PDC-TR 06-08
23 April 2025

US Army Corps
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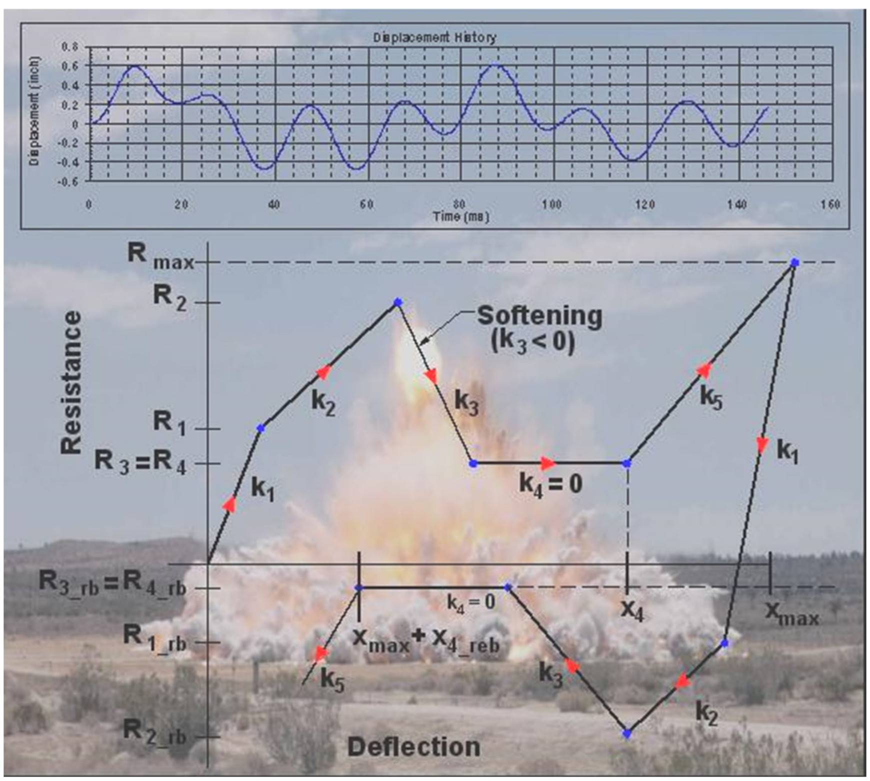
U.S. ARMY CORPS OF ENGINEERS PROTECTIVE DESIGN CENTER TECHNICAL REPORT

SINGLE DEGREE OF FREEDOM STRUCTURAL RESPONSE LIMITS



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Indicate the Military Department Preparing Activity responsible for the document.

U.S. ARMY CORPS OF ENGINEERS

This technical report supersedes PDC-TR 06-08, Revision 1, dated January 2008.

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FOREWORD

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TABLE OF CONTENTS

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CHAPTER 1 INTRODUCTION

1-1 BACKGROUND.

UFC 4-010-01 (*DoD Minimum Antiterrorism Standards for Buildings*) provides baseline minimum levels of protection with which all DoD inhabited buildings must comply. The process in UFC 4-020-01 (*DoD Security Engineering Facilities Planning Manual*) may determine that a building requires a higher level of protection or must address threats or aggressors beyond those considered in UFC 4-010-01, in which cases the structure must be specifically analyzed for blast loading. Structural engineers need guidance for the design of buildings required to resist blast overpressures associated with terrorist explosive threats where higher levels of protection are required and/or where more severe threats need to be considered.

The prevailing method used by the DoD to design structures to resist blast overpressures associated with terrorist explosive threats is single degree of freedom (SDOF) analysis. The SDOF analysis methodology is described in UFC 3-340-01 (*Design and Analysis of Hardened Structures to Conventional Weapons Effects*), UFC 3-340-02 (*Structures to Resist the Effects of Accidental Explosions*), and other non-government references. The SDOF analysis process has been automated in the SDOF Blast Effects Design Software (SBEDS) tool for many structural component types. Specific details concerning the assumptions and analytical methodology used in SBEDS are provided in PDC-TR 06-01 (*Methodology Manual for the Single-Degree-of-Freedom Blast Effects Design Spreadsheets (SBEDS)*).

1-2 PURPOSE AND SCOPE.

The U.S. Army Corps of Engineers Protective Design Center (PDC) has reviewed available test reports and consulted with technical experts in the field of blast design to develop the response limits defined in this technical report. Limits to band the four levels of protection defined in UFC 4-020-01 are provided. These limits should be used in the design of facilities required to resist blast overpressures associated with terrorist explosive threats. This report aims to provide structural engineers responsible for performing SDOF analyses with basic information concerning the application and derivation of these response limits.

1-3 REISSUES AND CANCELS.

This technical report supersedes PDC-TR 06-08, Revision 1, dated 7 January 2008.

1-4 APPLICABILITY.

This technical report applies to SDOF analyses involving structural and non-structural components requiring blast design protection. For components that are not included, testing shall be conducted to support the required performance.

1-5 GENERAL BUILDING REQUIREMENTS.

Not Applicable.

1-6 CYBERSECURITY.

Not Applicable.

1-7 COMMENTARY.

APPENDIX A contains commentary and background information concerning the response limits defined in this technical report.

1-8 GLOSSARY.

APPENDIX B contains acronyms, abbreviations, and terms.

1-9 REFERENCES.

CHAPTER 6 contains a list of references used in the body (main provisions) of this document. The publication date of the code or standard is not included in this document. Unless otherwise specified, the most recent edition of the referenced code or standard applies.

The last section in APPENDIX A contains a list of references associated with the commentary section.

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CHAPTER 2 LEVELS OF PROTECTION

2-1 GENERAL.

DoD facilities requiring protection from terrorist explosive threats should provide the building level of protection (LOP) determined by the process defined in UFC 4-020-01. Potential building levels of protection are listed in Table 2-1, along with a description of the potential global damage the structure will sustain. Table 2-2 describes the hazards associated with doors and glazing for the various building levels of protection.

Table 2-1 Structural Damage Associated with Building Levels of Protection

Building LOP	Descriptions of Potential Overall Structural Damage
Very Low	Heavy Damage – The structure is on the onset of structural collapse. Progressive collapse is unlikely. The interior space around the damaged area is unusable for its intended purpose.
Low	Moderate Damage – The structure damage will not be economically repairable. Progressive collapse will not occur. The interior space around the damaged area is unusable for its intended purpose.
Medium	Minor Damage – The structure damage will be economically repairable. The interior space around the damaged area can be used and is fully functional after cleanup and repairs.
High	Minimal Damage – The structure will have no permanent deformations. The structure is immediately operable.

