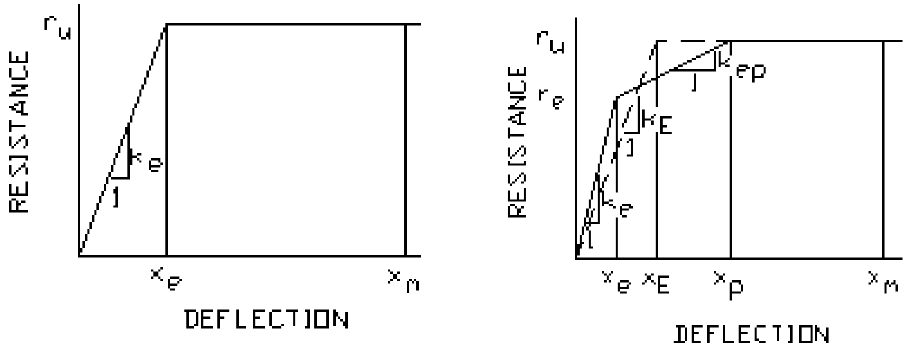
Table 8. Recommended Conservative Values for Compressive Strength

Type of Masonry Unit	Compressive Strength (f'm)
Ungrouted CMU	1350 psi
Fully Grouted CMU	1500 psi
Brick or Solid Masonry	1800 psi

Input for Unreinforced Masonry Walls

Item	Explanation
Brittle Flexural Response with Axial Load	• The response is based on assumptions summarized in Figure 14. The r_1 and r_2 values are the initial yield and ultimate flexural resistance. r_3 is calculated based on axial load from wall self-weight, see Equation 3. Conservatively, no axial P load is assumed. • Figure 15 shows resistance-deflection curves used to determine the P-i diagrams values in CEDAW. Flexural response is only assumed to occur up to the deflection causing ultimate flexural resistance. Arching from self-weight axial load controls the resistance softening at all larger deflections. • See comments on flexural response in next row.
Elastic-Plastic Flexural Response	• Flexural response is based on yielding at the maximum moment regions with resistance-deflection curves as shown in Figure 13. Load-mass, stiffness, and ultimate flexural resistance values for all one-way and two-way components are based on Chapter 3, TM 5-1300, except as below. • The elastic resistances for two-way components with fixed supports are based on Table 10-5 in UFC 3-340-01. For components with adjacent fixed supports, the elastic resistance is based only on the negative moment capacity acting on the yield lines pattern for ultimate resistance. • All ultimate resistances for two-way spanning components are calculated based on TM 5-1300 with a 1.08 increase factor to account for conservative approach in TM 5-1300 where only 2/3 maximum moment capacity is assumed in corners.
Loaded Width/Slab Width	• Ratio of loaded wall width divided by the slab width resisting blast load. This ratio is never less than 1.0. It equals 1.0 for a solid slab. It is typically > 1.0 for a concrete wall slab with window openings, where the window is assumed to transfer blast load into the wall. In this case, the loaded width is the center-to-center distance between windows and the slab width is the width of slab between windows. When loaded width/slab width (Bw) > 1, the weight of the slab is divided Bw, but the supported weight is not. Thus, the supported weight is independent of Bw.
Masonry Type	• See Table 9 and Table 10 for assumptions for each available masonry type. The user can also choose the User Defined option.
Double Wythe Wall Systems	• SBEDS allows input of double wythe walls, where the walls are assumed to deflect together. Both walls must have the same height and/or width dimensions and the same boundary conditions. They can have different cross sectional properties. • The load resisted by each wall in flexural response is based on the relative wall stiffnesses, as recommended in ACI 530-02, Section 2.1.5.3.1. The walls are assumed to act in flexural as two springs in parallel until one of the walls yields, when the two wall system is assumed to achieve maximum flexural resistance. See Equation 4 and Equation 5.
Percent of Void Spaced Grouted	• This input is only applicable for CMU walls. It is the percentage of the cells that are grouted, so that a fully grouted wall would have an input of 100%. Typically unreinforced CMU is not grouted.
Masonry Dynamic Tensile Strength, f_{dt}	• Input dynamic tensile strength for flexural response calculations. Recommended input value is $0.1f'm \geq 200$ psi.
Masonry Static Compressive Strength, f'_m	• Input masonry prism strength. If no measured values are available, see recommended values in Table 11 from TM 5-1300.
Applied Axial Load, P_A:	• Input dead and live axial load supported by wall at time of explosion. Conservatively, use P_A=0.
Moment Capacity	• The moment capacity for flexural response is $S*f_{dt}$, where S=elastic section

