

# UNIFIED FACILITIES CRITERIA (UFC)

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## DESIGN OF BUILDINGS TO RESIST PROGRESSIVE COLLAPSE



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### DESIGN OF BUILDINGS TO RESIST PROGRESSIVE COLLAPSE

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U.S. ARMY CORPS OF ENGINEERS (Preparing Activity)

NAVAL FACILITIES ENGINEERING COMMAND

AIR FORCE CIVIL ENGINEER SUPPORT AGENCY

Record of Changes (changes are indicated by \1\ ... /1/)

Change No.	Date	Location
1	27 January 2010	Changed all references from UFC 3-310-01 "Structural Load Data" to UFC 3-301-01 "Structural Engineering"
2	1 June 2013	Revised tie force equations; removed 0.9 factor and lateral loads from alternate path load combination; clarified definition of controlled public access; clarified live load reduction requirements; revised reinforced concrete and structural steel examples; added cold-formed steel example.
3	1 November 2016	Changed organization name to Air Force Civil Engineer Center (AFCEC), updated ASCE 41-13 references, changed all reference from "Occupancy Category" to "Risk Category", added a footnote "A" for Risk Category IV & V in Table 2-2, added section 1-8 for Design Submittals and section 1-9 for Recommendations for Specification, updated Chapter 17 section numbering. Editorial changes throughout.
4	10 June 2024	Removed inconsistent language with UFC-4-010-01.

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This UFC supersedes UFC 4-023-03, dated 25 January, 2005. The format of this document does not conform to UFC 1-300-01.

## FOREWORD

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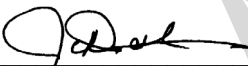
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### AUTHORIZED BY:



JAMES C. DALTON, P.E.  
Chief, Engineering and Construction  
U.S. Army Corps of Engineers



PAUL A. PARKER  
The Deputy Civil Engineer  
DCS/Installations & Logistics  
Department of the Air Force



JOSEPH E. GOTT, P.E.  
Chief Engineer  
Naval Facilities Engineering Command



MICHAEL McANDREW  
Director, Facility Investment and  
Management  
Office of the Deputy Under Secretary of Defense  
(Installations and Environment)

**UNIFIED FACILITIES CRITERIA (UFC)  
REVISION SUMMARY SHEET**

**Subject:** UFC 4-023-03, Design of Structures to Resist Progressive Collapse

**Cancels:** UFC 4-023-03, Design of Structures to Resist Progressive Collapse, Dated 25 January 2005

**Reasons for Change.** UFC 4-023-03 was updated for the following reasons:

- Removed inconsistent language within UFC 4-010-01.

**Description of Changes.** This update to UFC 4-023-03 is a minor revision to the 25 January 2005 version. The changes include:

- Review use of modifiers “new” and “existing”
- Revise Section 1-2, APPLICABILITY to match information from the Security DWG
- Revised Section 1-2.1, Building Type and Story Height for clarity.

**Impact.** The impact of this updated UFC 4-023-03 will vary depending upon the particular structure, structure type, location, and function. Retrofitting of existing buildings to resist progressive collapse is no longer mandatory, which aligns UFC 4-023-03 with UFC 4-010-01.

**Non-Unified Issues.** Document content is unified and consistent for all services and agencies of the Department of Defense.

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## CHAPTER 1 INTRODUCTION

### 1-1 PURPOSE AND SCOPE.

This Unified Facilities Criteria (UFC) provides the design requirements necessary to reduce the potential of progressive collapse for ~~4\~~ /4/ facilities that experience localized structural damage through normally unforeseeable events.

### 1-2 APPLICABILITY.

~~4\~~ The requirements outlined in this UFC are mandatory for all new construction of buildings three or more stories. For existing buildings, these requirements may be applied as required by the project proponents or AHJ. /4/

#### 1-2.1 Building Type and Story Height.

~~4\~~ /4/ For this UFC, penthouse structures and floors below grade (i.e., single and multiple level basements) will be considered a story if there is any space that is designed for human occupancy and that is equipped with means of egress as well as light and ventilation facilities that meet the local building code requirements. If any story will not be occupied, perhaps due to mechanical equipment or storage, that story will be omitted from the calculation of the number of stories.

At changes in building elevation from a one or two story section to a section with three or more stories, the appropriate progressive collapse design requirements from Section 2-2 shall be applied to the section with three or more stories. Special attention shall be given to potential deleterious effects associated with the attachment of the short building section to the building section with three or more stories.

#### 1-2.2 Clarification for Partial Occupancy.

When DoD personnel occupy 25% or more of the net interior useable space, the requirements of this UFC are applicable to the entire structure, not just the portion of the building occupied by DoD personnel; this requirement supersedes that given in UFC 4-010-01 *DoD Minimum Antiterrorism Standards for Buildings*.

~~2\~~

#### 1-2.3 Clarification for Controlled Public Access.

Per UFC 4-010-01 *DoD Minimum Antiterrorism Standards for Buildings*, controlled public access will be considered to include (but not be limited to) electronic access control devices or mechanical locks on all exterior doors. Where visitor processing makes locking visitor entrances during building operating hours impractical, providing personnel to control visitor access can be considered positive control at those entrances.

/2/

#### **1-2.4 Application by Other Organizations.**

This UFC may be employed by other federal and state government agencies as well as organizations that create and implement building codes (e.g. International Building Code) and material specific design codes (e.g., American Institute of Steel Construction, American Concrete Institute, The Masonry Society, American Iron and Steel Institute, American Forest and Paper Association). The responsibility for determining applicability rests with the specifying agency.

The material contained herein is not intended as a warranty on the part of DoD that this information is suitable for any general or particular use. The user of this information assumes all liability arising from such use. This information should not be used or relied upon for any specific application without competent professional examination and verification.

#### **1-3 GENERAL.**

Progressive collapse is defined in the commentary of the American Society of Civil Engineers Standard 7 *Minimum Design Loads for Buildings and Other Structures* (ASCE 7) as “the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it.” The standard further states that buildings should be designed “to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage.” As discussed in the commentary of ASCE 7, “except for specially designed protective systems, it is usually impractical for a structure to be designed to resist general collapse caused by severe abnormal loads acting directly on a large portion of it. However, structures can be designed to limit the effects of local collapse and to prevent or minimize progressive collapse.” The structural design requirements presented herein were developed to ensure prudent precautions are taken when the event causing the initial local damage is undefined and the extent of the initial damage is unknown.

##### **1-3.1 Significance of Progressive Collapse.**

Progressive collapse is a relatively rare event, in the United States and other Western nations, as it requires both an abnormal loading to initiate the local damage and a structure that lacks adequate continuity, ductility, and redundancy to resist the spread of damage. However, significant casualties can result when collapse occurs. This is illustrated by the April 19, 1995 bombing of the Alfred P. Murrah building in

Oklahoma City, in which the majority of the 168 fatalities were due to the partial collapse of the structure and not to direct blast effects. The recent escalation of the domestic and international terrorist threat has increased the probability that other US government structures will be attacked with explosives or other violent means.

### **1-3.2 Hardening of Structures to Resist Initial Damage.**

As the initiating event is unknown, the requirements in this UFC are not intended to directly limit or eliminate the initial damage. This is consistent with UFC 4-010-01, which applies where there is a known risk of terrorist attack, but no specific terrorist threat is defined; in this case, the goal is to reduce the risk of mass casualties in the event of an attack. For cases where specific explosive threats against a building have been identified, the designer shall employ the appropriate design methodology for hardening the building. However, even though a structure is designed to resist an identified explosive threat, the progressive collapse design requirements herein shall still apply.

### **1-3.3 Risk Considerations.**

Hazards and consequences are addressed in a typical risk assessment. Due to the limited database of progressive collapse events (from deliberate attack, vehicle impact, natural causes, etc), it is not possible to reasonably assess the probability of occurrence for a specific hazard or group of hazards. Therefore, the risk assessment reduces to a consideration of consequences. In general, consequences are measured in terms of human casualties and, therefore, the risk category of a building or structure is often the most critical issue. The progressive collapse design approaches in this UFC are primarily a function of the risk category of the building, although the structure's function is also considered. In Section 2-1, guidance is provided on choosing the Risk Category of a building, using the occupancy tables contained in UFC 3-301-01 *Structural Engineering*.

### **1-3.4 Design Approaches.**

ASCE 7 defines two general approaches for reducing the possibility of progressive collapse: Direct Design and Indirect Design.

#### **1-3.4.1 Direct Design Approaches.**

Direct Design approaches include "explicit consideration of resistance to progressive collapse during the design process..." These include: 1) the Alternate Path (AP) method, which requires that the structure be capable of bridging over a missing structural element, with the resulting extent of damage being localized, and 2) the Specific Local Resistance (SLR) method, which requires that the building, or parts of the building, provide sufficient strength to resist a specific load or threat.

### **1-3.4.2 Indirect Design Approaches.**

With Indirect Design, resistance to progressive collapse is considered implicitly "through the provision of minimum levels of strength, continuity and ductility". The commentary in ASCE 7 goes on to present general design guidelines and suggestions for improving structural integrity. These include: 1) good plan layout, 2) integrated system of ties, 3) returns on walls, 4) changing span directions of floor slabs, 5) load-bearing interior partitions, 6) catenary action of the floor slab, 7) beam action of the walls, 8) redundant structural systems, 9) ductile detailing, 10) additional reinforcement for blast and load reversal, if the designer must consider explosive loads, and 11) compartmentalized construction. However, no quantitative requirements for either direct or indirect design to resist progressive collapse are provided in ASCE 7.

In this UFC, Tie Forces (TF) are used to enhance continuity, ductility, and structural redundancy by specifying minimum tensile forces that must be used to tie the structure together. This approach is similar to that employed by the British after the Ronan Point apartment building collapse in 1968 and currently used in the Eurocode.

## **1-4 SUMMARY OF THE PROGRESSIVE COLLAPSE DESIGN PROCEDURE.**

For \4\ /4/ the level of progressive collapse design for a structure is correlated to the \3\ risk category /3/ (RC). The RC will either be assessed per Section 2-1.

The design requirements in this UFC were developed such that varying levels of resistance to progressive collapse are specified, depending upon the RC as discussed in Chapter 2. These levels of progressive collapse design employ:

- Tie Forces, which prescribe a tensile force strength of the floor or roof system, to allow the transfer of load from the damaged portion of the structure to the undamaged portion,
- Alternate Path method, in which the building must bridge across a removed element, and
- Enhanced Local Resistance, in which the shear and flexural strength of the perimeter columns and walls are increased to provide additional protection by reducing the probability and extent of initial damage.

## **1-5 REFERENCES.**

This UFC incorporates provisions from other publications by dated or undated reference. These references are cited at the appropriate places in the text and the citations for the publications are listed in Appendix A References. For dated references, subsequent amendments to, or revisions of, any of these publications apply to this UFC only when incorporated in it by amendment or revision. For undated references, the latest edition of the referenced publication applies (including amendments).

## 1-6 INSPECTION REQUIREMENTS.

Inspection requirements to verify conformance with this UFC are provided in Appendix H. These inspection requirements are modifications to the provisions of the International Building Code (IBC), which cover construction documents, structural tests and special inspections for buildings that have been designed to resist progressive collapse.