Table 2-2 Door and Glazing Hazards Associated with Building Levels of Protection

Building Level of Protection [1]	Description of Door or Glazing Hazard
Very Low	**Door**: "Category IV" damage level - The door becomes dislodged from its frame, and may fall to the ground, but it does not become a flying debris hazard.
	Glazing: "Low" hazard rating - The glazing will fracture, come out of the frame and it is likely to be propelled into the building, with the potential to cause serious injuries.
Low	**Door**: "Category III" damage level - The door experiences non-catastrophic failure but may have permanent deformation and may be inoperable.
	Glazing: "Very Low" hazard rating - The glazing will fracture and potentially come out of the frame, but at reduced velocity, it does not present a significant injury hazard.
Medium	**Door**: "Category II" damage level - The door is openable, but the door panel experiences measurable, permanent deformation.
	Glazing: "Minimal" and "No" hazard rating - The glazing will fracture, remain in the frame, and result in a minimal hazard consisting of glass dust and slivers.
High	**Door**: "Category I" damage level - The door is unchanged (no permanent deformation) and it is fully operable.
	Glazing: "No Break" hazard rating - The innermost surface of the glazing will not break.

1 – Mapping between LOP and relevant ASTM documents (F2247, F2927, and F2912) provided in commentary Table A-1.

2-2 COMPONENT DAMAGE.

Building LOP is based on the damage expected to be incurred by individual structural components. Components are generally categorized as primary structural, secondary structural, or non-structural. These categories are described in Table 2-3. Component damage can be assigned to one of the five regimes shown in Table 2-4.

Table 2-3 Component Descriptions

Component	Description
Primary Structural[1]	Structural components whose loss would affect other supported components and could affect the overall structural stability of the building in the area of loss. Examples of primary structural components include columns, girders, and other primary framing components directly or indirectly supporting other structural or non-structural components, and any load-bearing structural components required to maintain structural stability, such as load-bearing walls.
Secondary Structural[1]	A secondary structural component is supported by a primary structural component. Examples of secondary structural components include metal panels and standing seam roofs. Members that are essential to the vertical stability of the building, providing gravity loading resistance, are considered primary structural components. If failure of a component or group of components would cause destabilization or progressive collapse of the structure prior to post-blast evacuation it shall be analyzed as a primary structural component.
Non-Structural	Non-structural components whose loss would have little effect on the overall structural stability of the building in the area of loss. Examples of non-structural components include interior non-load bearing walls and architectural items (e.g., doors) attached to building structural components.

1 – Refer to definitions of primary structural frame and secondary structural members as provided by the latest version of the International Building Code for further clarification.

Table 2-4 Component Damage Levels

Component Damage Level	Description of Component Damage
Blowout[1]	The component is overwhelmed by the blast load, causing significant debris velocities.
Hazardous Failure	The component has failed. Debris velocities range from insignificant to moderate.
Heavy Damage	The component has not failed, but it has significant permanent deflections, causing it to be unrepairable.
Moderate Damage	The component has some permanent deflection. If necessary, it is generally repairable, although replacement may be more economical and aesthetic.
Superficial Damage[2]	The component has no visible permanent damage.

1 – This is not a level of protection and should never be a design goal. It only defines a realm of more severe structural response.
2 – For the purpose of defining Superficial Damage for concrete and masonry components, “no visible permanent damage” does not include hairline crack damage; some cracking is always expected for these types of materials, even under conventional working loads.

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CHAPTER 3 COMPONENT DAMAGE AND BUILDING LOP RELATIONSHIP

When determining the building LOP, the consequence of damage to primary structural components is weighted more heavily than that of secondary structural and non-structural components. Table 3-1 shows the allowable damage for the various component categories for each building LOP.

The damage assigned to each component category (i.e., primary, secondary, or non-structural) is based on the component in that category that is expected to sustain the most damage. For example, if one component is expected to sustain heavy damage and all other primary members are expected to sustain moderate damage, the damage for primary components is classified as heavy (i.e., the damage associated with the individual component sustaining the most damage).

The building LOP is based on the LOP associated with the component category sustaining the most damage. For example, if primary structural and non-structural components are expected to sustain only superficial damage but secondary structural components are expected to sustain heavy damage, the building LOP would be “low” (i.e., the building LOP associated with secondary structural components sustaining heavy damage).

Table 3-1 Building LOP – Component Damage Relationship

Building Level of Protection	Component Damage	
	Primary Component	Secondary or Non-Structural Component
Very Low	Heavy Damage	Hazardous Failure
Low	Moderate Damage	Heavy Damage
Medium	Superficial Damage	Moderate Damage
High	Superficial Damage	Superficial Damage

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CHAPTER 4 RESPONSE LIMITS FOR COMMON COMPONENTS

4-1 GENERAL.

Specific values for the maximum support rotation (θ) and/or ductility ratio (μ) associated with the boundaries between the various component damage levels are provided for common components in this chapter.

Important information for the application of response limits is contained in the text of each section as well as table footnotes; the user is urged to carefully read both the text and the footnotes.

4-2 DESIGN CONSIDERATIONS.

4-2.1 SDOF Design Process.

The following steps are followed when using SDOF analysis to design a blast-loaded component:

a. A trial component is selected and the equivalent SDOF system is determined.

b. Blast load is calculated based on the charge weight, distance between the charge and the component being considered (standoff distance), and the orientation of the component with respect to the charge.

c. The maximum deflection of the SDOF system is calculated and used to calculate the corresponding maximum support rotation (θ) and ductility ratio (μ).

d. The calculated θ and/or μ are then compared to the response limits defined in this chapter to determine the level of protection the component would provide for the structure.

e. If an acceptable level of protection is provided, the design of the component is finalized (i.e., shear capacity is verified, residual axial capacity is checked, connections are designed, etc.); if not, another component is selected, and the process is restarted.

4-2.2 General Assumptions and Limitations of SDOF Approach.

SDOF analysis simplifies complex loading, material, and system phenomena. It does not address localized physical phenomena. The assumptions associated with the development of this technical report are identified below.

- The blast load is far-field and oriented relative to the component so as to result in a pressure history load function that can be idealized as uniform over the tributary area of the component. Typically, the far-field criterion is met with a scaled distance of approximately 3 ft/lb$^{1/3}$, and the uniformity criterion is met when the maximum and minimum pressures across the span of the component are within 25% of each other.

- The component is not subjected to localized damage resulting from primary or secondary fragments.
- The component's response is assumed to be based on its fundamental flexural mode. This implies that for inelastic response, the ultimate capacity of all component types is controlled by the flexural capacity of the component, and not the shear or buckling capacity, except as noted.
- The component's connections do not fail.
- The component does not fail in shear.
- The resistance function used in SDOF analysis is generated in accordance with the methodology described in PDC-TR 06-01 or PDC-TR 18-02.

4-2.3 Components in Combined Flexure and Axial Loading.

4-2.3.1 Components in Compression.

For the tables contained in this technical report, the response limits for components in flexure shall be used when the axial compressive load due to gravity is too low to require consideration of combined axial and flexural loads. The response limits for components in "Combined Flexure and Compression" should be used when the axial compressive load demand is greater than 10% of the axial compressive load capacity. The axial compressive load demand should be based on gravity loads (i.e., exclude dynamic axial loads) as shown in Equation 4-1. The axial compressive load capacity is a nominal capacity accounting for both strength and stability mechanisms, should include any applicable static increase factors (SIFs) but exclude dynamic increase factors (DIFs), and should include relevant safety factors stemming from standard design codes (i.e., ϕ for LRFD, Ω for ASD).