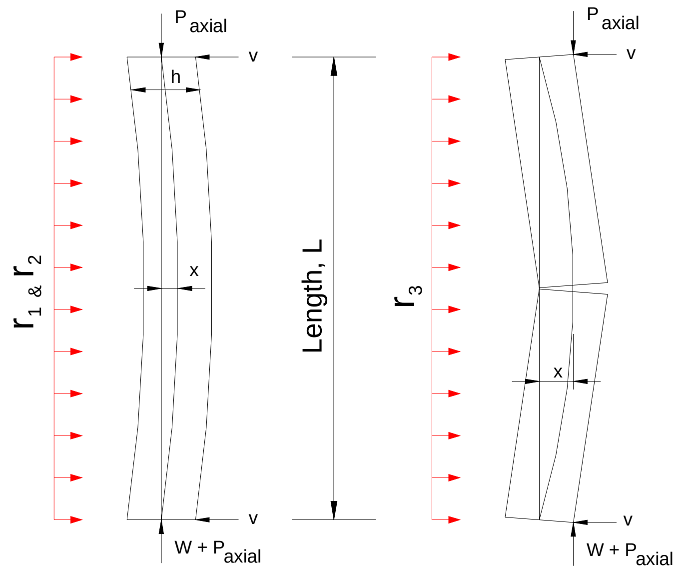
	modulus.



Determinate Boundary Conditions

Indeterminate Boundary Conditions
(Solid Curve Used for Flexure Only)
(Dashed Curve for Flexure and Tension Membrane)

Figure 13. Resistance-Deflection Curve For Flexural Response

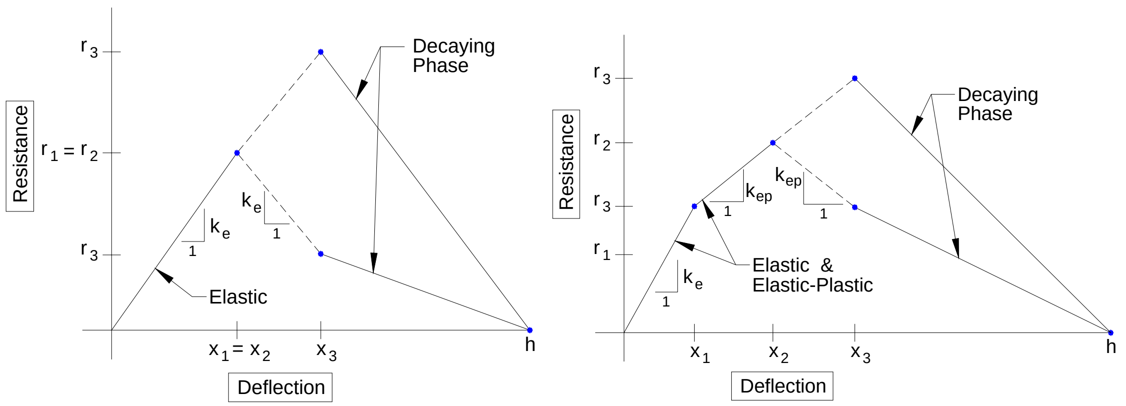


Response prior to ultimate flexural resistance, r_2

Response after r_2 where x= arching moment arm

Figure 14. Response of Brittle Unreinforced Masonry Wall Under Combined Lateral and Axial Load