## 1-7 SECURITY ENGINEERING UFC SERIES.

This UFC is one of a series of security engineering Unified Facilities Criteria that cover minimum standards, planning, preliminary design, and detailed design for security and antiterrorism. The manuals in this series are designed to be used sequentially by a diverse audience to facilitate development of projects throughout the design cycle. The manuals in this series include the following:

**DoD Minimum Antiterrorism Standards for Buildings.** UFC 4-010-01 *DoD Minimum Antiterrorism Standards for Buildings* and UFC 4-010-02 *DoD Minimum Standoff Distances for Buildings* establish standards that provide minimum levels of protection against terrorist attacks for the occupants of all DoD inhabited buildings. These UFC are intended to be used by security and antiterrorism personnel and design teams to identify the minimum requirements that must be incorporated into the design of all new construction and major renovations of inhabited DoD buildings. They also include recommendations that should be, but are not required to be, incorporated into all such buildings.

**Security Engineering Facility Planning Manual.** UFC 4-020-01 *Security Engineering Facility Planning Manual* presents processes for developing the design criteria necessary to incorporate security and antiterrorism features into DoD facilities and for identifying the cost implications of applying those design criteria. Those design criteria may be limited to the requirements of the minimum standards, or they may include protection of assets other than those addressed in the minimum standards (people), aggressor tactics that are not addressed in the minimum standards, or levels of protection beyond those required by the minimum standards. The cost implications for security and antiterrorism are addressed as cost increases over conventional construction for common construction types. The changes in construction represented by those cost increases are tabulated for reference, but they represent only representative construction that will meet the requirements of the design criteria. The manual also includes a means to assess the tradeoffs between cost and risk. The Security Engineering Facility Planning Manual is intended to be used by planners as well as security and antiterrorism personnel with support from planning team members.

**Security Engineering Facility Design Manual.** UFC 4-020-02 *Security Engineering Facility Design Manual* provides interdisciplinary design guidance for

developing preliminary systems of protective measures to implement the design criteria established using UFC 4-020-01. Those protective measures include building and site elements, equipment, and the supporting manpower and procedures necessary to make them all work as a system. The information in UFC 4-020-02 is in sufficient detail to support concept level project development, and as such can provide a good basis for a more detailed design. The manual also provides a process for assessing the impact of protective measures on risk. The primary audience for the Security Engineering Facility Design Manual is the design team, but security and antiterrorism personnel can also use it.

**Security Engineering Support Manuals.** In addition to the standards, planning, and design UFC mentioned above, there is a series of additional UFC that provide detailed design guidance for developing final designs based on the preliminary designs developed using UFC 4-020-02. These support manuals provide specialized, discipline specific design guidance. Some address specific tactics such as direct fire weapons, forced entry, or airborne contamination. Others address limited aspects of design such as resistance to progressive collapse or design of portions of buildings such as mailrooms. Still others address details of designs for specific protective measures such as vehicle barriers or fences. The Security Engineering Support Manuals are intended to be used by the design team during the development of final design packages.

/3\

## **1-8 DESIGN SUBMITTALS.**

Design submittals for DoD projects requiring compliance with these criteria will include the following elements as a minimum;

- Narratives of the design includes the Risk Category, Design Approach (Tie Forces (TF) or Alternate Path (AP), and Design Method (Linear Static, Nonlinear Static or Nonlinear Dynamic),
- A list of the design software used to perform the progressive collapse mitigation, and
- The electronic input files for all phases of the design review.

## **1-9 RECOMMENDATIONS FOR SPECIFICATION.**

Additional recommended specification language for new and existing buildings is included in Appendix C-11. The recommendations in this document include the sample paragraphs for the Request-For-Proposal (RFP) specification.

/3/

## CHAPTER 2 PROGRESSIVE COLLAPSE DESIGN REQUIREMENTS FOR NEW AND EXISTING CONSTRUCTION

For both new and existing buildings, the level of progressive collapse design will be based on the risk category (RC) of the structure. The risk category will be determined per Section 2-1. The RC is used to define the corresponding level of progressive collapse design for new and existing construction as detailed in Section 2-2.

Chapter 3 Design Procedures provides the approaches and requirements for applying Tie Forces (TF), Alternate Path (AP), and Enhanced Local Resistance (ELR). The overall techniques for these three approaches are the same for each construction type, but the details may vary with material type. Chapters 4 through 8 provide the material specific design requirements. Finally, Appendix C provides insight into the development of these approaches.

### 2-1 RISK CATEGORY DETERMINATION.

Determine the Risk Category (RC) of a particular structure by using Table 2-1 for the situation that most closely matches the building. The Risk Category is taken from the risk category definitions in UFC 3-301-01 *Structural Engineering*; the RC level can be considered as a measure of the consequences of a progressive collapse event and is based on two main factors: level of occupancy and building function or criticality.

**Table 2-1. Risk Categories**

Nature of Occupancy	Risk Category <sup>c</sup>
<ul style="list-style-type: none"> <li>Buildings in Risk Category I in Table 2-2 of UFC 3-301-01.</li> <li>Low Occupancy Buildings<sup>A</sup></li> </ul>	I
<ul style="list-style-type: none"> <li>Buildings in Risk Category II in Table 2-2 of UFC 3-301-01.</li> <li>Inhabited buildings with less than 50 personnel, primary gathering buildings, billeting, and high occupancy family housing<sup>A,B</sup></li> </ul>	II
<ul style="list-style-type: none"> <li>Buildings in Risk Category III in Table 2-2 of UFC 3-301-01.</li> </ul>	III
<ul style="list-style-type: none"> <li>Buildings in Risk Category IV in Table 2-2 of UFC 3-301-01.</li> <li>Buildings in Risk Category V in Table 2-2 of UFC 3-301-01.</li> </ul>	IV

<sup>A</sup> As defined by UFC 4-010-01 *DoD Minimum Antiterrorism Standards for Buildings*

<sup>B</sup> Risk Category II is the minimum occupancy category for these buildings, as their population or function may require designation as Risk Category III, IV, or V.

<sup>c</sup> Section 1604.5.1 Multiple occupancies of the International Building Code (IBC) is applicable for determination of the Risk Category including the provisions for structurally separated structures.

## 2-2 DESIGN REQUIREMENTS \4\./4/.

The design requirements for each \3\ risk category /3/ (RC) are shown in Table 2-2. The details are provided in the following sections.

**Table 2-2. Risk Categories and Design Requirements**

\3\ Risk Category /3/	Design Requirement
I	No specific requirements
II	Option 1: Tie Forces (TF) for the entire structure and Enhanced Local Resistance (ELR) for the corner and penultimate columns or walls at the first story. <b>OR</b> Option 2: Alternate Path (AP) for specified column and wall removal locations.
III	Alternate Path for specified column and wall removal locations and Enhanced Local Resistance (ELR) for all perimeter first story columns or walls.
IV <sup>A</sup>	Tie Forces and Alternate Path for specified column and wall removal locations and Enhanced Local Resistance for all perimeter first story columns or walls.

\3\ <sup>A</sup> For buildings in Risk Category IV in Table 2-2 of UFC 3-301-01, the minimum structural requirements for Tie Force application in Section 3-1.1 can be exempted. The minimum structural requirements shall remain for buildings in Risk Category V. /3/

### 2-2.1 \3\ Risk Category /3/ I Design Requirement.

Progressive collapse design is not required for these structures.

### 2-2.2 \3\ Risk Category /3/ II Design Requirement.

For RC II structures, one of two options may be chosen. In the first, the designer shall incorporate the Tie Force requirement for the entire structure and Enhanced Local Resistance for the first story corner and penultimate columns and walls (a penultimate column or wall is the closest column or wall to the corner). In the second option, the designer shall design or analyze the building with the Alternate Path method to show that the structure can bridge over the removal of columns, load-bearing walls, or beams supporting columns or walls at specified locations.



### **2-2.2.1 Option 1 for \3\ Risk Category /3/ II: Tie Force and Enhanced Local Resistance.**

The requirements in 2-2.2.1.1 and 2-2.2.1.2 for Tie Forces and Enhanced Local Resistance shall be satisfied, if this option is chosen.

#### **2-2.2.1.1 Tie Force Requirement for RC II Option 1.**

The procedure and requirements for Tie Forces for framed and load-bearing wall structures are presented in Section 3-1.

If a vertical structural member cannot provide the required vertical tie force strength, either re-design the member or use the AP method to prove that the structure can bridge over the element when it is removed.

For elements with inadequate horizontal tie force strength, the designer shall re-design the element in the case of new construction or retrofit the element in the case of existing construction. The AP method cannot be used as an alternative for inadequate horizontal ties.

#### **2-2.2.1.2 Enhanced Local Resistance Requirement for RC II Option 1.**

The Enhanced Local Resistance requirement is applied to the first story corner and penultimate columns and walls only. For this requirement for RC II Option 1, the flexural demand of the column or wall is not increased; however, the design shear strength of the column or wall and the connections to the slabs, floor system or other lateral load resisting elements shall be greater than the shear demand associated with the flexural demand of the component. The procedure is presented in Section 3-3.

### **2-2.2.2 Option 2 for \3\ Risk Category /3/ II: Alternate Path.**

If the Alternate Path requirement is chosen, then the structure shall be able to bridge over vertical load-bearing elements that are notionally removed one at a time from the structure at specific plan and elevation locations, as required in Section 3-2. The procedures and general requirements for the Alternate Path method are provided in Section 3-2 with specific requirements for each material given in Chapters 4 through 8. If bridging cannot be demonstrated for one of the removed load-bearing elements, the structure shall be re-designed or retrofitted to increase the bridging capacity.

If the results of the analyses are similar for multiple locations due to the redundancy of the building, a formal analysis is not required for every location, provided that one typical analysis is performed, and that this observation is annotated in the design documents.

Note: for load-bearing wall structures, the Alternate Path approach will often be the most practical choice.

### **2-2.3 \3\ Risk Category /3/ III Design Requirement.**

For \3\ Risk Category /3/ III, two requirements shall be satisfied: Alternate Path and Enhanced Local Resistance as discussed in the following sections.

#### **2-2.3.1 Alternate Path Requirement for \3\ Risk Category /3/ III.**

The structure shall be able to bridge over vertical load-bearing elements that are notionally removed one at a time from the structure at specific plan and elevation locations, as required in Section 3-2. If bridging cannot be demonstrated for one of the removed load-bearing elements, the structure shall be re-designed or retrofitted to increase the bridging capacity. Note that the structural re-design or retrofit is not applied to just the deficient element, i.e., if a structure cannot be shown to bridge over a removed typical column at the center of the long side, the engineer shall develop suitable or similar re-designs or retrofits for that column and other similar columns.

The procedures and general requirements for the Alternate Path method are provided in Section 3-2 with specific requirements for each material given in Chapters 4 through 8.

#### **2-2.3.2 Enhanced Local Resistance Requirement for \3\ Risk Category /3/ III.**

The Enhanced Local Resistance requirement is applied to all first story perimeter columns and walls. For this requirement, for RC III, the flexural demand of the column or wall need not be increased; however, the design shear strength of the column or wall and the connections to the slabs, floor system or other lateral load resisting elements shall be greater than the shear demand associated with the flexural demand of the component. The procedure is presented in Section 3-3.