4-2.3.2 Components in Tension.

For components under combined flexure and axial tension, the response limits for components in flexure should be used. When performing a SDOF analysis, the component's resistance function may need to be modified to account for the altered flexural capacity stemming from the concurrent tensile loading.

4-2.3.3 Load Combinations.

When determining the axial load demand, Equation 4-1 should be used where the gravity loads may include factored dead (D), live (L), roof live (L_r), snow (S), and rain (R) loads. The load factors generating the most severe load case shall be used.

Equation 4-1. Load Combination for Gravity Loads

$$(0.9\ or\ 1.2)D + (0.0\ or\ 0.5)L\ + (0.0\ or\ 0.2)(L_r\ or\ 0.7S\ or\ R)$$

When evaluating a blast event the load combination shown in Equation 4-2 should be used. The load factors generating the most severe load case shall be used.

Equation 4-2. Load Combination During Blast Event

$$(0.9\ or\ 1.2)D\ +\ A_k\ + (0.0\ or\ 0.5)L\ + (0.0\ or\ 0.2)(L_r\ or\ 0.7S\ or\ R)$$

where A_k is the load or load effect resulting from the blast event.

For Equation 4-1 and Equation 4-2:

- Components in buildings for which the actual dead load effect can be established with a high degree of certainty should utilize a load factor of 1.0 for *D*.
- Elements for which risk assessment indicates that the full live load effect is likely to occur simultaneously with the blast effect should use bounding load factors of 0.0 and 1.0 (instead of 0.0 and 0.5) for *L*.

4-2.3.4 Residual Axial Capacity Evaluation

Following the blast event, sufficient residual axial capacity is required to prevent structural collapse. For components: (A) subjected to concurrent blast and compression loading, (B) whose axial gravity load demand is greater than 30% of the axial compressive load capacity, and (C) where the expected damage level is Moderate or worse, a residual axial capacity check is required. The calculated residual axial capacity should account for the potential loss of cross-sectional area (e.g., concrete cover) stemming from the blast event, permanent deformations (e.g., P-Delta), and exclude dynamic strength increase factors (DIFs). The load combination for the evaluation of residual axial capacity is shown in Equation 4-1. The load factors generating the most severe load case shall be used.

4-2.3.5 Supported Members Exposed to Blast Loading.

When supported members are exposed to blast loading, their reactions may generate axial (and other) loads in supporting column/wall elements (e.g., floor beams subjected to direct blast loads may cause dynamic axial loads in supporting columns). Generally, a detailed analysis is necessary to accurately determine an appropriate dynamic axial load deriving from these supported members. In lieu of a detailed analysis or other engineering justification, the ultimate resistance (r_u) of the supported member may be used to calculate the axial force demand acting concurrently with the blast loading on the supporting member. Consideration should be given to the orientation of the applied axial load (i.e., net upwards or net downwards).

4-2.3.6 P-Delta Effects.

If it is determined that axial load effects should be considered when performing a SDOF assessment of a component, P-Delta effects should be considered in the SDOF analysis.

4-2.4 Component Rebound Response.

The response limits defined in this technical report are for the movement of the component in the same direction as the applied positive phase of the blast load, i.e., inbound response. Rebound response occurs after the component reaches maximum deflection in the inbound direction and begins moving back in the opposite direction of the applied positive phase of the blast load. The assessment of rebound shall be in accordance with the following:

For primary structural components:

- The component's rebound response shall be considered. Response limits for rebound shall be the same as the inbound response limits. In addition, the component's connections shall be designed to resist inbound and rebound reaction forces.

For secondary structural and non-structural components, the rebound is treated differently depending on the building LOP:

- Very Low and Low Levels of Protection – Rebound response is not considered as component failure in rebound would typically result in the failed component landing outside the building envelope, thus posing minimal hazard to the occupants.
- Medium and High Levels of Protection – The component's rebound response shall be considered. Response limits for rebound shall be the same as the inbound response limits. In addition, the component's connections shall be designed to resist the inbound and rebound reaction forces.

The rebound requirements listed above are applicable unless the Authority Having Jurisdiction (AHJ) has different requirements, in which case, the AHJ's requirements shall apply.

4-3 REINFORCED CONCRETE.

The response limits for the damage level boundaries of reinforced concrete components are shown in Table 4-1. The reinforcement index (RI) is calculated using Equation 4-3. A primary assumption, which is listed in Section 4-2.2, is that the component's response is governed by its fundamental flexural mode. Deep beams (i.e., as defined in ACI 318), which are unlikely to respond in a flexural mode, are not explicitly covered by these response limits.

Equation 4-3. Reinforcement Index

$$RI = \left[\frac{A_s f_s + A_p f_p}{b d f_c}\right] \cdot 100$$

where:

RI = reinforcement index (%)

A_s = area of conventional reinforcing steel including welded wire reinforcement in tension within width "b"
A_p = area of prestressed or post-tensioned reinforcing steel in tension within width "b"
b = width of compression face of member
d = distance from extreme compression fiber to centroid of tensile reinforcing steel
f_c = average expected unconfined compressive strength of concrete (includes applicable SIF)
f_s = average expected yield strength of reinforcing steel (includes applicable SIF)
f_p = average expected yield strength of prestressed or post-tensioned steel (includes applicable SIF)
Note: Dynamic increase factors (DIF) should not be used to calculate RI since response limits are calibrated to RI of blast tested components calculated without DIF.

Table 4-1 only shows response limits for solid concrete cross sections. This is not meant to discourage the use of insulated, or "sandwich", reinforced concrete panels for blast resistant design. Insulated reinforced concrete panels are relatively new and there is currently insufficient blast test data available to determine response limits for the full range of relevant parameters for this panel type, including the insulation thickness, shear connector properties, and percentage of composite response between the reinforced wythes of the panels. Blast tests have been conducted on insulated panels with a limited range of these parameters. These tests, and the observed relationship between maximum support rotation and observed component damage level, are discussed in the commentary. See Appendix A and discussion for this figure that describes limitations for the plotted test data.

Table 4-1 Response Limits for Flexural Response of Reinforced Concrete Components