Help TOC



Determinate Component Indeterminate Component

Figure 15. Resistance-Deflection Curves for Unreinforced Masonry with Brittle Flexural Response and Axial Load From Wall Self Weight

$$r_3 = \frac{8}{L^2}(h - x_2)\left(\frac{WL}{2}\right)$$

Equation 3

where:
- r_3 = maximum resistance from axial load effects
- x_2 = flexural yield deflection
- x_3 = flexural deflection at $r_2 + (r_3 - r_2)/K_{ep}$
- K_{ep} = elastic-plastic stiffness for indeterminate components, otherwise equal to elastic stiffness
- h = overall wall thickness
- W = areal self-weight of wall
- L = span length equal to wall height

Table 9. Masonry Block Information Assumed In CEDAW

Type	Density (pcf)	Solid Ratio (%)
Brick	120	100
European Insulated Block (see Figure 16)	60 – 120*	50**
Heavy Weight CMU	135	See Table 7
Light Weight CMU	95	
Medium Weight CMU	120	

* 87 pcf assumed in CEDAW calculations – use User Defined option if incorrect.
** An approximate, generally conservative value for blocks shown in Figure 16.

Table 10. Assumed Dimensions and Area Ratios for CMU Blocks

Block Thickness (in)	Face Shell Thickness (in)	Web Thickness (in)	Solid Ratio (%)
4	0.75	0.75	50
6	1.25	1.0	55
8	1.25	1.25	49
10	1.25	1.25	43
12	1.25	1.25	40

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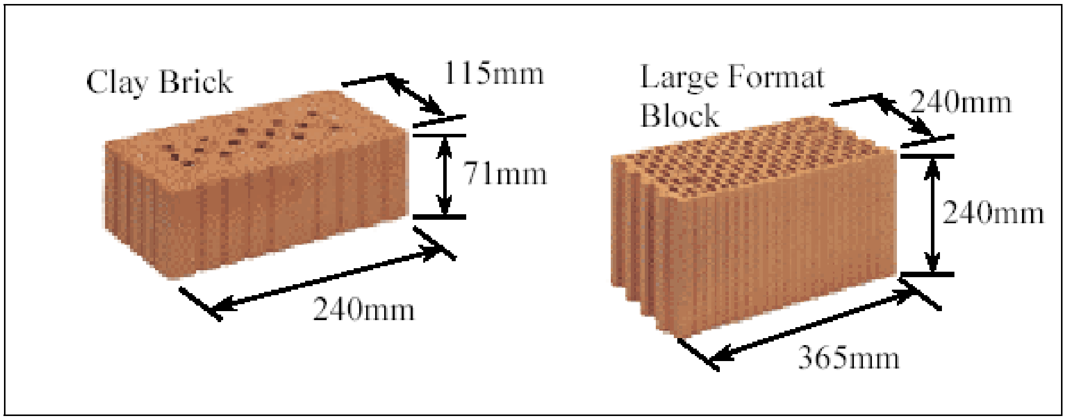

Figure 16. Small and Large European Insulated Blocks

Table 11. Recommended Conservative Values for Compressive Strength

Type of Masonry Unit	Compressive Strength (f'm)
Ungrouted CMU	1350 psi
Fully Grouted CMU	1500 psi
Brick or Solid Masonry	1800 psi

$$R_1 = K_1\Delta \qquad R_2 = K_2\Delta \qquad therefore\ \frac{R_1}{R_2} = \frac{K_1}{K_2}\ and\quad K_i \propto \frac{C_K E_i I_i}{L^4} \quad so\ \frac{R_1}{R_2} = \frac{K_1}{K_2} = \frac{E_1 I_1}{E_2 I_2}$$

$$R_i \propto \frac{C_m M_i}{L^2} \qquad therefore\ \frac{R_1}{R_2} = \frac{M_1}{M_2} = \frac{\sigma_1 S_1}{\sigma_2 S_2} \quad and\quad \frac{\sigma_1}{\sigma_2} = \frac{R_1 S_2}{R_2 S_1} = \frac{E_1 I_1 S_2}{E_2 I_2 S_1} = \sigma_r\ , Let\quad \sigma_{ry} = \frac{\sigma_{1y}}{\sigma_{2y}}$$