### **2-2.4 \3\ Risk Category /3/ IV Design Requirement.**

The design requirements for \3\ Risk Category /3/ IV include Alternate Path, Tie Forces and Enhanced Local Resistance as discussed in the following paragraphs.

#### **2-2.4.1 Tie Force Requirement for \3\ Risk Category /3/ IV.**

For RC IV, the designer shall provide adequate internal, peripheral and vertical Tie Force strengths. The procedure and requirements for applying the Tie Force approach are provided in Section 3-1.

If a structural member cannot provide the required vertical tie force strength, the designer shall either re-design the member or use the Alternate Path method to prove that the structure can bridge over the element when it is removed. For elements with inadequate horizontal tie force strength, the Alternate Path method cannot be used. In this case, the designer shall re-design the element in the case of new construction or retrofit the element for existing construction.

The minimum structural requirements for Tie Force application in Section 3-1.1 can be exempted for \3\ Risk Category /3/ IV, however, for \3\ Risk Category /3/ V, the minimum structural requirements shall remain.

**2-2.4.2 Alternate Path Requirement for \3\ Risk Category /3/ IV.**

For RC IV, use the same AP requirement as for RC III; see Section 2-2.3.1.

**2-2.4.3 Enhanced Local Resistance Requirement for \3\ Risk Category /3/ IV.**

The Enhanced Local Resistance requirement is applied to all first story perimeter columns and walls. The flexural demand of the columns and walls for a design with only gravity loads meeting the AP requirements shall be increased respectively by a factor of 2 and a factor of 1.5 and compared to the flexural demands for the building as designed to meet the AP requirements and all other building code requirements; the larger of these two flexural demands is used to determine the shear demand. The design shear strength of the column or wall and the connections to the slabs, floor system or other lateral load resisting elements shall be greater than this shear demand. Procedures for Enhanced Local Resistance are given in Section 3-3.

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## CHAPTER 3 DESIGN PROCEDURES

The progressive collapse design requirements employ three design/analysis approaches: Tie Forces (TF), Alternate Path (AP), and Enhanced Local Resistance (ELR). This chapter discusses the required procedures for these approaches.

### 3-1 TIE FORCES.

In the Tie Force approach, the building is mechanically tied together, enhancing continuity, ductility, and development of alternate load paths. Tie forces can be provided by the existing structural elements that have been designed using conventional design methods to carry the standard loads imposed upon the structure. There are three horizontal ties that must be provided: longitudinal, transverse, and peripheral. Vertical ties are required in columns and load-bearing walls. Figure 3-1 illustrates these ties for frame construction. Note that these “tie forces” are different from “reinforcement ties” as defined in ACI 318 *Building Code Requirements for Structural Concrete*.

Unless the structural members (beams, girders, spandrels) and their connections can be shown capable of carrying the required longitudinal, transverse, or peripheral tie force magnitudes while undergoing rotations of 0.20-rad (11.3-deg), the longitudinal, transverse, and peripheral tie forces are to be carried by the floor and roof system. Acceptable floor and roof systems include cast-in-place concrete, composite decks, and precast concrete floor planks with concrete topping, reinforcement, and mechanical anchorage that meet the requirements of Sections 3-1.4 through 3-1.6. Other floor or roof systems may be used, provided that the ability to carry the required tie strength while undergoing rotations of 0.20-rad (11.3-deg) is adequately demonstrated to and approved by an independent third-party engineer or by an authorized representative of the facility owner.

\2\ Note: The use of structural composites such as fiber reinforced polymer (FRP) to provide the tie forces is not acceptable unless the designer can prove that the FRP is capable of meeting the rotation requirement of 0.20-rad (11.3-deg) specified in the previous paragraph. Also, the tie force design must not allow portions of the structure to break free and fall to the floor above, i.e., positive mechanical anchorage or connection must be provided between the tie force members and the other structural elements (floors, beams, etc). /2/

\2\

#### 3-1.1 Minimum Structural Requirements for Tie Force Application.

To apply the Tie Force method, the structure must meet the requirements listed in the following paragraphs.

For framed and two-way load-bearing wall structures, the number of bays in both directions must be four or greater. A bay is defined as the square or rectangular floor area with boundaries demarked by vertical load-bearing elements, such as columns at the corners or load-bearing walls along the edges.

For one-way load-bearing wall structures, the number of bays in the one-way span direction must be four or greater. The length of the load-bearing walls (or width of the building) must be at least **4  $h_w$**  where  $h_w$  is the clear story height.

/2/

### 3-1.2 Load and Resistance Factor Design for Tie Forces.

Following the Load and Resistance Factor Design (LRFD) approach, the design tie strength is taken as the product of the strength reduction factor,  $\phi$ , and the nominal tie strength  $R_n$  calculated in accordance with the requirements and assumptions of applicable material specific codes. Include any over-strength factors provided in Chapters 9 to 11 of ASCE 41, where these over-strength factors are referred to as “factors to translate lower bound material properties to expected strength material properties” and are given in Tables 9-3 (structural steel), 10-4 (reinforced concrete), and 11-2 (masonry). For wood and cold-formed steel, Chapter 12 of ASCE 41 provides default expected strength values; note that for wood construction, a time effect factor  $\lambda$  is also included. Per the LRFD approach, the design tie strength must be greater than or equal to the required tie strength:

$$\phi R_n \geq R_u \quad \text{Equation (3-1)}$$

where

- $\phi R_n$  = Design tie strength
- $\phi$  = Strength reduction factor
- $R_n$  = Nominal tie strength calculated with the appropriate material specific code, **including the over-strength factors from Chapters 5 to 8 of ASCE 41.**
- $R_u$  =  $\sum \gamma_i Q_i$  = Required tie strength
- $\gamma_i$  = Load factor
- $Q_i$  = Load effect

The required tie strengths are provided in the following sub-sections for framed and load-bearing wall structures.

\2\2/

### 3-1.3 Floor Loads.

#### 3-1.3.1 Uniform Floor Load.

Use the floor load in Equation 3-2 to determine the required tie strengths:

$$w_F = 1.2D + 0.5L \quad \text{Equation (3-2)}$$

Where  $w_F$  = Floor Load (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)

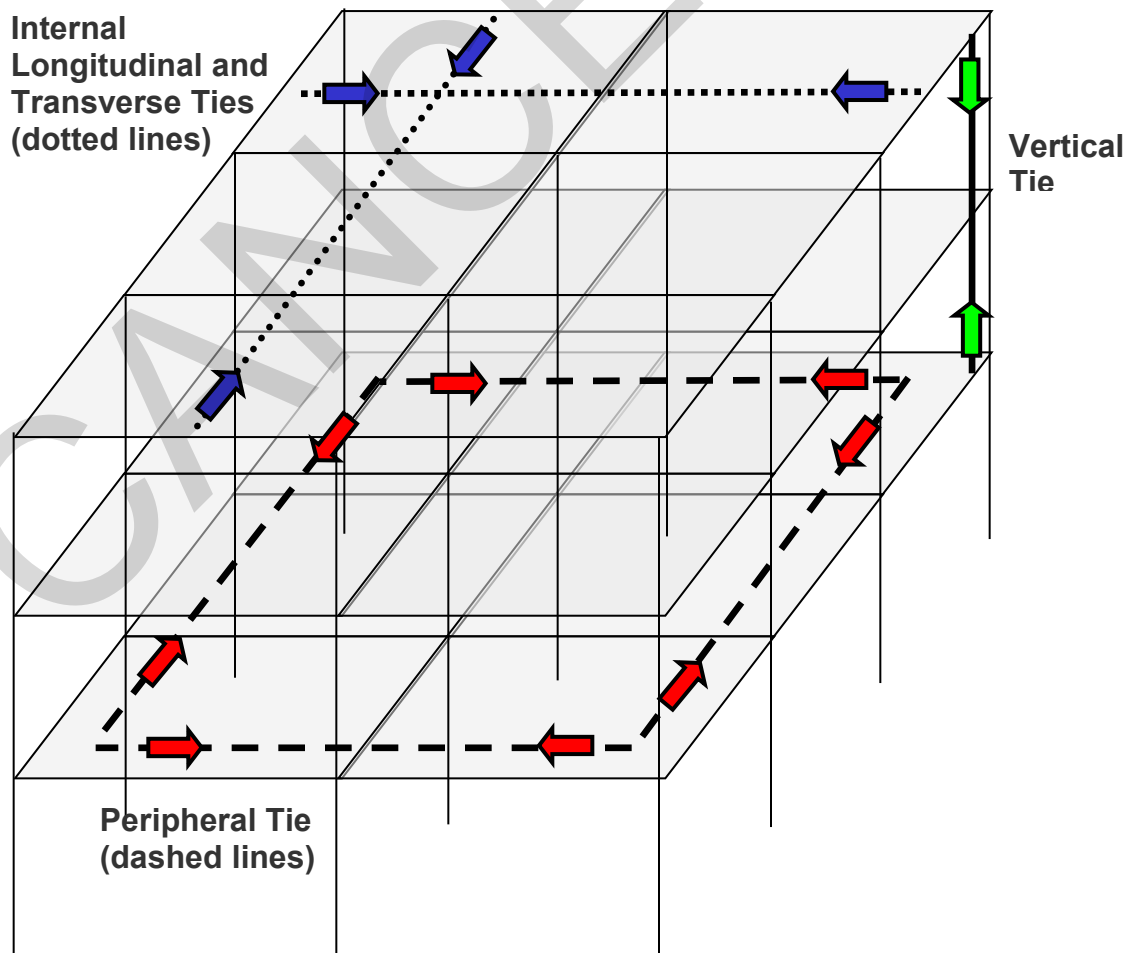
$D$  = Dead Load (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)

$L$  = Live Load (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)

If the Dead Load or Live Load vary over the plan of the floor, use the procedure in Section 3-1.3.2 to determine the effective  $w_F$ .

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Figure 3-1. Tie Forces in a Frame Structure



- NOTE:**
1. Peripheral, longitudinal and transverse ties are not required in floors above crawlspaces if public access control is provided.
  2. Vertical ties are not required to extend to the foundation and shall be straight. /2/

### 3-1.3.2 Consideration for Non-Uniform Load Over Floor Area.

#### 3-1.3.2.1 Concentrated Loads.

If a concentrated load is located within a bay or one portion of the bay has a different loading than the rest of the bay, distribute the load evenly over the bay area and include in the dead or live load, as appropriate, in Equation 3-2.