<table>
<tr><th rowspan="2">Solid Cross Section Type</th><th rowspan="2">RI [1]</th><th>Superficial Damage</th><th>Moderate Damage</th><th>Heavy Damage</th><th>Hazardous Failure</th></tr>
<tr><th>θ</th><th>θ</th><th>θ</th><th>θ</th></tr>
<tr><td rowspan="2">Conventional reinforcement, longitudinal bars at each face [7], no shear reinforcement</td><td>$RI \leq 3\%$</td><td>0.5°</td><td>4°</td><td>8°</td><td>12°</td></tr>
<tr><td>$RI > 3\%$</td><td>0.5°</td><td>$\Theta = 6.4(RI)^{-0.43}$</td><td>$\Theta = 12.8(RI)^{-0.43}$</td><td>$\Theta = 18.2(RI)^{-0.38}$</td></tr>
<tr><td rowspan="3">Conventional reinforcement, longitudinal bars at tensile face only, no shear reinforcement</td><td>$RI \leq 5\%$</td><td>0.5°</td><td>2.5°</td><td>5°</td><td>7°</td></tr>
<tr><td>$5\% \leq RI < 15\%$</td><td>0.5°</td><td>$\Theta = 3.5(RI)^{-0.20}$</td><td rowspan="2">$\Theta = 7.9(RI)^{-0.28}$</td><td rowspan="2">$\Theta = 10.4(RI)^{-0.25}$</td></tr>
<tr><td>$RI > 15\%$</td><td>0.5°</td><td>2°</td></tr>
<tr><td rowspan="2">Conventional reinforcement, longitudinal bars at each face [7], shear reinforcement [6]</td><td>$RI \leq 5\%$</td><td>0.5°</td><td>5°</td><td>8°</td><td>12°</td></tr>
<tr><td>$RI > 5\%$</td><td>0.5°</td><td>$\Theta = 7.9(RI)^{-0.28}$</td><td>$\Theta = 10.4\ (RI)^{-0.16}$</td><td>$\Theta = 14.1\ (RI)^{-0.10}$</td></tr>
<tr><td rowspan="2">All cross sections with prestressed reinforcement</td><td>$RI \leq 5\%$</td><td>0.5°</td><td>3°</td><td>5°</td><td>7°</td></tr>
<tr><td>$RI > 5\%$</td><td>0.5°</td><td>$\Theta = 8.0(RI)^{-0.61}$</td><td>$\Theta = 14.7(RI)^{-0.67}$</td><td>$\Theta = 21.6(RI)^{-0.7}$</td></tr>
<tr><td>All cross sections with welded-wire reinforcement [3]</td><td>All</td><td>0.5°</td><td>2°</td><td>2.5°</td><td>4°</td></tr>
<tr><td rowspan="2">With tension membrane (L/h>=5)</td><td>Conventional reinforcement</td><td>0.5°</td><td>Note 4</td><td>12°</td><td>20°</td></tr>
<tr><td>Prestressed</td><td>0.5°</td><td>Note 4</td><td>6°</td><td>10°</td></tr>
<tr><td>Combined Flexure & Compression [8]</td><td>All</td><td>0.5°</td><td>Note 5</td><td>Note 2</td><td>Note 2</td></tr>
</table>

1 – Reinforcement index (RI) is calculated as shown in Equation 4-3. All equations in this table are for RI expressed as a percentage (e.g., RI=3.0% is entered as 3.0). Reinforced concrete components responding primarily in flexure must be designed for yielding of steel reinforcement prior to concrete crushing in the maximum moment regions.
2 – Response limits for heavy damage and hazardous failure are equal to those for moderate damage if a component has an axial compressive load demand that is greater than 10% of the axial compressive load capacity.
3 – This category only applies for components in which welded-wire reinforcement (WWR) constitutes more than 30% of the reinforcement index. The amount of WWR should be minimized as much as practical for blast design that is based on all other categories since WWR is significantly less ductile.
4 – Tension membrane does not affect response limits for Moderate damage. Use same response limits as if component does not have tension membrane response.
5 – Same response limits as if component does not have axial load.
6 – Shear reinforcement is defined herein as transverse reinforcement spaced not greater than half the least cross-sectional dimension of the component.
7 – Compression face reinforcement area at least equal to one-third of tensile face reinforcement area.
8 – This response type applies if a component has a compressive axial load exceeding the threshold listed in Section 4-2.3.1.

4-4 MASONRY.

The response limits for the damage level boundaries of reinforced and unreinforced masonry components are shown in Table 4-2 and Table 4-3, respectively. Prestressed masonry is not included in these response limits due to a lack of applicable test data. The response limits for reinforced masonry are based on the reinforcement index (RI), which is calculated using Equation 4-4.

Equation 4-4. Reinforcement Index for Reinforced Masonry

$$RI = \left[\frac{A_s f_s}{b d f_m}\right] \cdot 100$$

where:

RI = reinforcement index (%)
A_s = area of reinforcing steel in tension within width “b”
b = width of compression face of member. Limited to the effective compressive width per latest TMS 402.
d = distance from extreme compression fiber to centroid of tensile reinforcing steel
f_m = average expected compressive strength of masonry prism (includes applicable SIF)
f_s = average expected yield strength of reinforcing steel (includes applicable SIF)
Note: Dynamic increase factors (DIF) should not be used to calculate RI since response limits are calibrated to RI of blast tested components calculated without DIF.

Unreinforced masonry is not allowed for new designs per UFC 4-010-01. The response limits for unreinforced masonry herein are applicable only when used in a dynamic analysis where the wall resistance is controlled by brittle flexural response and axial load arching or compression membrane response. Do not use a dynamic analysis method with an elastic-plastic resistance vs. deflection relationship for unreinforced masonry with the response limits in this section, as this will lead to an unconservative calculation of the damage level of the component. On the other hand, it is appropriate to use a dynamic analysis method with elastic-plastic resistance vs. deflection relationship for reinforced masonry components.

Table 4-2 Response Limits for Flexural Response of Reinforced Masonry Components

Response Type	Reinf. Index [1]	Maximum Rebar Spacing (in)	Superficial Damage	Moderate Damage	Heavy Damage	Hazardous Failure
			θ	θ	θ	θ
Flexure [4]	*RI* ≤ 10%	32 (48) [3]	0.5°	4°	12°	17°
	RI > 10%		0.5°	$12.5(RI)^{-0.5}$	$23.5(RI)^{-0.29}$	$30.3(RI)^{-0.25}$
	All RI	32<s≤48	0.5°	2°	6°	9°
Combined Flexure & Compression [2,4]	*RI* ≤ 10%	32 (48) [3]	0.5°	4°	4°	4°
	RI > 10%		0.5°	$12.5(RI)^{-0.5}$	$12.5(RI)^{-0.5}$	$12.5(RI)^{-0.5}$
	All RI	32<s≤48	0.5°	2°	2°	2°

1 – Reinforcement index (RI) is calculated as shown in Equation 4-4. All equations in this table are for RI expressed as a percentage (e.g., RI=3.0% is entered as 3.0). Reinforced masonry components responding primarily in flexure must be designed for yielding of steel reinforcement prior to masonry crushing in the maximum moment regions.
2 – This response type applies if a component has a compressive axial load exceeding the threshold listed in Section 4-2.3.1.
3 – These response limits also apply for rebar spacing up to 48 inches when all cells are grouted (i.e., solid masonry wall).
4 – For the case of CMU block with ungrouted cells, these response limits are applicable when the peak applied blast pressure is 40 psi or less. Blast testing shows that the face cells may shatter at higher blast pressures.

Table 4-3 Response Limits for Flexural Response of Unreinforced Masonry Components

Response Type	Superficial Damage	Moderate Damage	Heavy Damage	Hazardous Failure
	μ	θ	θ	θ
Flexure [1,3]	1	1°	4°	8°
Combined Flexure & Compression [1,2,3]	1	1°	1°	1°

1 –For the case of CMU block with ungrouted cells, these response limits are applicable when the peak applied blast pressure is 40 psi or less. Blast testing shows that the face cells may shatter at higher blast pressures.
2 – This response type applies if a component has a compressive axial load exceeding the threshold listed in Section 4-2.3.1.
3 – The maximum wall deflection is limited to the wall thickness for all damage levels.