If $\frac{\sigma_r}{\sigma_{ry}} \geq 1$, Wall 1 yields first and $R_u = R_{1u} + R_2 = R_{1u}\left(1 + \frac{E_2 I_2}{E_1 I_1}\right)$,

Else, $R_u = R_{2u} + R_1 = R_{2u}\left(1 + \frac{E_1 I_1}{E_2 I_2}\right)$ Also, $\mathrm{Max}(R_{1u}, R_{2u}) \leq R_u \leq (R_{1u}+R_{2u})$

Equation 4

See Equation 5 for parameter definitions

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$$K = K_1 + K_2$$

Equation 5

where:

i =1 for inner wall and 2 for outer wall
R_i = resistance i^{th} wall
R_{iu} = ultimate or maximum resistance of i^{th} wall
R_u = ultimate or maximum resistance of two-wall system
K_i = flexural stiffness of i^{th} wall
K = flexural stiffness of two-wall system
E_i = Young's modulus of walls
I_i = moment of inertia of i^{th} wall
S_i = section modulus of i^{th} wall
Δ = equal midpan deflections of both walls
L = equal spans of both walls
σ_i = maximum flexural stress in walls
M_i = maximum moment capacity of i^{th} wall
σ_{iy} = yield stress of i^{th} wall
C_m = moment constants dependent on boundary conditions - same for each wall
C_K = stiffness constants dependent on boundary conditions - same for each wall

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Input for Wood Beams

Item	Explanation
Response Mode	• Only flexural response is considered for wood components with a number of available boundary and load conditions. No composite action with cladding is assumed based on limited testing where very well connected cladding did not significantly increase measured ultimate resistance of test walls.
Wood Density	• Wood density ranges based on wood type from 25 to 60 lb/ft^3. The AISC Manual of Steel Construction has a wide range of values in Part 6 under Miscellaneous Data.
Wood Species and Grade	• A wood species and grade can be selected and CEDAW will use properties from the National Design Specification® (NDS®) for Wood Construction Supplement: Design Values for Wood Construction, 2005 Edition in Table 12. The user can also choose the User Defined option and directly enter wood properties.
Modulus of Elasticity, E	• Wood modulus of elasticity generally ranges from $1x10^6$ psi to $2x10^6$ psi. CEDAW will use a value from Table 12 for a user designated wood species and grade.
Dynamic Yield Strength in flexure, f_y	• Input the ultimate dynamic rupture strength for wood in flexure. Typically, there is at least a 2.5 safety factor on allowable static strengths and a dynamic increase factor of 2.0 for very short term loading. Therefore, allowable static wood flexural strengths can be increased by at least a factor of 5.0. CEDAW will calculate a dynamic flexural strength equal to five times the allowable strength in Table 12for a user designated wood species and grade. The moment capacity if based on S*f_y, where S=section modulus.

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Table 12. NDS Wood Properties