#### 3-1.3.2.2 Load Variations.

The load magnitude may vary significantly over the plan area of a given story, e.g. manufacturing activities may be located in one section of the floor and office space in another; see Figure 3-2. Calculate the floor load for each bay using Equation 3-2. Determine the effective floor load that will be used to determine the longitudinal, transverse, and peripheral Tie Forces, as follows:

- 1) If the difference between the minimum and maximum floor load in the bays on the floor plan is less than or equal to 25% of the minimum floor load and the area associated with the maximum floor load is
  - a. Less than or equal to 25% of the total floor plan area, use an effective  $w_F$ , calculated by computing the total force acting on the floor and dividing by the total plan area.
  - b. Greater than 25% of the floor plan, use the maximum floor load as the effective  $w_F$ .
- 2) If the difference between the minimum and maximum floor load in the bays on the floor plan is greater than 25% of the minimum floor load, either:
  - a. Use the maximum floor load as the effective  $w_F$ , or,
  - b. Divide the floor plan into sub-areas, where a sub-area is a region composed of contiguous or adjacent bays that have the same floor load. Each sub-area shall have its own longitudinal and transverse ties and peripheral ties. In addition, a peripheral tie will be placed in the boundary between the sub-areas, as shown in Figure 3-2. The required strength of the peripheral tie between the sub-areas shall be equal to the sum of the required peripheral tie force on the heavily loaded sub-area and the required peripheral tie force on the lightly loaded sub-area. In this case, the internal ties are not



required to be continuous from one side of the structure to the other but may be interrupted at the sub-area peripheral tie, providing that the internal ties from both sub-areas are properly anchored with seismic hooks to the sub-area peripheral tie. If desired, the longitudinal and transverse internal ties on the lightly loaded sub-area may be continued across the heavily loaded sub-area as part of the longitudinal and transverse internal ties of the heavily loaded side. Note that sufficient embedment or anchoring must be provided to develop the strength of all peripheral ties, at the sub-area boundary and at the exterior of the building.

- c. Note that the peripheral tie between the sub-areas may be omitted if the transverse and longitudinal ties from the heavily loaded sub-area continue across the lightly loaded sub-area and are anchored to the exterior peripheral ties. For instance, in Figure 3-2, the longitudinal ties from the heavily loaded sub-area could extend the full length of the floor plan, the transverse ties from the heavily loaded sub-area in the three left bays could extend the full width of the floor plan and transverse ties from the lightly loaded sub-area in the right two bays could extend the full width of the floor plan.

### **3-1.3.3 Cladding and Façade Loads.**

Cladding and façade loads are used for the calculation of the peripheral and vertical tie forces and are omitted for the transverse and longitudinal tie calculations.

### **3-1.4 Required Tie Strength, Distribution, and Location.**

The required tie strength, distribution, and location for longitudinal, transverse, peripheral, and vertical ties are defined in the following sub-sections for framed and load-bearing wall structures.

The design tie strengths are considered separately from the forces that are typically carried by each structural element due to live load, dead load, wind load, etc. In other words, the design tie strength of a slab, beam, column, rebar, or connection **with no other loads acting** must be greater than or equal to the required tie strength. In addition, the tie member itself, its splices and its connections only resist the calculated tensile forces. There are no structural strength or stiffness requirements to be applied to the structural members that are anchoring these horizontal tie forces.

\2\ /2/

#### **3-1.4.1 Longitudinal and Transverse Ties.**

Use the floor and roof system to provide the required longitudinal and transverse tie resistance. The structural members (beams, girders, spandrels, etc) may be used to provide some or all of the required tie forces, if they and their connections can be shown capable of carrying the total internal tie force acting over the structural

member spacing while undergoing a 0.20-rad (11.3-deg) rotation, i.e., if the required longitudinal tie force is 10-k/ft (146-kN/m) and the beams are located at 10-ft (3.05-m) on center, the designer must show that the beam can carry a tensile force of 100-k (445-kN) with rotations of 0.20-rad (11.3-deg).

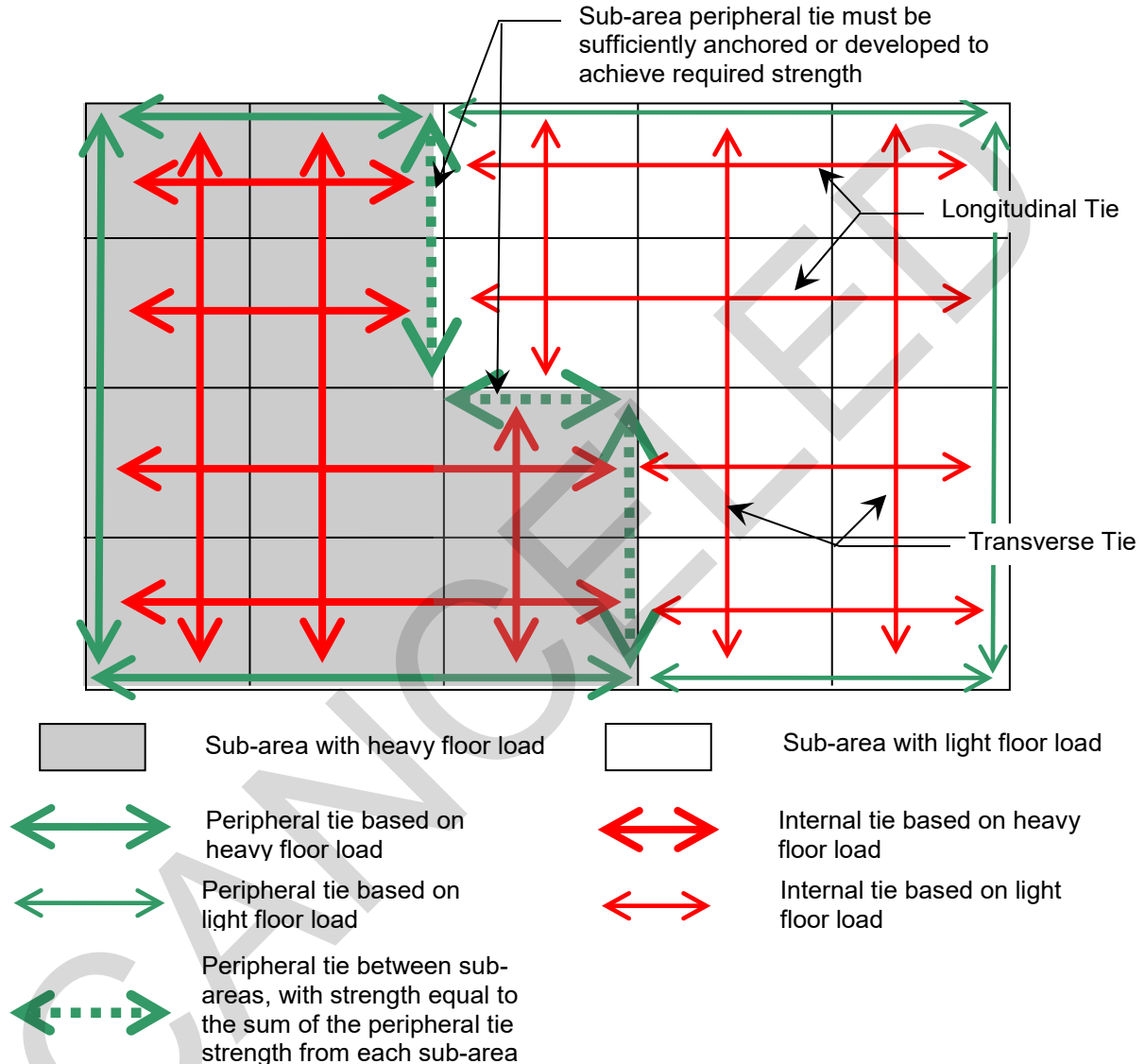
/2/ Note that longitudinal and transverse ties are not required in floors above crawlspaces if public access control is provided. See Section 1-2.3 for definition of public access control. /2/

#### **3-1.4.1.1 Framed Structures, Including Flat Plate and Flat Slab.**

Longitudinal and transverse tie forces shall be distributed orthogonally to each other throughout the floor and roof system. The longitudinal and transverse ties must be anchored to peripheral ties at each end. Spacing must not be greater than **0.2  $L_T$** , or **0.2  $L_L$**  where  $L_T$  and  $L_L$  are the greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the transverse and longitudinal directions, respectively.

/2/

Figure 3-2. Sub-areas, Peripheral and Internal Ties for Non-uniform Floor Loads



/2/

For flat plate and flat slab structures without internal beams, girders, or spandrels, place no more than twice the required tie strength (force per unit length) in the column area, which is the column strip  $0.2 L_L$  or  $0.2 L_T$  wide that is centered on the column lines and runs in the direction of the tie under consideration, as shown in Figure 3-3. For instance, if the required longitudinal tie force  $F_L$  is 10-k/ft (146-kN/m), and  $L_T$  is 20-ft (6.1-m), then the width of the column area is  $0.2 L_T$  or 4-ft (1.22-m) and the maximum total tie force allowed in the column area is  $2 \times 10\text{-k/ft} \times 0.2 \times 20\text{-ft}$  or 80-k (356-kN). The remainder of the required total tie force is distributed in the floor or roof system.

For framed buildings with internal beams, girders, or spandrels, internal ties may cross over these elements, but are not to be placed parallel to these members and within the member or within the area directly above the member, unless the member can be shown capable of a 0.20-rad (11.3-deg) rotation. The internal ties that would fall within this area must be placed on either side of the beam, so that the total required tie strength for the adjacent bays is maintained. An illustration of this restriction is shown in Figure 3-4.

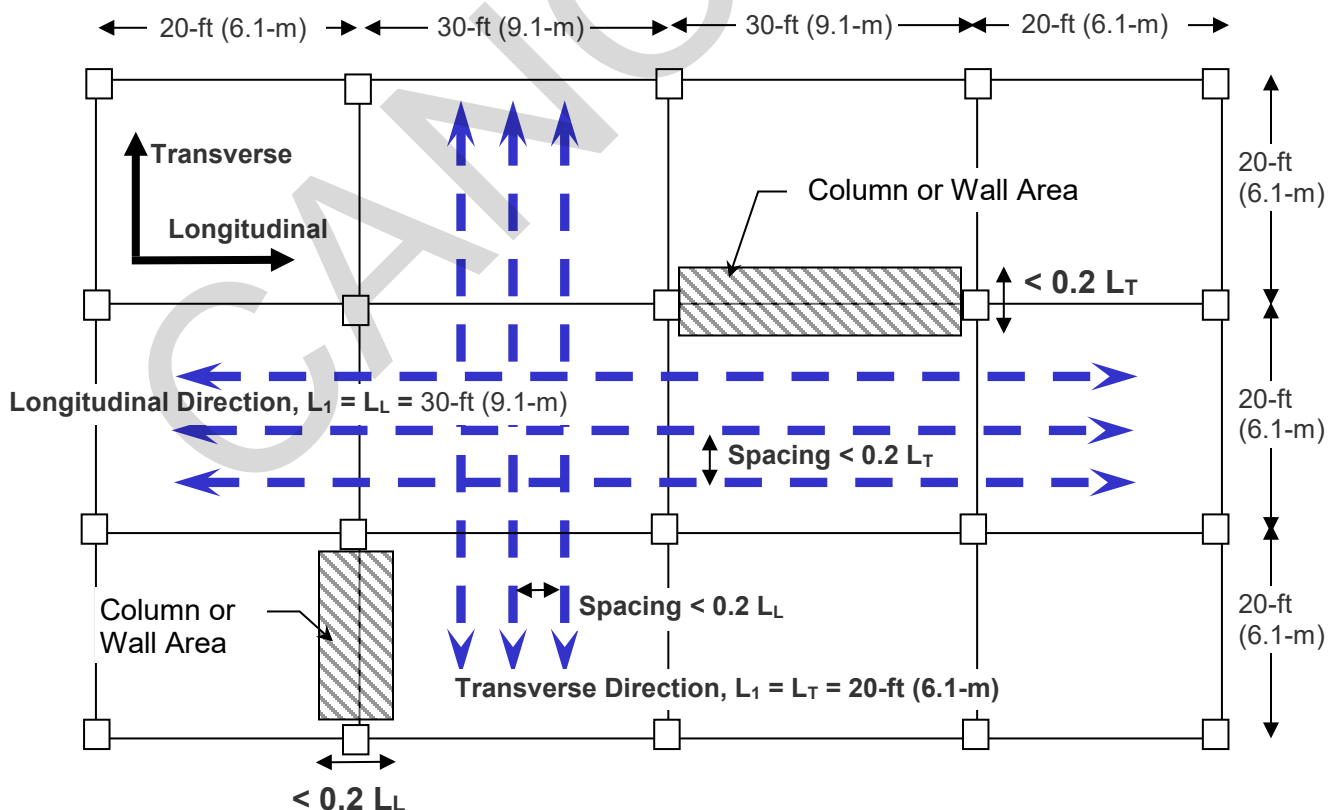
The required tie strength  $F_t$  (lb/ft or kN/m) in the longitudinal or transverse direction is:

$$F_t = 3 w_F L_1 \quad \text{Equation (3-3)}$$

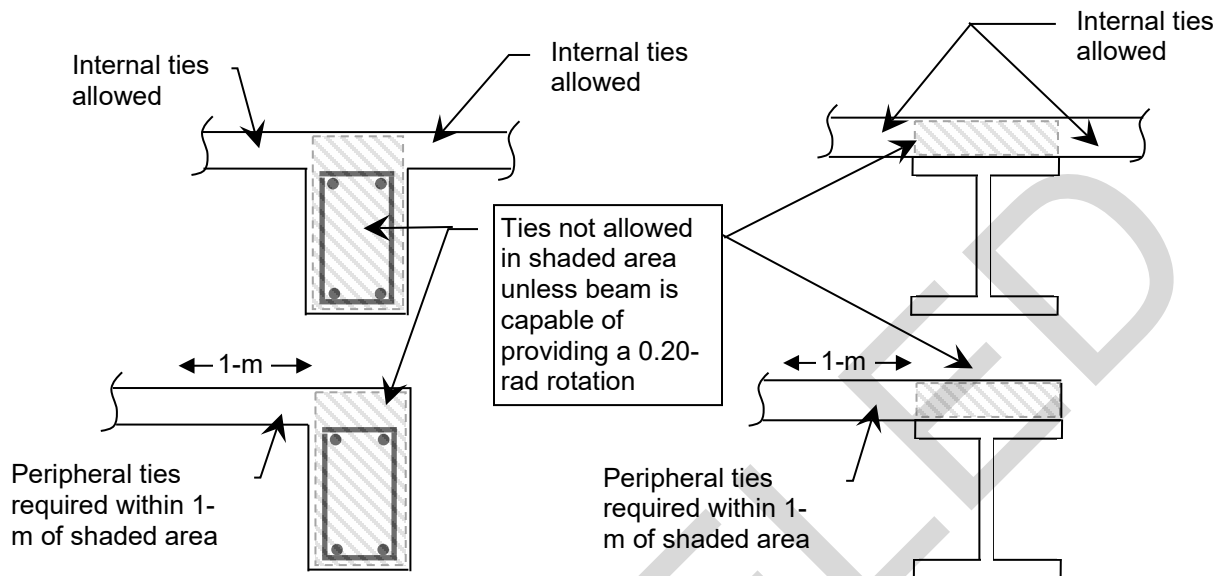
Where  $w_F$  = Floor load, determined per Section 3-1.3, in (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)  
 $L_1$  = Greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the direction under consideration (ft or m)

12\

**Figure 3-3. Determination of  $L_1$  and Column Area for Frame and Two-way Span Load-bearing Wall Construction**



**Figure 3-4. Location Restrictions for Internal and Peripheral Ties that are Parallel to the Long Axis of a Beam, Girder or Spandrel**



/2/

### 3-1.4.1.2 Load-bearing Wall Structures.

A longitudinal and a transverse tie force shall be distributed orthogonally to each other throughout the floor and roof system. These ties must be anchored to peripheral ties at each end.

For two-way spans, spacing must not be greater than  $0.2 L_T$ , or  $0.2 L_L$  where  $L_T$  and  $L_L$  are the greatest of the distances between the centers of the walls supporting any two adjacent floor spaces in the transverse and longitudinal directions, respectively; see Figure 3-3. No more than twice the required tie strength shall be placed in the wall area, which is defined as the area bounded by the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the direction of the tie under consideration and within  $0.1 L_T$  or  $0.1 L_L$  of the wall line, as appropriate, as shown in Figure 3-3. For two-way spans, the required tie strength  $F_i$  (lb/ft or kN/m) in the longitudinal or transverse direction is:

$$F_i = 3 w_F L_1 \quad \text{Equation (3-4)}$$

Where  $w_F$  = Floor load, determined per Section 3-1.3, (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)  
 $L_1$  = Greater of the distances between the centers of the columns, frames, or walls supporting any two adjacent floor spaces in the direction under consideration (ft or m)

For one-way spans, spacing of the longitudinal and transverse ties must not be greater than  $0.2 L_L$ , where  $L_L$  is the greatest of the distances between the centers of the walls supporting any two adjacent floor spaces in the longitudinal direction; see Figure 3-5. A maximum of twice the required tie strength shall be placed in the wall area, which is defined as the area within  $0.1 L_L$  of the wall line, as shown in Figure 3-5. For one-way spans,  $L_L$  is the greater of the distances between the centers of the wall in the one-way span direction, as shown in Figure 3-5. In the transverse direction  $L_T$  is  $5 h_w$ , where  $h_w$  is the clear story height. The required tie strength  $F_i$  (lb/ft or kN/m) in the longitudinal or transverse direction is

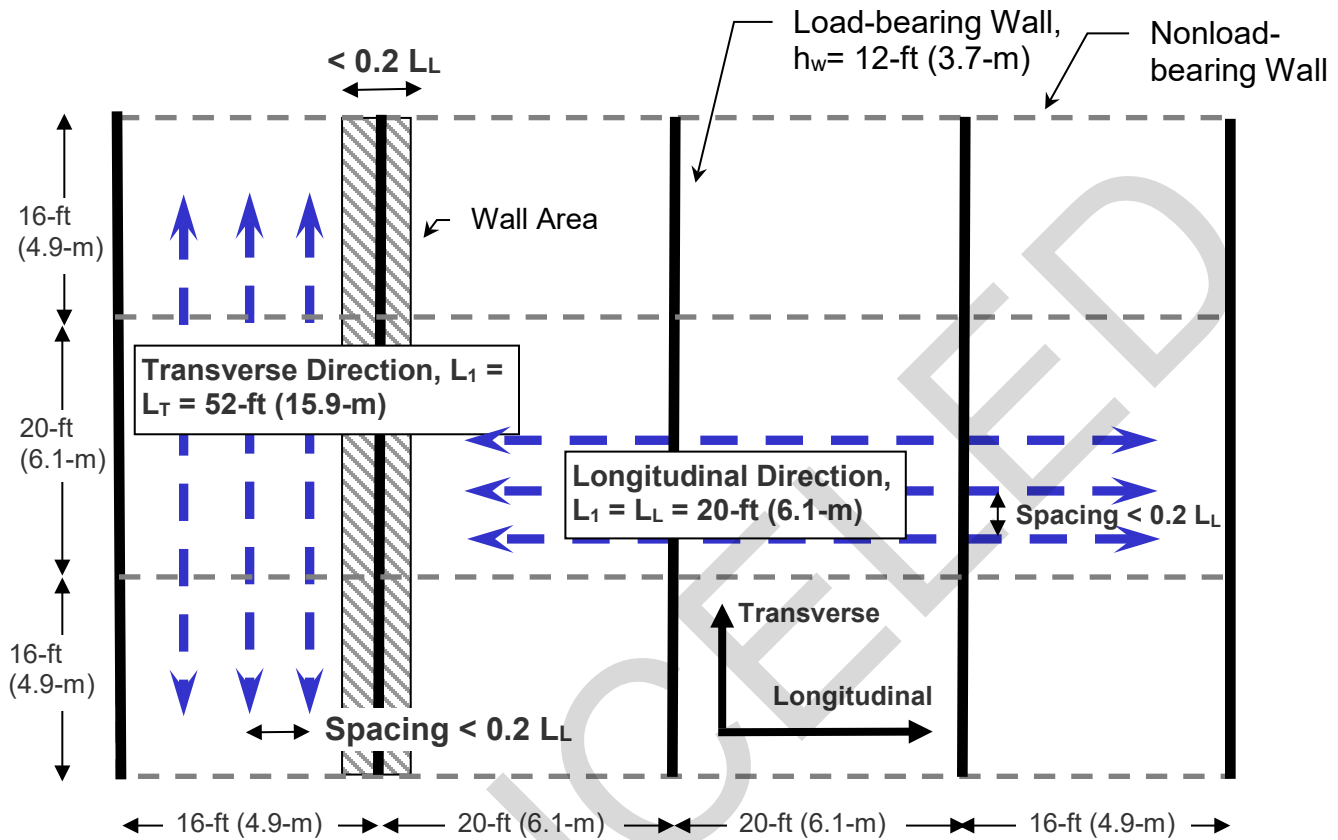
$$F_i = 3 w_F L_1 \quad \text{Equation (3-5)}$$

Where

- $w_F$  = Floor load, determined per Section 3-1.3, ((lb/ft<sup>2</sup> or kN/m<sup>2</sup>)
- $L_1$  = Either  $L_L$  = the greater of the distances between the centers of the walls supporting any two adjacent floor spaces in the longitudinal direction or  $L_T$  = lesser of  $5 h_w$  or the building width in the transverse direction (ft or m)
- $h_w$  = Clear story height (ft or m)

\2\ /2/

**Figure 3-5. Determination of  $L_1$  and Column Area for One-way Load-bearing Wall Construction with  $h_w = 12\text{-ft}$  (3.7-m)**



### 3-1.4.2 Peripheral Ties.

Use the floor and roof system to carry the required peripheral tie strength. The structural members (beams, girders, spandrels, etc) may be used instead, if they can be proven capable of carrying the peripheral tie force while undergoing a 0.20-rad (11.3-deg) rotation.

\2\ Note that peripheral ties are not required in floors above crawlspaces if public access control is provided. See Sections 1-2.1 and 1-2.3 for definitions of occupied space and public access control, respectively.

For buildings with one- and two-story sections attached to a section with three or more stories, peripheral ties shall be placed in any contiguous floors at the boundary between the short and tall sections. /2/

Place peripheral ties within 3.3-ft (1.0-m) of the edge of a floor or roof and provide adequate development or anchors at corners, re-entrant corners or changes of construction. For framed buildings with perimeter beams, girders, or spandrels, peripheral ties may not be placed parallel to these members and within the member or

within the area directly above the member, unless the member can be shown capable of a 0.20-rad (11.3-deg) rotation. If perimeter beams, girders, or spandrels are present, the 3.3-ft (1.0-m) shall be measured from the interior edge of the beam, girder or spandrel. An illustration of this restriction is shown in Figure 3-4.