4-5 MASONRY WALLS RETROFITTED WITH DUCTILE POLYMER.

The response limits for the damage level boundaries of unreinforced masonry walls retrofitted with a ductile polymer are shown in Table 4-4. These response limits are based on blast tests where spray-on polyurea with an elongation of 89% or more per ASTM D412 was applied to unreinforced masonry walls with thicknesses no greater than 3/8 inch. This retrofit is intended for non-load bearing walls due to its relatively large yield deflection under lateral blast load. Therefore, no response limits are shown for combined flexure and compression.

This retrofit is typically only applied to the protected side of unreinforced masonry walls (i.e., the side in tension during blast loading). The rebound response of the wall depends on overspray of the polymer onto the supports so that the polymer can develop tension membrane response (to help prevent wall failure during rebound); a 6-inch overspray along each support is recommended based on high explosive and shock tube tests. This overspray also helps ensure that damage during the inbound response controls the overall damage level for the retrofitted wall.

The response limits in Table 4-4 are intended for walls that do not develop any arching or compression membrane response, so that blast resistance is only provided by ductile flexural response of the retrofitted wall. Limited blast test data shows that the response limits shown in Table 4-4 are conservative when arching or compression membrane response is present due to in-plane restraint by boundary conditions.

Table 4-4 Response Limits for Unreinforced Masonry Walls with a Ductile Polymer Retrofit

Response Type[1,2]	Superficial Damage	Moderate Damage	Heavy Damage	Hazardous Failure
	μ [3]	θ	θ	θ
Flexure	1	2°	8°	15°

1 – Polymer is a spray-on polyurea with an elongation of 89% or more per ASTM D412 and thickness no greater than 3/8 inch applied to the tension face of wall.
2 – Retrofit should not be used for load bearing walls.
3 – Ductility ratio is based on yielding of the polymer in maximum moment region of wall.

4-6 MASONRY WALLS RETROFITTED WITH UNBONDED MEMBRANE CATCH SYSTEM.

The response limits for unbonded membrane catch systems are defined by maximum permissible membrane strains for each damage level. These strains are dependent on the membrane material. These strain response limits are lower than the membrane material failure strains to implicitly account for strain concentrations that occur at the anchorage system, which are not calculated with simplified dynamic design methods (e.g., SDOF method). Static and blast tests show that unbonded membrane catch systems fail at the support anchorage.

The maximum permissible dynamic strain for each damage level, ε_m, is calculated with Equation 4-5, where μ' is a damage factor and ε'_f is the strain in a given membrane material exclusive of strain concentrations when failure may occur at the anchorage of the catch system (see Table 4-5 and Table 4-6). Note that ε'_f and ε_m include plastic strains. The ε'_f strains in Table 4-6 are back-calculated based on basic membrane theory from measured maximum deflections of membrane systems with anchorage systems similar to Figure 4-1 that are at or near failure occurring at the anchorage in static tests or have maximum observed damage without failure in available blast tests. Figure 4-1 shows an optimized anchorage system for unbonded membrane catch systems based on static testing of membrane systems. The development of the ε'_f strains in Table 4-6 and damage factors in Table 4-5 is discussed in the commentary. The response limits in this section are applicable to the membrane materials listed in Table 4-6 for catch systems with an anchorage as shown in Figure 4-1 (i.e., where the membrane material is clamped to the support with a bolted steel plate or similar component as stated in the figure). These response limits are also applicable for other anchorage systems that allow equal or higher calculated membrane strains to ε'_f in Table 4-6 at membrane failure.

Equation 4-5. Limit Maximum Strain for SDOF Analysis of Membrane Catch System

$$\varepsilon_m = \varepsilon'_f \mu'$$

where:

ε_m = maximum permissible membrane strain for a given damage level as calculated with simplified dynamic analysis (e.g., SDOF method) utilizing membrane theory
ε'_f = maximum strain in a given membrane material when failure may occur due to strain concentrations (i.e., higher strains) at anchorage system (See Table 4-6)
μ' = damage factor based on damage level from Table 4-5

Table 4-5 Damage Factors for Unbonded Membrane Catch System Retrofits

Response Type [1]		Superficial Damage	Moderate Damage	Heavy Damage	Hazardous Failure
		μ'	μ'	μ'	μ'
Masonry Wall with Unbonded Membrane Retrofit	Membrane	0.2	0.5	1.0	2.0

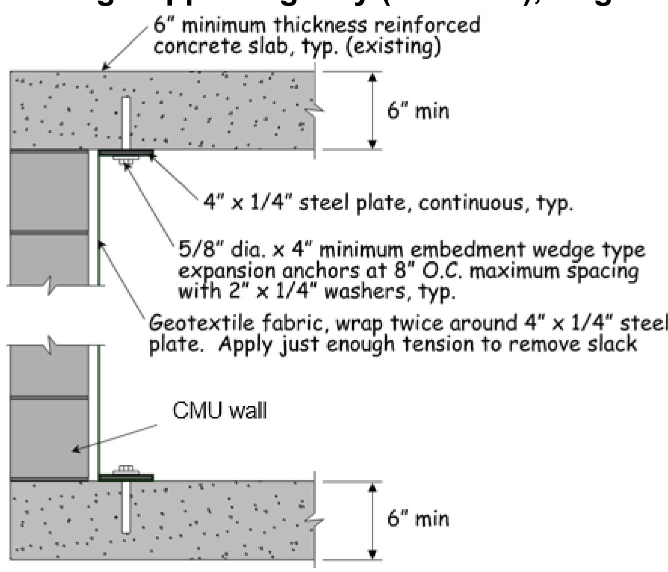
1 – Damage factors used in Equation 4-5 to determine maximum permissible dynamic membrane strain in catch system for given damage level.

Table 4-6 Material Dependent Failure Strains for SDOF Analysis of Unbonded Membrane Catch System Retrofits

Material	Material Failure Strain [1]	Strain at Failure in Catch System (ε'_f) [2,3]	Comments
Mild steel sheet	0.5	0.02	Sheet must be thinner than 18 gage thick
Polypropylene	0.16	0.07	See Note 4
Woven Geotextile	0.07	0.04	See Note 5

1 – This is an engineering failure strain shown only for reference.
2 – Calculated maximum strains (not including localized strain concentrations near connections) in membrane of catch system at or near failure based on static and blast tests (see commentary).
3 – Values for ε'_f are based on tests where membranes are connected to rigid supporting structure with minimum 0.25 inch thick steel clamping plates with smooth edges free of burs and 8 inch maximum bolt spacing (12 inches for polyurea membrane catch system). The bolts are sized to develop the maximum calculated membrane force in the catch system. The bolt diameter is a minimum 0.5 inches and holes through plate and membrane material are match drilled. (See Figure 4-1).
4 – Specific to woven polypropylene fabric similar to trade name Curv® with minimum elongation at failure of 0.16 in/in
5 – Woven geotextile with minimum 7% elongation at failure per ASTM D4632.