Species	Allowable Flexural Strength (psi)								Modulus of Elasticity (psi)							
	Select Structural	No. 1	No. 2	No. 3	Stud	Con-struction	Stand-ard	Utility	Select Structural	No. 1	No. 2	No. 3	Stud	Con-struction	Stand -ard	Utility
ALASKA CEDAR	1,150	975	800	450	625	900	500	250	1.4E+06	1.3E+06	1.2E+06	1.1E+06	1.1E+06	1.2E+06	1.1E+06	1.0E+06
ALASKA HEMLOCK	1,300	900	825	475	650	950	525	250	1.7E+06	1.6E+06	1.5E+06	1.4E+06	1.4E+06	1.4E+06	1.3E+06	1.2E+06
ALASKA SPRUCE	1,400	950	875	500	675	1,000	550	275	1.6E+06	1.5E+06	1.4E+06	1.3E+06	1.3E+06	1.3E+06	1.2E+06	1.1E+06
ALASKA YELLOW CEDAR	1,350	900	800	475	625	925	500	250	1.5E+06	1.4E+06	1.3E+06	1.2E+06	1.2E+06	1.3E+06	1.1E+06	1.1E+06
ASPEN	875	625	600	350	475	700	375	175	1.1E+06	1.1E+06	1.0E+06	9.0E+05	9.0E+05	9.0E+05	9.0E+05	8.0E+05
BALD CYPRESS	1,200	1,000	825	475	650	925	525	250	1.4E+06	1.4E+06	1.3E+06	1.2E+06	1.2E+06	1.2E+06	1.1E+06	1.0E+06
BEECH-BIRCH-HICKORY	1,450	1,050	1,000	575	775	1,150	650	300	1.7E+06	1.6E+06	1.5E+06	1.3E+06	1.3E+06	1.4E+06	1.3E+06	1.2E+06
COAST SITKA SPRUCE	1300	925	925	525	725	1050	600	275	1.7E+06	1.5E+06	1.5E+06	1.4E+06	1.4E+06	1.4E+06	1.3E+06	1.2E+06
COTTONWOOD	875	625	625	350	475	700	400	175	1.2E+06	1.2E+06	1.1E+06	1.0E+06	1.0E+06	1.0E+06	9.0E+05	9.0E+05
DOUGLAS FIR-LARCH	1,500	1,000	900	525	700	1,000	575	275	1.9E+06	1.7E+06	1.6E+06	1.4E+06	1.4E+06	1.5E+06	1.4E+06	1.3E+06
DOUGLAS FIR-LARCH (NORTH)	1,350	850	850	475	650	950	525	250	1.9E+06	1.6E+06	1.6E+06	1.4E+06	1.4E+06	1.5E+06	1.4E+06	1.3E+06
DOUGLAS FIR-SOUTH	1,350	925	850	500	675	975	550	250	1.4E+06	1.3E+06	1.2E+06	1.1E+06	1.1E+06	1.2E+06	1.1E+06	1.0E+06
EASTERN HEMLOCK-BALSAM FIR	1,250	775	575	350	450	675	375	175	1.2E+06	1.1E+06	1.1E+06	9.0E+05	9.0E+05	1.0E+06	9.0E+05	8.0E+05
EASTERN HEMLOCK-TAMARACK	1,250	775	575	350	450	675	375	175	1.2E+06	1.1E+06	1.1E+06	9.0E+05	9.0E+05	1.0E+06	9.0E+05	8.0E+05
EASTERN SOFTWOODS	1,250	775	575	350	450	675	375	175	1.2E+06	1.1E+06	1.1E+06	9.0E+05	9.0E+05	1.0E+06	9.0E+05	8.0E+05
EASTERN WHITE PINE	1,250	775	575	350	450	675	375	175	1.2E+06	1.1E+06	1.1E+06	9.0E+05	9.0E+05	1.0E+06	9.0E+05	8.0E+05
HEM-FIR	1,400	975	850	500	675	975	550	250	1.6E+06	1.5E+06	1.3E+06	1.2E+06	1.2E+06	1.3E+06	1.2E+06	1.1E+06
HEM-FIR (NORTH)	1,300	1,000	1,000	575	775	1,150	650	300	1.7E+06	1.6E+06	1.6E+06	1.4E+06	1.4E+06	1.5E+06	1.4E+06	1.3E+06
MIXED MAPLE	1,000	725	700	400	550	800	450	225	1.3E+06	1.2E+06	1.1E+06	1.0E+06	1.0E+06	1.1E+06	1.0E+06	9.0E+05