\2\

### 3-1.4.2.1 Framed and Two-way Load-Bearing Wall Buildings.

For framed and two-way load-bearing wall buildings, the required peripheral tie strength  $F_p$  (lb or kN) is

$$F_p = 6 w_F L_1 L_p + 3 W_C \quad \text{Equation (3-6)}$$

Where  $w_F$  = Floor load, determined per Section 3-1.3, (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)  
 $W_C$  = 1.2 x Dead load of cladding over the length of  $L_1$ , (lb or kN), where 1.2 is the LRFD dead load factor  
 $L_1$  = For peripheral ties at the edge of the building: The greater of the distances between the centers of the columns, frames, or walls, at the perimeter of the building in the direction under consideration (ft or m).  
= For peripheral ties at openings (see Figure 3-6), the length of the bay in which the opening is located, in the direction under consideration.  
 $h_w$  = Clear story height (ft or m)  
 $L_p$  = 3.3-ft (1.0-m)

Note that the dead load includes the self-weight of the members and superimposed dead loads.

### 3-1.4.2.2 One-way Load-Bearing Wall Buildings.

For one-way load-bearing wall buildings, the required peripheral tie strength  $F_p$  (lb or kN) is

$$F_p = 6 w_F L_1 L_p + 3 W_C + 3 W_W \quad \text{Equation (3-7)}$$

Where  $w_F$  = Floor load, determined per Section 3-1.3, (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)  
 $W_C$  = 1.2 x Dead load of cladding over the length of  $L_1$ , (lb or kN), where 1.2 is the LRFD dead load factor,  
 $W_W$  = 1.2 x Dead load of wall over the length of  $h_w$ , where 1.2 is the LRFD dead load factor,  
 $h_w$  = Clear story height (ft or m)



- $L_1$  = For exterior peripheral ties perpendicular to the load-bearing walls (longitudinal direction in Figure 3-5), the greatest of the distances between the centers of the walls in the direction under consideration (m or ft).
  - = For exterior peripheral ties parallel to the load-bearing walls (transverse direction in Figure 3-5),  $2 h_w$  (m or ft).
  - = For peripheral ties at openings (see Figure 3-6), the length of the bay in which the opening is located, in the direction under consideration (ft or m)
- $L_p$  = 3.3-ft (1.0-m)

Notes: 1) For the end load-bearing walls, only the wall load is acting, as the end wall is the façade, and  $W_c = 0$ .; 2) The dead load includes the self-weight of the members and superimposed dead loads; 3) See Commentary C-5.4.2 for additional information.  
/2/

### 3-1.4.3 Vertical Ties.

\2\ Use the columns and load-bearing walls to carry the required vertical tie strength. Each column and load-bearing wall shall be tied continuously from the roof level down to the first column- or wall-supported floor above the foundation, i.e., the vertical ties are not required to extend to the foundation. Vertical ties shall be straight. /2/

The vertical tie must have a design strength in tension equal to the largest vertical load received by the column or wall from any one story, using the tributary area and the floor load  $w_F$  as determined in Section 3-1.3.

### 3-1.5 Continuity of Ties.

\2\ The load path for peripheral ties must be continuous between building corners and edge of openings. /2/ For internal longitudinal and transverse ties, the path must be continuous from one edge to the other. However, interruptions due to courtyards, mezzanines, elevator/stairwell cores, etc, are allowed, as shown in Figure 3-6, when a peripheral tie is placed at the interruption; \2\ note that peripheral ties around an opening are not required if the opening fits between the longitudinal and transverse ties that meet the spacing requirements of Sections 3-1.4.1.1 and 3-1.4.1.2./2/ Insure that sufficient embedment or anchoring is provided to develop the strength of the peripheral ties placed at the interruption; anchor the longitudinal and transverse ties to the peripheral ties with seismic hooks. Along a particular load path, different structural elements may be used to provide the required tie strength, providing that they are adequately connected.

Each column and load-bearing wall shall be tied continuously from the roof level down to the first column- or wall-supported floor above the foundation, i.e., the vertical ties are not required to extend to the foundation.

Re-entrant corners are allowed for all types of construction, providing that the transverse, longitudinal, and peripheral ties are adequately developed and anchored, per Section 3-1.6.

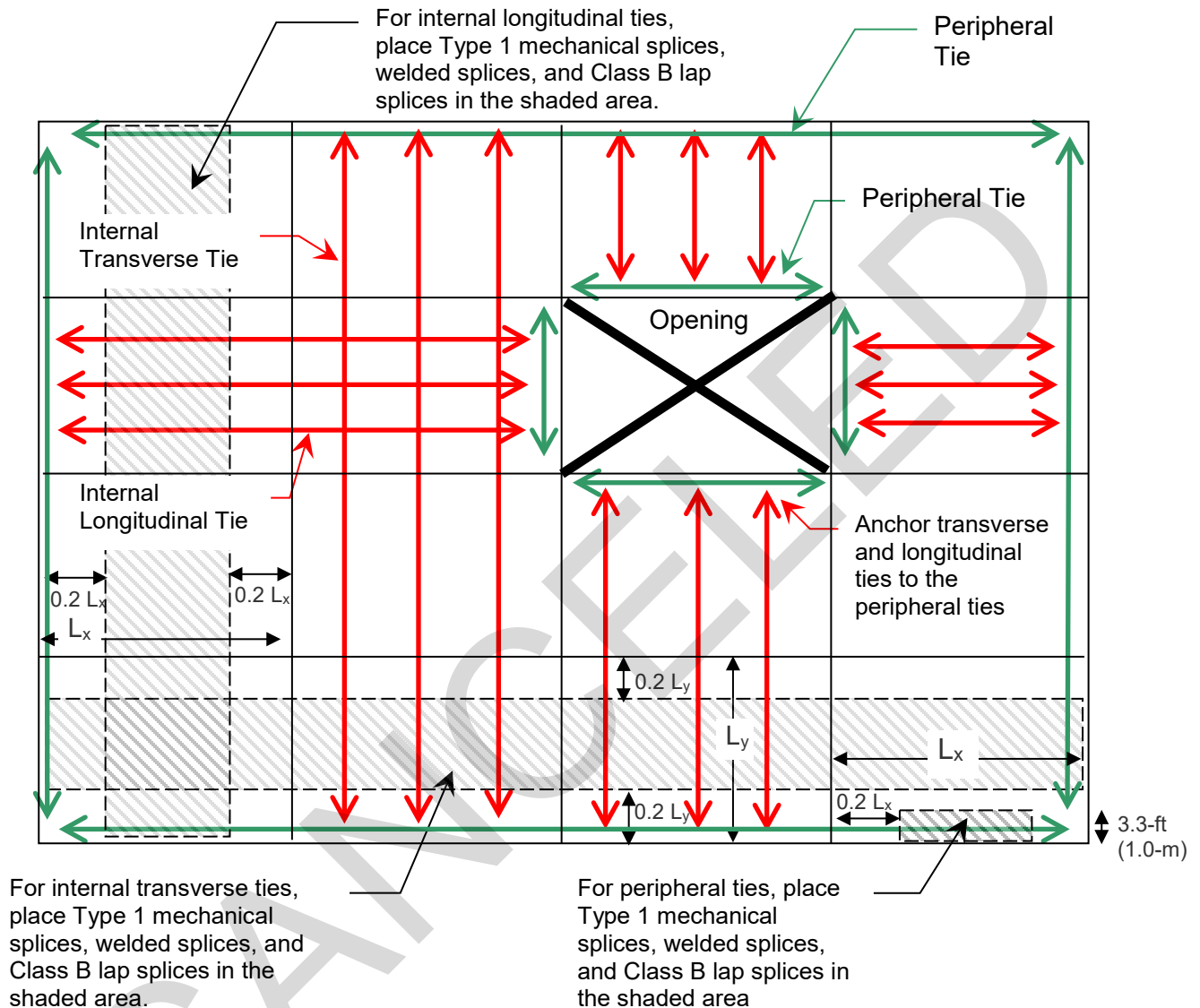
### **3-1.6 Splices, Anchorage, and Development of Ties.**

#### **3-1.6.1 Cast-in-Place Reinforced Concrete Floor and Roof Systems.**

For cast-in-place construction, including composite construction with sheet metal decks and reinforced concrete topping, and, for precast floor systems with a concrete topping and reinforcement, splices in steel reinforcement used to provide the design tie strength shall be lapped with Class B lap splices, welded, or mechanically joined with Type 1 or Type 2 mechanical splices, per ACI 318. Splices shall be staggered within the allowable shaded areas shown in Figure 3-6.

Type 2 mechanical splices may be used at any location in the slab. For internal longitudinal and transverse ties, Type 1 mechanical splices, welded splices, and Class B lap splices shall be located no closer than 20% of the bay spacing in the direction of the tie to any vertical load carrying elements (i.e., these splices must be within the middle 60% of the slab or floor or roof system bay, in the direction of the tie); see Figure 3-6. For peripheral ties, Type 1 mechanical splices, welded splices, and Class B lap splices shall be placed no closer than 20% of the span distance in the peripheral tie direction; see Figure 3-6. Note that noncontact splices are not allowed.

**Figure 3-6. Splice Locations and Interruptions in Internal Tie Forces**



Use seismic hooks as defined in ACI 318 to anchor and connect internal longitudinal and transverse ties to the peripheral ties; when multiple rebar are used as the peripheral tie, the seismic hooks on the longitudinal and transverse ties shall engage the furthest peripheral tie rebar (the rebar closest to the edge of the structure). The strength of the peripheral ties shall be developed within the 3.3-ft (1.0-m) wide area at the edge of the floor or roof, using development lengths from Chapter 12 of ACI 318. At re-entrant corners or at substantial changes in construction, take care to insure that the transverse, longitudinal, and peripheral ties are adequately anchored and developed. /2/

### **3-1.6.2 Precast Concrete Floor and Roof Systems.**

For precast concrete floor and roof systems, the rebar within the precast planks may be used to provide the internal tie forces, providing the rebar is continuous across the structure and properly anchored; this may be difficult to accomplish in the short direction of the plank. Also, the rebar may be placed within a concrete topping; in this case, provide positive mechanical engagement between the reinforcement and the precast floor system, with sufficient strength to insure that the precast units do not separate from the topping and fall to the space below. Do not rely on the bond strength between the topping and precast units, as the bond can be disrupted by the large deformations associated with catenary behavior. This attachment between the rebar in the concrete topping and the precast planks may be accomplished with hooks, loops or other mechanical attachments that are embedded in the precast floor units.

### **3-1.6.3 Composite Construction Floor and Roof Systems.**

If composite construction with steel decks and concrete topping is employed, provide sufficient connection between the steel beam and the composite floor such that the beam will not fall to the space below.

### **3-1.6.4 Other Floor and Roof Systems and Structural Elements.**

If other floor and roof systems and structural elements can be shown capable of carrying the tie forces required in Sections 3-1.4.1 and 3-1.4.2 while undergoing a rotation of 0.20-rad (11.3-deg) and while meeting the continuity requirement of Section 3-1.5, provide adequate splicing and anchorage that allows development of the transverse, longitudinal, and peripheral tie forces required in Section 3-1.6.