Figure 4-1 Anchorage System for Unbonded Membrane Retrofit (Air Force Civil Engineering Support Agency (AFCESA), August 2000)



4-7 MASONRY AND REINFORCED CONCRETE WALLS RETROFITTED WITH FRP.

The response limits for the damage level boundaries of masonry and concrete walls and panels retrofitted with FRP (fiber reinforced polymers) are shown in Table 4-7.

The FRP retrofit must be applied to both faces of the wall if the retrofit is intended to resist both the inbound and rebound phases of the wall response to blast load. If a FRP retrofit is only applied to one face of the wall (typically to resist tensile stresses during inbound response), the response limits for the wall material without FRP are applicable for the response of the wall during rebound (e.g., the response limits for reinforced concrete components if a reinforced concrete wall only has FRP on one face).

Table 4-7 Response Limits for Flexural Response of Reinforced Concrete and Masonry Walls with FRP Retrofits

Response Type		Superficial Damage	Moderate Damage	Heavy Damage	Hazardous Failure
		μ	μ	μ	μ
Unreinforced Masonry	Flexure	0.5	0.8	1.3	1.8
	Combined Flexure & Compression [1]	0.5	0.8	0.8	0.8
Concrete and Masonry with Conventional Steel Reinforcement	Flexure	0.5	0.8	1.3	1.8
	Combined Flexure & Compression [1]	0.5	0.8	0.8	0.8

1 – This response type applies if a component has a compressive axial load exceeding the threshold listed in Section 4-2.3.1.

4-8 HOT ROLLED STRUCTURAL STEEL.

The response limits for the damage level boundaries of hot rolled structural steel components are shown in Table 4-8.

Table 4-8 Response Limits for Flexural Response of Hot Rolled Structural Steel Components

Response Type		Superficial Damage		Moderate Damage		Heavy Damage		Hazardous Failure	
		μ	θ	μ	θ	μ	θ	μ	θ
Flexure	Compact member	1	-	3	3°	12	10°	20	16°
	Non-compact member	0.7	-	0.85	-	1.0	-	1.2	-
	Plate [2]	1	-	8	3°	20	10°	40	16°
Combined Flexure & Compression [1]	Compact member	1	-	3	3°	3	3°	3	3°
	Non-compact member	0.7	-	0.85	-	0.85	-	0.85	-

1 – This response type applies if a component has a compressive axial load exceeding the threshold listed in Section 4-2.3.1.
2 – Flat plate bent about weak axis.

4-9 COLD FORMED STEEL.

The response limits for the damage level boundaries of cold formed steel girts, purlins, and decking are shown in Table 4-9 and Table 4-10.

Table 4-9 Response Limits for Flexural Response of Cold Formed Steel Girts and Purlins

Response Type		Superficial Damage		Moderate Damage		Heavy Damage		Hazardous Failure	
		μ	θ	μ	θ	μ	θ	μ	θ
Girts and Purlins	Not designed with Tension Membrane Action [1]	1	-	-	4°	-	10°	-	16°
	Designed with Tension Membrane Action [2]	1	-	-	4°	-	12°	-	20°

1 – Girt or purlin is bolted to each support and therefore has some inherent tension membrane capacity but the available tension membrane capacity of component is not calculated and included in dynamic analysis.
2 – The girt or purlin is designed with a tension membrane capacity that is included in the dynamic analysis.

Table 4-10 Response Limits for Flexural Response of Cold Formed Steel Decking

Response Type		Superficial Damage		Moderate Damage		Heavy Damage		Hazardous Failure	
		μ	θ	μ	θ	μ	θ	μ	θ
One-Way Corrugated Metal Deck	Full tensile membrane capacity [1]	1	-	3	3°	6	6°	10	12°
	Limited tensile membrane capacity [2]	1	-	1.8	1.3°	3	2°	6	4°
Standing Seam Metal Deck		1	-	1.8	1.3°	3	2°	6	4°

1 – Deck has connections adequate to fully yield the cross-section.
2 – Deck is connected at both supports with screws or welds that are not sufficient to fully yield the cross-section.

The response limits for the damage level boundaries of cold formed steel stud walls are shown in Table 4-11. The configurations for this component type include Improved Steel Stud Walls (ISSW) and Ductile Steel Stud Walls (DSSW) which both exhibit more ductility than conventionally constructed stud walls. ISSW and DSSW stud walls require more rigorous construction and connection requirements that provide increased response limits but are also more expensive to construct, particularly DSSW. These construction requirements are shown in C.4-9.F and C.4-9.G of the commentary. Some, but not all, of the construction requirements are noted in Table 4-11. There are separate SDOF-based design methodologies for ISSW and DSSW that have been developed for the PDC. The design methodology for DSSW is included in the current version of SBEDS design software (V6.2) distributed by the PDC (see PDC-TR 06-01).

Table 4-11 Response Limits for Flexural Response of Cold Formed Steel Stud Walls

Configuration Type		Superficial Damage		Moderate Damage		Heavy Damage		Hazardous Failure	
		μ	θ	μ	θ	μ	θ	μ	θ
Steel Studs	Studs supported with slip track [1,2]	0.5	-	0.8	-	1.0	-	1.1	-
	Studs connected top & bottom [1,3]	1	-	1.75	-	3.0	-	4.0	-
	Improved steel stud walls (ISSW) [1,4]	1	-	4	4°	-	7°	-	9°
	Ductile steel stud walls (DSSW) with bearing connection to support [5]	1	-	5	5°	8	8°	-	10°
	Ductile steel stud walls (DSSW) with heavy connections to support [6]	1	-	5	5°	-	12°	-	16°

1 – Minimum 0.5 inch OSB or equivalent sheathing attached to blast loaded face of studs with minimum #8 screws at 12 inch spacing along height of studs.
2 – Single or double slip tracks at top support and minimum 1 screw per flange attaching the stud to the bottom track.
3 – Applies when studs have a positive attachment to each support (e.g., each flange attached to track with Tek screws or any clip angle attaching web to support).
4 – Applies when stud wall construction meets requirements in commentary section C.4-9.F. These requirements include studs attached to channel track with 3 inch flanges with 3 #12 Tek screws (20 g to 16 g studs) or 6 #12 Tek Screws (14g and thicker) per flange. Track must have a doubler section and plate washers at bolted attachment to supports.
5 – Applies when stud wall construction meets requirements in commentary section C.4-9.G for DSSW with bearing supports. These requirements include a minimum 16 g steel plate on blast loaded side of studs well attached to studs, 12 g channel track with 2#12 screws attaching each flange of stud to track, lateral bracing for studs at midspan and quarter span locations, minimum 3 inch bearing surface for wall against support.
6 – Applies when stud wall construction meets requirements in commentary section C.4-9.G for DSSW with connection to supports. These requirements include a minimum 16 g steel plate on the blast loaded side of studs well attached to studs, 12 g channel track with 3 inch flange and minimum 4#12 screws attaching each flange of stud to track or bolted connection of stud to support, and lateral bracing for studs at midspan and quarter span locations.

4-10 OPEN WEB STEEL JOISTS.

The response limits for the damage level boundaries of open web steel joists are shown in Table 4-13.