Species	Allowable Flexural Strength (psi)								Modulus of Elasticity (psi)							
	Select Structural	**No. 1**	**No. 2**	**No. 3**	**Stud**	**Con-struction**	**Stand-ard**	**Utility**	**Select Structural**	**No. 1**	**No. 2**	**No. 3**	**Stud**	**Con-struction**	**Stand -ard**	**Utility**
MIXED OAK	1,150	825	800	475	625	925	525	250	1.1E+06	1.0E+06	9.0E+05	8.0E+05	8.0E+05	9.0E+05	8.0E+05	8.0E+05
NORTHERN RED OAK	1,400	1,000	975	550	750	1,100	625	300	1.4E+06	1.4E+06	1.3E+06	1.2E+06	1.2E+06	1.2E+06	1.1E+06	1.0E+06
NORTHERN SPECIES	975	625	625	350	475	700	400	175	1.1E+06	1.1E+06	1.1E+06	1.0E+06	1.0E+06	1.0E+06	9.0E+05	900,000
NORTHERN WHITE CEDAR	775	575	550	325	425	625	350	175	8.0E+05	7.0E+05	7.0E+05	6.0E+05	6.0E+05	7.0E+05	6.0E+05	6.0E+05
RED MAPLE	1,300	925	900	525	700	1,050	575	275	1.7E+06	1.6E+06	1.5E+06	1.3E+06	1.3E+06	1.4E+06	1.3E+06	1.2E+06
RED OAK	1,150	825	800	475	625	925	525	250	1.4E+06	1.3E+06	1.2E+06	1.1E+06	1.1E+06	1.2E+06	1.1E+06	1.0E+06
REDWOOD	1,350	975	925	525	575	825	450	225	1.4E+06	1.3E+06	1.2E+06	1.1E+06	9.0E+05	9.0E+05	9.0E+05	8.0E+05
SOUTHERN PINE	2,850	1,850	1,500	850	850	1,100	625	300	1.8E+06	1.7E+06	1.6E+06	1.4E+06	1.4E+06	1.5E+06	1.8E+06	1.7E+06
SOUTHERN PINE (MIXED)	2,050	1,450	1,300	750	750	1,000	550	275	1.6E+06	1.5E+06	1.4E+06	1.2E+06	1.2E+06	1.3E+06	1.6E+06	1.5E+06
SPRUCE-PINE-FIR	1,250	875	875	500	675	1,000	550	275	1.5E+06	1.4E+06	1.4E+06	1.2E+06	1.2E+06	1.3E+06	1.2E+06	1.1E+06
SPRUCE-PINE-FIR (SOUTH)	1,300	875	775	450	600	875	500	225	1.3E+06	1.2E+06	1.1E+06	1.0E+06	1.0E+06	1.0E+06	9.0E+05	9.0E+05
WESTERN CEDARS	1,000	725	700	400	550	800	450	225	1.1E+06	1.0E+06	1.0E+06	9.0E+05	9.0E+05	9.0E+05	8.0E+05	8.0E+05
WESTERN WOODS	900	675	675	375	525	775	425	200	1.2E+06	1.1E+06	1.0E+06	9.0E+05	9.0E+05	1.0E+06	9.0E+05	8.0E+05
WHITE OAK	1,200	875	850	475	650	950	525	250	1.1E+06	1.0E+06	9.0E+05	8.0E+05	8.0E+05	9.0E+05	8.0E+05	8.0E+05
YELLOW CEDAR	1200	800	800	475	625	925	525	250	1.6E+06	1.4E+06	1.4E+06	1.2E+06	1.2E+06	1.3E+06	1.2E+06	1.1E+06
YELLOW POPLAR	1,000	725	700	400	550	800	450	200	1.5E+06	1.4E+06	1.3E+06	1.2E+06	1.2E+06	1.3E+06	1.1E+06	1.1E+06