### **3-1.7 Structural Elements and Connections with Inadequate Tie Strength**

If the vertical design tie strength of any structural element or connection is less than the vertical required tie strength, the designer must either: 1) revise the design to meet the tie force requirements or 2) use the Alternate Path method to prove that the structure is capable of bridging over this deficient element.

The AP method shall not be applied to structural elements or connections that cannot provide the required longitudinal, transverse, or peripheral tie strength; in this case, the designer must redesign or retrofit the element and connection such that a sufficient design tie strength is developed.

## **3-2 ALTERNATE PATH METHOD.**

The Alternate Path method is used in two situations: 1) for Option 1 of \3\ Risk Category /3/ II and for \3\ Risk Category /3/ IV, when a vertical structural element cannot provide the required tie strength, the designer may use the AP method to determine if the structure can bridge over the deficient element after it has been notionally removed, and 2) for \3\ Risk Category /3/ II Option 2, \3\ Risk Category /3/ III, and \3\ Risk

Category /3/ IV, the AP method must be applied for the removal of specific vertical load-bearing elements which are prescribed in Section 3-2.9.

### **3-2.1 General.**

This method follows the general LRFD philosophy by employing a modified version of the ASCE 7 load factor combination for extraordinary events and resistance factors to define design strengths. Three analysis procedures are employed: Linear Static (LSP), Nonlinear Static (NSP) and Nonlinear Dynamic (NDP). These procedures follow the general approach in ASCE 41 with modifications to accommodate the particular issues associated with progressive collapse. Much of the material-specific criteria from Chapters 9 to 12 of ASCE 41 are explicitly adopted in Chapters 4 to 8 of this document. The topics of each ASCE 41 Chapter are:

- Steel or cast iron, ASCE 41 Chapter 9,
- Reinforced concrete, ASCE 41 Chapter 10,
- Reinforced or un-reinforced masonry, ASCE 41 Chapter 11,
- Timber, light metal studs, gypsum, or plaster products, ASCE 41 Chapter 12.

Note that some of the deformation and strength criteria in ASCE 41 Chapters 9 to 12 have been superseded by requirements that are specified in the material specific Chapters 4 to 8 in this UFC.

### **3-2.2 Alternative Rational Analysis.**

For the performance of the Alternate Path analysis and design, nothing in this document shall be interpreted as preventing the use of any alternative analysis procedure that is rational and based on fundamental principles of engineering mechanics and dynamics. For example, simplified analytical methods employing hand calculations or spreadsheets may be appropriate and more efficient for some types of buildings, such as load-bearing wall structures.

The results of any alternative rational analyses shall meet the acceptance criteria contained in Section 3-2.10 and in Chapters 4 through 8. The analyses shall include the specified locations for removal of columns and load-bearing walls in Section 3-2.9 and the ASCE 7 extreme event load combination, with the load increase factors in Sections 3-2.11.5 and 3-2.12.5 for linear static and nonlinear static analyses, respectively. The designer shall verify that these criteria are applicable to the alternative rational analyses. If a Linear Static approach is employed, the requirements of Section 3-2.11.1 must be met. All projects using alternative rational analysis procedures shall be reviewed and approved by an independent third-party engineer or by an authorized representative of the facility owner.

### 3-2.3 Load and Resistance Factor Design for Alternate Path Method.

The Alternate Path method employed in this UFC follows the general philosophy of the standard LRFD approach but with modifications to facilitate the integration of the ASCE 41 procedures, which are not LRFD. For LRFD, the design strength is taken as the product of the strength reduction factor  $\phi$  and the nominal strength  $R_n$  calculated in accordance with the requirements and assumptions of applicable material specific codes. The design strength must be greater than or equal to the required strength:

$$\phi R_n \geq R_u \quad \text{Equation (3-8)}$$

where

$\phi R_n$	= Design strength
$\phi$	= Strength reduction factor
$R_n$	= Nominal strength
$R_u$	= $\sum \gamma_i Q_i$ = Required strength
$\gamma_i$	= Load factor
$Q_i$	= Load effect

Items to note relative to the integration of the LRFD and the ASCE 41 approaches: /2/

- While ASCE 41 requires that all  $\phi$  factors be taken as unity, this UFC requires that strength reduction factors,  $\phi$ , be used as specified in the appropriate material specific code, for the action or limit state under consideration.
- ASCE 41 uses the term “action” in the way LRFD defines “required strength”. ASCE 41 further differentiates actions into “deformation-controlled” and “force-controlled”. These terms are defined later.
- In this UFC, the LRFD “nominal strength” is defined as either the “expected strength” when deformation actions are being considered or the “lower-bound strength” for force-controlled actions; ASCE 41 sets all  $\phi$  factors to 1 and therefore, the expected and lower bound strengths are the nominal strengths in this document.
- This UFC and ASCE 41 both employ the same “over-strength factors” to translate lower bound material properties to expected strength material properties. The over-strength factors are provided in ASCE 41 Tables 9-3 (structural steel), 10-4 (reinforced concrete), and 11-2 (masonry). For wood and cold-formed steel, Chapter 12 of ASCE 41 provides default expected strength values; note that for wood construction, a time effect factor  $\lambda$  is also included.

Note that live load reductions (LLRs) per ASCE 7 are permitted for all live loads used in Alternate Path analysis and design. For framed structures, where the floor slab is supported by beams and girders, the analyst may use the LLR for each beam individually or may use the same LLR for the entire structure. In the latter case, the LLR shall be equal to the smallest LLR (greatest live load) for any beam in the bays above the column removal location. For flat-slab structures, load-bearing wall structures and other situations where the floor system transfers loads directly to the columns or walls, the LLR shall be computed for, and applied to, the floor in each bay. In all cases, the LLRs shall be based on the structural configuration before the column or load-bearing wall section is removed. /2/

### **3-2.4 Primary and Secondary Components.**

Designate all structural elements and components as either primary or secondary. Classify structural elements and components that provide the capacity of the structure to resist collapse due to removal of a vertical load-bearing element as primary. Classify all other elements and components as secondary. For example, a steel gravity beam may be classified as secondary if it is assumed to be pinned at both ends to girders and the designer chooses to ignore any flexural strength at the connection; if the connection is modeled as partially restrained and thus contributes to the resistance of collapse, it is a primary member.

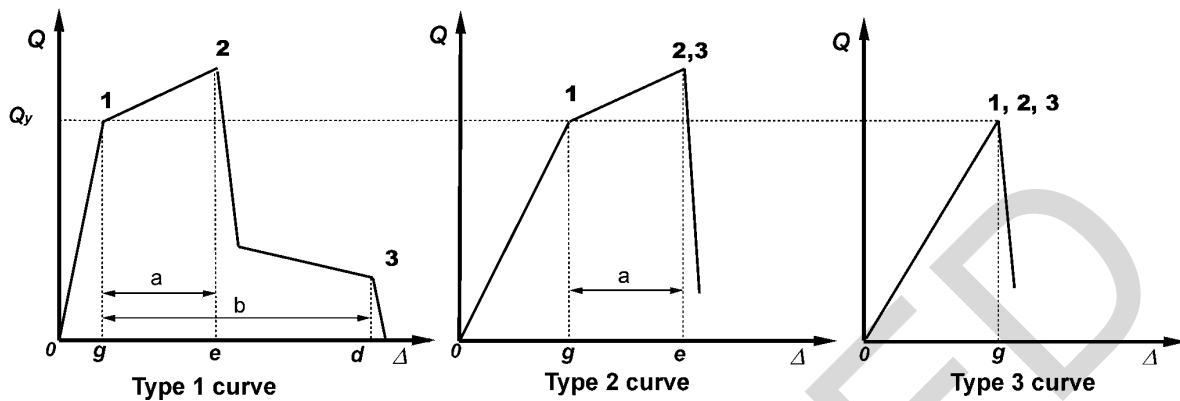
### **3-2.5 Force and Deformation Controlled Actions.**

Classify all actions as either deformation controlled or force-controlled using the component force versus deformation curve shown in Figure 3-7. Examples of deformation and force controlled actions are listed in Table 3-1. Note that a component might have both force and deformation controlled actions. Further, classification as a force or deformation controlled action is not up to the discretion of the user and must follow the guidance presented here.

Define a primary component action as deformation controlled if it has a Type 1 curve and  $e \geq 2g$ , or, it has a Type 2 curve and  $e \geq 2g$ . Define a primary component action as force-controlled if it has a Type 1 or Type 2 curve and  $e < 2g$ , or, if it has a Type 3 curve.

Define a secondary component action as deformation controlled if it has a Type 1 curve for any  $e/g$  ratio or if it has a Type 2 curve and  $e \geq 2g$ . Define a secondary component action as force controlled if it has a Type 2 curve and  $e < 2g$ , or, if it has a Type 3 curve.

Figure 3-7. Definition of Force Controlled and Deformation Controlled Actions, from ASCE 41



### 3-2.6 Expected and Lower Bound Strength.

When evaluating the behavior of deformation controlled actions, use the expected strength,  $Q_{CE}$ .  $Q_{CE}$  is defined as the statistical mean value of the strength,  $Q$  (yield, tensile, compressive, etc., as appropriate), for a population of similar components, and includes consideration of the variability in material strengths as well as strain hardening and plastic section development. Note that  $Q_{CE}$  relates to any deformation-controlled action presented in Table 3-1, e.g., the expected strength for the moment in a deformation-controlled, laterally braced beam would be  $Q_{CE} = M_{CE} = Z F_{YE}$ , where  $Z$  is the plastic section modulus and  $F_{YE}$  is the expected yield strength. If a database to determine  $F_{YE}$  is not available,  $F_{YE}$  is obtained by multiplying the lower-bound strength  $F_{YL}$  (the nominal strength or strength specified in the construction documents) by the appropriate factor from Chapters 9 to 12 in ASCE 41, as discussed in Section 3-2.7.

When evaluating the behavior of force-controlled actions, use a lower bound estimate of the component strength,  $Q_{CL}$ .  $Q_{CL}$  is defined as the statistical mean minus one standard deviation of the strength,  $Q$  (yield, tensile, compressive, etc., as appropriate), for a population of similar components. Note that  $Q_{CL}$  relates to any force-controlled action presented in Table 3-1, e.g., the lower bound strength of a steel column under axial compression would be  $Q_{CL} = P_{CL}$ , where  $P_{CL}$  is based on the lowest value obtained for the limit states of column buckling, local flange buckling, or local web buckling, calculated with the lower bound strength,  $F_{YL}$ . Where data to determine the lower bound strength are not available, use the nominal strength or strength specified in the construction documents.

### 3-2.7 Material Properties.

Expected material properties such as yield strength, ultimate strength, weld strength, fracture toughness, elongation, etc., shall be based on mean values of tested material properties. Lower bound material properties shall be based on mean values of tested material properties minus one standard deviation.



If data to determine the lower bound and expected material properties do not exist, use nominal material properties, or properties specified in construction documents, as the lower bound material properties unless otherwise specified in Chapters 9 through 12 of ASCE 41. Calculate the corresponding expected material properties by multiplying lower bound values by appropriate factors specified in Chapters 9 through 12 of ASCE 41 to translate from lower bound material properties to expected material values. If factors for converting from a lower bound to expected material property are not specified, use the lower bound material property as the expected material property.