Table 4-12 Response Limits for Open Web Steel Joists

Response Type	Superficial Damage		Moderate Damage		Heavy Damage		Hazardous Failure	
	μ	θ	μ	θ	μ	θ	μ	θ
Flexural Response - Downward [1]	1	-	-	3°	-	6°	-	10°
Shear Response [2]	0.7	-	0.8	-	0.9	-	1	-
Flexural Response - Upward [3]	1	-	1.5	-	2	-	3	-

1 – Flexural response limits for downward loading are based on the assumption that deformation occurs by tensile yielding in the bottom chord. Top chords may need additional bracing to prevent lateral buckling.
2 – Shear response limits apply if member capacity is limited by the capacity of the web members, or web connections, or support connections. Shear response ductility ratio is equal to the peak shear force divided by the shear capacity.
3 – Additional anchors may be needed to prevent pull-out failures. Bottom chords may need additional bracing to prevent lateral buckling.

4-11 WOOD.

The response limits for the damage level boundaries of wood components are shown in Table 4-13.

Table 4-13 Response Limits for Wood Components

Response Type		Superficial Damage		Moderate Damage		Heavy Damage		Hazardous Failure	
		μ	θ	μ	θ	μ	θ	μ	θ
Dimension Lumber	Flexure	1	-	1.5	-	2	-	2	-
	Combined Flexure and Compression [1]	1	-	1.5	-	N/A [2]			
Cross-Laminated Timber (CLT)	Flexure	1	-	2	4°	-	6°	-	8°
	Combined Flexure and Compression [1]	1	-	2	4°	N/A [2]			
Glued-Laminated Timber (Glulam)	Flexure	0.7	-	0.8	-	0.9	-	1.0	-
	Combined Flexure and Compression [1]	0.7	-	0.8	-	N/A [2]			

1 – This response type applies if a component has a compressive axial load exceeding the threshold defined in Section 4-2.3.1

2 – Axially loaded members must be designed to exhibit Moderate Damage or less.

4-12 GLAZING SYSTEM FRAMING.

The response limits for the damage level boundaries of glazing system framing are shown in Table 4-14. Glazing shall meet the hazard performance associated with the Building LOP as shown in Table 2-2.

Table 4-14 Response Limits for Flexural Response of Glazing System Framing

Material Type	Superficial Damage		Moderate Damage		Heavy Damage		Hazardous Failure	
	μ	θ	μ	θ	μ	θ	μ	θ
Aluminum	1	-	5	3°	7	6°	10	10°
Other Materials	SEE APPLICABLE MATERIAL SPECIFIC SECTION							

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CHAPTER 5 ILLUSTRATIVE EXAMPLE

5-1 GENERAL.

The content of this document is ordered below as one might use in practice:

1. Define the building Level of Protection (LOP). See the discussion in Section 2-1.

2. Identify the component to be assessed and determine if it is a primary, secondary, or non-structural component (see Table 2-3).

3. Based on the building LOP and the component type, determine the allowable damage level (see Table 3-1 and damage descriptions in Table 2-4).

4. Determine the axial load demand on the component under gravity (see Equation 4-1).

5. Determine if "Flexure" or "Combined Flexure and Compression" response limits should be used (see Sections 4-2.3.1 and 4-2.3.2).

6. Determine the upper bound response limit for the component per Sections 4-3 through 4-12.

7. Perform SDOF analyses.

    a. Confirm the assumptions and limitations listed in Section 4-2.2 are satisfied or acceptable.

    b. Determine the loads to be considered during the SDOF analysis (see Equation 4-2). Beyond the static loads, consideration should be given if blast loading on supported members are expected to introduce additional dynamic axial loads (see Section 4-2.3.5).

    c. Account for P-Delta effects as appropriate (see Section 4-2.3.6).

    d. Investigate the rebound response as discussed in Section 4-2.4.

8. If required, evaluate the residual axial capacity of the component (see Section 4-2.3.4).

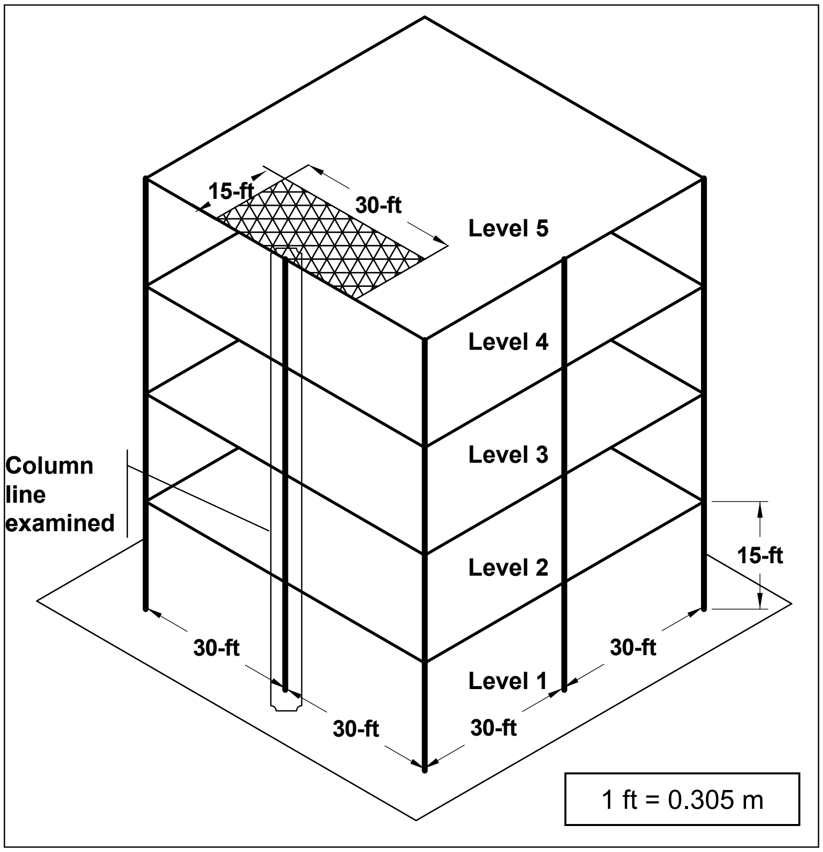
5-2 EXAMPLE PROBLEMS.

In this section the selection of response limits is demonstrated. For the framed structure shown in Figure 5-1 the response limits for various columns in the identified column line, for various building LOP, will be discussed. The following loads and properties will be used throughout this section:

- Total dead load (member weights and superimposed dead load) on each level is 80 psf.

- Live load on each level (excluding roof) is 40 psf.
- Most severe roof loading (max(L_r, $0.7S$, R) is 15 psf.
- Tributary area for the column is 15 ft x 30 ft = 450 ft^{2}.
- Level 5 consist of a concrete topped metal deck, with ultimate resistance (r_u) of 3 psi, supported by open web joists.

Figure 5-1 Framed Structure for Example



5-2.2 LEVEL 1-2 COLUMN.

The building level of protection (LOP) is assumed to be “Very Low”.

The component to be assessed (column on level 1-2) is determined to be a primary component from Table 2-3.