Input for Reinforced Concrete Columns

Item	Explanation
Shear Response Type	• This component is intended for perimeter ground level reinforced concrete columns subject to close-in explosive loading. Flexural response is assumed until the reactions from the column exceed the column shear capacity. Shear yielding with limited ductility based on available data is then assumed. The columns are analyzed in CEDAW to determine possible failure (i.e., a VLLOP) and therefore potential progressive collapse of supported components. Progressive collapse should be assumed unless it can be shown by analysis or test data that progressive collapse is unlikely for a given case of column failure.
Spacing, B	• Input the width of the building area that effectively loads the column with the applied blast load. If wall cladding spans vertically between floors and does not load the columns, input the column width. If such cladding is rigid, some engineering judgment may be required to estimate the width of the cladding that will effectively load the column. This is also true when the column supports low strength cladding. This recognizes that when the cladding is much less blast resistant than the column, it only transfers a portion of the blast load applied to the full tributary wall area supported by the column before failing when subject to blast loads high enough to cause column failure.
Reinforcing Shear Steel Yield Strength Dynamic and Static Strength Increase Factors	• Default values of 1.1 for the static increase factor and 1.17 for the dynamic increase factor are used in CEDAW, as recommended in TM 5-1300. See UFC 3-340-01 Chapter 4 for more detailed information as a dynamic increase factor as a function of strain-rate.
Distance of Cover to Center of Bars, d_c	• Input the typical distance from the face of concrete to the center of the longitudinal steel. This is typically near 2 inches.
Supported Weight, w	• Input the weight of cladding material assumed to load the column. If the cladding spans vertically between floors, this will be zero unless the cladding spans in front of the column so that blast load causes the cladding to bear against the column. In this case, input the cladding weight.
Spacing of shear steel, bs:	• Enter the vertical spacing between stirrups or ties placed around the longitudinal column reinforcing. Note that spacing must be less than one-half the distance from the loaded face to the longitudinal rebar along the unloaded face (i.e., less than one-half the depth to the positive moment rebar) in order for the shear steel to contribute to the column shear strength. If this is not the case, leave the shear steel area and spacing input cells blank.
Concrete Shear Strength Dynamic Increase Factor (CTDIF)	• The concrete shear strength is calculated $2(f'c*SIF)^{0.5}*CTDIF$, where f'c is the input compressive strength, the SIF is a 1.1 static increase factor, and the CTDIF is a dynamic increase factor based on the input scaled standoff. The SPAN32 computer code distributed by the U.S. Army Corps of Engineers was run with typical reinforced concrete columns subject to close-in blast loads from a range of scaled standoffs and conservative CTDIF calculated by SPAN32 for a variety of typical column dimensions and charge weights are used in CEDAW as follows where Z= scaled standoff: For $Z < 1$ ft/lb$^{1/3}$ DIF=2.1 For $1 < Z < 2.3$ ft/lb$^{1/3}$ DIF=1.7 For $Z > 2.3$ ft/lb$^{1/3}$ DIF=1.35 It is very rare that a VLLOP is calculated for a column when Z > 3 so the fact that the DIF can get closer to 1.0 at higher Z is not usually relevant.

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Input for Steel Columns Subject to Connection Failure

Item	Explanation
Connection Failure Response Type	• This component is intended for perimeter ground level steel columns subject to close-in explosive loading where the column connections are in shear at the bottom, due to a shear plane through the anchor bolts, or at the top of the ground floor. Flexural response is assumed until the reactions from the column exceed the ultimate bolt capacity of the connection. Ground floor columns can be analyzed in CEDAW to determine possible failure (i.e., a VLLOP) and therefore potential progressive collapse of supported components. Progressive collapse should be assumed unless it can be shown by analysis or test data that progressive collapse is unlikely for a given case of column failure.
Spacing, B	• Input the width of the building area that effectively loads the column with the applied blast load. If wall cladding spans vertically between floors and does not load the columns, input the column loaded width or depth depending on the column orientation (i.e., dimension parallel to blast loaded surface) as shown below on the input sheet. If such cladding is rigid, some engineering judgment may be required to estimate the width of the cladding that will effectively load the column. This is also true when the column supports low strength cladding. In the case of a typical pre-engineered building, approximately 20% of the tributary wall width may be a good assumption for the input spacing. This recognizes that the cladding is much less blast resistant than the column and only transfers a portion of the blast load applied to the full tributary wall area supported by the column before failing when subject to blast loads high enough to cause column failure.
Supported Weight	• Input the weight of cladding material assumed to load the column. If the cladding spans vertically between floors, this will be zero unless the cladding spans in front of the column so that blast load causes the cladding to bear against the column. In this case, input the cladding weight.
Number of bolts and Bolt diameter	• Input the number of anchor bolts and anchor bolt diameter unless there is no shear plane through the anchor bolts. In this case, input the bolts at the top of the ground level column span that are in shear.
Connection Steel Type	• Select the type of steel for the bolts in shear. Typical anchor bolts are A307 steel. Typical structural bolts are A325 steel.
Bolt Dynamic Strength Increase Factors, DIF	• The ultimate dynamic yield strength used for shear in the anchor bolts is $f_{dy}=f_y*$ DIF. The ultimate dynamic shear strength is 60% of the ultimate dynamic yield strength. A low dynamic increase factor is recommended for high strength steels, such as most bolts. Typically, the DIF should be 1.05.

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Limitations of CEDAW Methodology

Item	Explanation
General Usage	• CEDAW is intended for use as a siting tool or preliminary analysis tool for buildings with conventional construction. More accurate estimates of component blast damage and LOP can be obtained by using a time-stepping analysis of the component response to the blast load. • CEDAW assumes the ultimate capacity of all components except columns are controlled by flexural and/or tension membrane response. This assumes that shear failure and connection failure will not control component failure, which is usually a reasonable assumption for a preliminary siting study of conventionally constructed buildings. More careful analysis to verify this assumption may be necessary.
Accuracy of P-i diagrams	• The CEDAW P-i curves are curve-fits to results to Single-Degree-of-Freedom (SDOF) time-stepping analyses for typical components and are not perfect curve-fits for all cases. The curve-fits are most approximate for cases with tension membrane and for unreinforced masonry components due to a less accurate method used to develop scaling parameters for the P-i curve-fitting method for these cases. • The response parameter terms that are part of the SDOF analyses used to define the boundaries between LOP on the P-i diagrams are based in part on representative explosive test data on structural components. There is scatter in this data, where as much as 10% to 20% of the data can be unconservative compared to the defined boundaries the some of the LOP for some of the component types. All the defined boundaries have at least as many points that are conservative compared to unconservative and the very large majority are either very consistent with the data or generally quite conservative to data that has significant scatter. The largest databases and largest amount of scatter among data points occur for unreinforced masonry type components, especially those with both wall thicknesses greater than 8 inches and high axial loads.
Use of CEDAW P-i Diagrams for Primary Reinforced Concrete and Steel Framing Components	• Available data from typical reinforced concrete and steel frame buildings indicate that heavy framing components are almost always much more resistant to blast loads than the surrounding wall cladding components in conventional buildings. Failed cladding will typically cause severe injury to building occupants and will therefore be the controlling factor determining the LOP provided to building occupants. However, ground floor column failure can cause progressive collapse to all supported components above and this can lead to very severe injuries throughout the collapse area. Therefore, CEDAW includes an approximate method for estimating column failure, which is reported as a VLLOP for the column. • The P-i diagrams in CEDAW were developed to match available data that was almost exclusively from relatively low strength structural components typically used for conventional design. In particular, all reinforced concrete data slab/beam data was from slabs with reinforcing ratios less than 1%. All steel beam data was from cold-formed steel girts and purlins. Therefore, there was no data from hot-rolled steel beams or reinforced concrete beams. All data for reinforced concrete columns was from large, close-in loads causing response controlled by the column diagonal shear capacity. • The P-i diagrams for hot-rolled steel beams are based on assumptions between component response levels and level of protection that are considered most valid for steel beams used as secondary framing members and may be unconservative for deep, heavy girders and columns in flexure- especially those with fixed end conditions. • The P-i diagrams for reinforced concrete beams are based on assumptions between component response levels and level of protection that are considered most valid for singly reinforced beams with reinforcing ratios less than 1% and symmetrically reinforced beams with stirrups throughout maximum moment regions. The same limitations apply for reinforced concrete slabs.

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Item	Explanation
Interpretation of results	• The methods used to determine the exact boundaries on the P-i diagrams between each LOP involve a significant amount of engineering judgment. This is in part due to the qualitative nature of the LOP definitions and due to scatter in data used to help define the boundaries for the LOP on the P-i curves. This should be kept in mind when the results show that the input blast load is near a boundary of a LOP on a P-i diagram.
Usage of CEDAW for scaled standoffs less than 3.0 ft/lb$^{1/3}$	• The methods used to develop the P-i curves in CEDAW assume a spatially uniform blast load on the component. This assumption becomes less accurate as the scaled standoff (Z) becomes less than 3.0 ft/lb$^{1/3}$. As Z approaches 1.0 ft/lb$^{1/3}$, other failure modes not considered in CEDAW can control, such as localized shear and/or spall failure.