The allowable damage level is determined to be “Heavy” from Table 3-1.

The bounding load combinations (low and high load factors) under gravity load are from Equation 4-1:

- 0.9D -> 0.9 x (80 psf) x (4 levels) x 450 ft^2 = 129,600 lb

- 1.2D + 0.5L + 0.2(L_r or $0.7S$ or R) ->

  1.2 x (80 psf) x (4 levels) x 450 ft^2 +

  0.5 x (40 psf) x (3 levels) x 450 ft^2 +

  0.2 x (15 psf) x (1 level) x 450 ft^2 = 201,150 lb

From Section 4-2.3.1, the response limits for components in "Combined Flexure and Compression" should be used if the axial compressive load demand is greater than 10% of the axial compressive load capacity. That is, if the factored axial compressive load capacity of the column is less than 2,011,500 lb, then "Combined Flexure and Compression" response limits should be used.

If reinforced concrete will be used for the level 1-2 column the limits from Table 4-1 apply. Assuming shear reinforcing will be used throughout the member, tension membrane is not applicable, and the column has a reinforcement index of 6%, the response limit for "Flexure" would be $10.4(6)^{-0.16} = 7.8°$. That is, the rotation demand should not exceed 7.8° for the column to be considered having Heavy damage (or less).

Since the column does not directly support other structural members subjected to blast, it is expected that the peak axial compressive load (stemming from the supported members) and the column's peak lateral displacement (stemming from the lateral blast load) occur at notably different instances in time. That is, it is unlikely that the blast load applied to the roof causes a large dynamic axial load in the level 1-2 column concurrently with the column being subjected to the lateral blast load. Thus, only the gravity axial force is utilized during the dynamic SDOF analysis (see 4-2.3.5).

Assuming the column's factored axial compressive capacity is 1,600,000 lb (i.e., the gravity axial load ratio is 201,150 lb / 1,600,000 lb = 12.6% > 10%), the SDOF analysis would utilize the rotation response limit of $7.9(RI)^{-0.28}$, include an axial load of 201,150 lb, include P-Delta effects (see Section 4-2.3.6), and rebound would be considered (see Section 4-2.4). However, a residual axial capacity evaluation is not required as the three criteria in Section 4-2.3.4 are not met (e.g., the gravity load demand [201,150 lb] is not greater than 30% of the axial compressive load capacity [0.3 x 1,600,000 lb = 480,000 lb]).

5-2.3 LEVEL 4-5 COLUMN.

The building level of protection (LOP) is assumed to be "Medium".

The component to be assessed (column on level 4-5) is determined to be a primary component from Table 2-3.

The allowable damage level is determined to be "Superficial" from Table 3-1.

The bounding load combinations (low and high load factors) under gravity load are from Equation 4-1:

- 0.9D -> 0.9 x (80 psf) x (1 levels) x 450 ft^2 = 32,400 lb
- 1.2D + 0.5L + 0.2(L_r or $0.7S$ or R) ->

  1.2 x (80 psf) x (1 levels) x 450 ft^2 +

  0.2 x (15 psf) x (1 level) x 450 ft^2 = 44,550 lb

From Section 4-2.3.1, the response limits for components in "Combined Flexure and Compression" should be used if the axial compressive load demand is greater than 10% of the axial compressive load capacity. That is, if the factored axial compressive load capacity of the column is less than 445,500 lb, then "Combined Flexure and Compression" response limits should be used.

If a steel column will be used for the level 4-5 column the limits from Table 4-8 apply. Assuming the column is compact, the response limit for either "Flexure" or "Combined Flexure and Compression" is μ = 1.

Assuming the column's factored axial compressive capacity is 800,000 lb (i.e., the gravity axial load ratio is 44,550 lb / 800,000 lb = 5.5% > 10%), the SDOF analysis would utilize the "Flexure" response limit of μ = 1, include an axial load of 44,550 lb, not include P-Delta effects (see Section 4-2.3.6), and rebound would be considered (see Section 4-2.4). Further, a residual axial capacity evaluation is not required as the three criteria in Section 4-2.3.4 are not met (e.g., the gravity load demand [44,550 lb] is not greater than 30% of the axial compressive load capacity [0.3 x 800,000 lb = 240,000 lb]).

Following this initial check, a second check is performed. Since the column directly supports a roof structure it is reasonable to add the ultimate resistance of the members framing into the column at the roof level as a dynamic axial load during the SDOF analysis (see the discussion in 4-2.3.5). In this case, the additional dynamic load (concurrent with the static axial load) is estimated as 3psi x (450 ft^2) = 194,400 lb. Thus, during the 'high load factor' SDOF assessment, a total axial load effect of 44,550 lb + 194,400 lb = 238,950 lb would be considered. In this case, the SDOF analysis would still utilize the "Flexure" response limit of μ = 1, include a dynamic axial load of 238,950 lb, include P-Delta effects (see Section 4-2.3.6), and rebound would be considered (see Section 4-2.4). A residual axial capacity evaluation is still not required as the gravity load demand (44,550 lb) is not greater than 30% of the axial compressive load capacity (0.3 x 800,000 lb = 240,000 lb).

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CHAPTER 6 REFERENCES IN MANDATORY PROVISIONS

AMERICAN CONCRETE INSTITUTE

https://www.concrete.org/

ACI 318, *Building Code Requirements for Structural Concrete*

AMERICAN SOCIETY OF CIVIL ENGINEERS

https://www.asce.org/

ASCE 7, *Minimum Design Loads and Associated Criteria for Buildings and Other Structures*

ASTM INTERNATIONAL

https://www.astm.org/

ASTM F2247, *Standard Test Method for Metal Doors Used in Blast Resistant Applications (Equivalent Static Load Method)*

ASTM F2912, *Standard Specification for Glazing and Glazing Systems Subject to Airblast Loadings*

ASTM F2927, *Standard Test Method for Door Systems Subject to Airblast Loadings*

UNIFIED FACILITIES CRITERIA

https://www.wbdg.org/ffc/dod/unified-facilities-criteria-ufc

UFC 3-340-01, *Design and Analysis Of Hardened Structures To Conventional Weapons Effects* [EXPORT CONTROLLED, DISTRIBUTION C]

UFC 3-340-02, *Structures to Resist The Effects Of Accidental Explosions*

UFC 4-010-01, *DoD Minimum Antiterrorism Standards for Buildings*

UFC 4-020-01, *DoD Security Engineering Facilities Planning Manual*

U.S. ARMY CORPS OF ENGINEERS PROTECTIVE DESIGN CENTER

https://pdc.usace.army.mil

PDC-TR 06-01, *Methodology Manual for the Single-Degree-of-Freedom Blast Effects Design Spreadsheets (SBEDS)*

PDC-TR 18-02, *Analysis Guidance for Cross-Laminated Timber Construction Exposed to Airblast Loading*

OTHER REFERENCES

Air Force Civil Engineering Support Agency (AFCESA). (August 2000). *Engineering Technical Letter (ETL) 00-9: Airblast Protection Retrofit for Unreinforced Concrete Masonry Walls.* AFCESA