**Table 3-1. Examples of Deformation Controlled and Force Controlled Actions, from ASCE 41**

Component	Deformation Controlled Action	Force Controlled Action
Moment Frames <ul style="list-style-type: none"> <li>• Beams</li> <li>• Columns</li> <li>• Joints</li> </ul>	Moment (M) M --	Shear (V) Axial load (P), V V <sup>1</sup>
Shear Walls	M, V	P
Braced Frames <ul style="list-style-type: none"> <li>• Braces</li> <li>• Beams</li> <li>• Columns</li> <li>• Shear Link</li> </ul>	P -- -- V	-- P P P, M
Connections	P, V, M <sup>2</sup>	P, V, M

1. Shear may be a deformation controlled action in steel moment frame construction.

2. Axial, shear, and moment may be deformation controlled actions for certain steel and wood connections.

### 3-2.8 Component Force and Deformation Capacities.

Methods for calculation of individual component strengths and deformation capacities shall comply with the requirements in the individual ASCE 41 material chapters.

As shown in the acceptance criteria given in Sections 3-2.11.7, 3-2.12.7 and 3-2.13.6, the expected and lower-bound strengths shall be multiplied by the strength reduction factors that are specified in the material specific design codes (i.e.,

the  $\phi$  factors in ACI 318, the AISC Steel Construction Manual, etc). Note that  $\phi$  factors are taken as 1.0 in ASCE 41. /2/

### 3-2.8.1 Component Capacities for Nonlinear Procedures.

For nonlinear procedures, component capacities for deformation controlled actions shall be taken as permissible inelastic deformation limits, and component capacities for force-controlled actions shall be taken as lower-bound strengths,  $Q_{CL}$ , multiplied by the appropriate strength reduction factor  $\phi$ , as summarized in Table 3-2.

**Table 3-2. Calculation of Component Capacities for Nonlinear Static and Nonlinear Dynamic Procedures**

Parameter	Deformation Controlled	Force Controlled
Deformation Capacity	Deformation limit	N/A
Strength Capacity	N/A	$\phi Q_{CL}$

### 3-2.8.2 Component Capacities for the Linear Static Procedure.

For the linear static procedure, component capacities for deformation-controlled actions shall be defined as the product of ***m-factors*** and expected strengths,  $Q_{CE}$ , multiplied by the appropriate strength reduction factor  $\phi$ . Capacities for force-controlled actions shall be defined as lower-bound strengths,  $Q_{CL}$ , multiplied by the appropriate strength reduction factor  $\phi$ , as summarized in Table 3-3.

**Table 3-3. Calculation of Component Capacities for the Linear Static Procedure**

Parameter	Deformation Controlled	Force Controlled
Material Strength	Expected Material Strength	Lower Bound Strength
Strength Capacity	$\phi m Q_{CE}$	$\phi Q_{CL}$

### 3-2.9 Removal of Load-Bearing Elements for the Alternate Path Method.

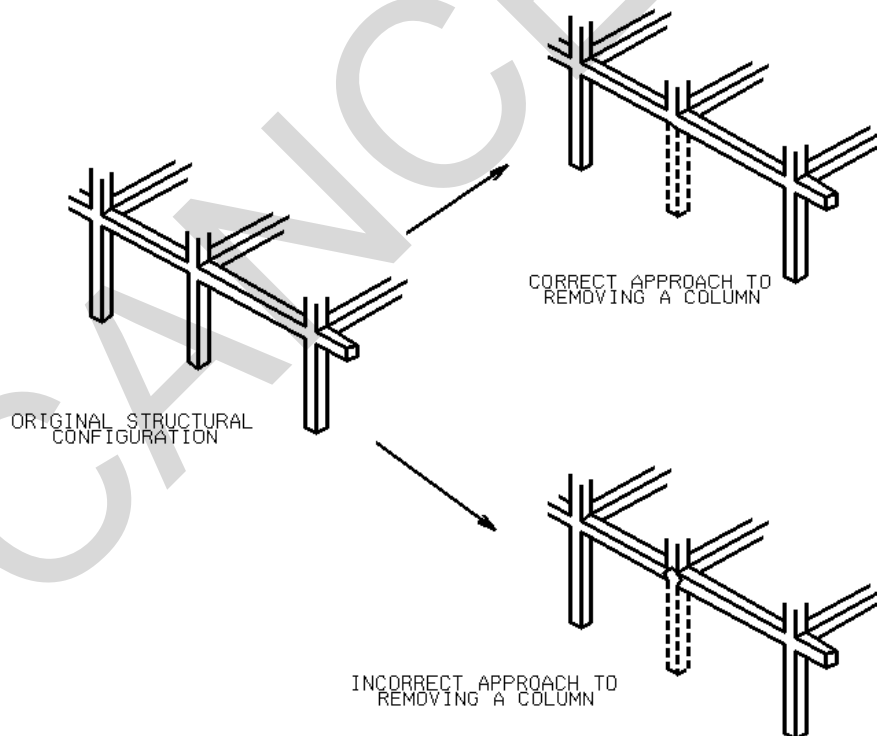
Load-bearing elements are removed for the following two cases:

- 1) For RC II Option 1 and RC IV structures, where an element cannot provide the required vertical tie strength,
- 2) For RC II Option 2, RC III, and RC IV structures, where AP is applied to elements for which the location and size are specified to verify that the structure has adequate flexural resistance to bridge over the missing element.

For both external and internal column removal, for the purposes of AP analysis, beam-to-beam continuity is assumed to be maintained across a removed column; see Figure 3-8.

The details of the size and location of the removed load-bearing elements are described in the following sub-paragraphs.

**Figure 3-8. Removal of Column from Alternate Path Model**



### **3-2.9.1 Extent of Removed Load-Bearing Elements.**

#### **3-2.9.1.1 RC II Option 1 (Deficient Vertical Tie Force).**

For each column that cannot provide the required vertical tie force, remove the clear height between lateral restraints.

For each load-bearing wall that cannot provide the required vertical tie force, the length of the removed section of wall is twice the clear story height  $H$ , if the length of the deficient wall is greater than  $2H$ . If the length of the deficient wall is less than  $2H$ , remove just that portion of deficient wall. In both instances, remove the clear height between lateral restraints. Note that discontinuities, such as joints, segmented walls, or openings for doors or windows, can be located within the  $2H$  length, providing that the loads above that discontinuity are carried by the remainder of the wall in the  $2H$  length.

#### **3-2.9.1.2 RC II Option 2, RC III, and RC IV**

For each column, remove the clear height between lateral restraints.

For each load-bearing wall, remove a length that is twice the clear story height  $H$ . Remove the clear height between lateral restraints. Note that only planar sections of wall are removed, i.e., if a shear wall has a C-shaped cross-section in plan, only the flange or only the web are removed, but not both. However, for external corners, where one or both of the intersecting walls is load bearing, remove a length of wall equal to the clear story height  $H$  in each direction

### **3-2.9.2 Location of Removed Load-Bearing Elements.**

#### **3-2.9.2.1 RC II Option 1 (Deficient Vertical Tie Force).**

Remove the column that cannot provide the required vertical tie force.

For a deficient load-bearing wall or section thereof that is longer than  $2H$ , determine the location(s) for removal by using the guidance for wall removal locations provided in Sections 3-2.9.2.4 and 3-2.9.2.5. Additionally, use engineering judgment to shift the location of the removed  $2H$  section of wall within the length of deficient wall to evaluate worse case scenarios. If the length of the deficient load-bearing wall is less than  $2H$ , remove just the section that is deficient.

#### **3-2.9.2.2 RC II Option 2, RC III and IV External Columns.**

For RC II Option 2, RC III and RC IV, as a minimum, remove external columns near the middle of the short side, near the middle of the long side, and at the corner of the building, as shown in Figure 3-9. Also remove columns at locations where the plan geometry of the structure changes significantly, such as abrupt decrease in bay size or re-entrant corners, or, at locations where adjacent columns are lightly loaded, the bays

have different tributary sizes, and members frame in at different orientations or elevations. Use engineering judgment to identify these critical column locations.

\2\If any other column is within a distance of 30% of the largest dimension of the associated bay from the column removal location, it must be removed simultaneously. /2/

For each plan location defined for element removal, perform AP analyses for:

1. First story above grade
2. Story directly below roof
3. Story at mid-height
4. Story above the location of a column splice or change in column size

For example, if a corner column is specified as the removed element location in a ten story building with a column splice at the third story, one AP analysis is performed for removal of the ground story corner column; another AP analysis is performed for the removal of the corner column at the tenth story; another AP analysis is performed for the fifth story corner column (mid-height story) and one AP analysis is performed for the fourth story corner column (story above the column splice).

#### **3-2.9.2.3 RC II Option 2, RC III and RC IV Internal Columns.**

For RC II Option 2, RC III and RC IV structures with underground parking or areas of uncontrolled public access, remove internal columns near the middle of the short side, near the middle of the long side and at the corner of the uncontrolled space, as shown in Figure 3-10; see Section 1-2.3 for a definition of controlled public access. The removed column extends from the floor of the underground parking area or uncontrolled public floor area to the next floor (i.e., a one story height must be removed). Internal columns must also be removed at other critical locations within the uncontrolled public access area, as determined with engineering judgment. For each plan location, the AP analysis is only performed for the story with the parking or uncontrolled public area.

\2\If any other column is within a distance of 30% of the largest dimension of the associated bay from the column removal location, it must be simultaneously removed as well. /2/

#### **3-2.9.2.4 RC II Option 2, RC III and RC IV External Load-Bearing Walls.**

As a minimum, remove external load-bearing walls near the middle of the short side, near the middle of the long side and at the corner of the building, as shown in Figure 3-11. For external corners, where one or both of the intersecting walls is load bearing, remove a length of wall equal to the clear story height  $H$  in each direction. Also remove load-bearing walls at locations where the plan geometry of the structure changes significantly, such as at an abrupt decrease in bay size or at re-entrant corners, as well as at locations where adjacent walls are lightly loaded, the bays have different sizes,

and members frame in at different orientations or elevations. Use engineering judgment to identify these critical locations. The length of the removed wall section is specified in Section 3-2.9.1. The designer must use engineering judgment to shift the location of the removed wall section by a maximum of the clear story height  $H$  if that creates a worst-case scenario.

For each plan location defined for element removal, perform AP analyses for the following stories:

1. First story above grade
2. Story directly below roof
3. Story at mid-height
4. Story above the location of a change in wall size

For example, if a wall section at the middle of the long side is specified as the removed element location in a six story wood building with a change in wall framing at the third story, one AP analysis is performed for removal of the ground story wall section; another AP analysis is performed for the removal of the wall section at the sixth story; another AP analysis is performed for the third story (mid-height story) and one AP analysis is performed for the fourth story (story above the change in wall framing).

If any other load-bearing element such as a column is within a distance of 30% of the clear story height ( $H$ ) behind or in front of the wall removal location, it must be removed simultaneously.

Figure 3-9. Location of External Column Removal for RC II Option 2, RC III and IV Structures

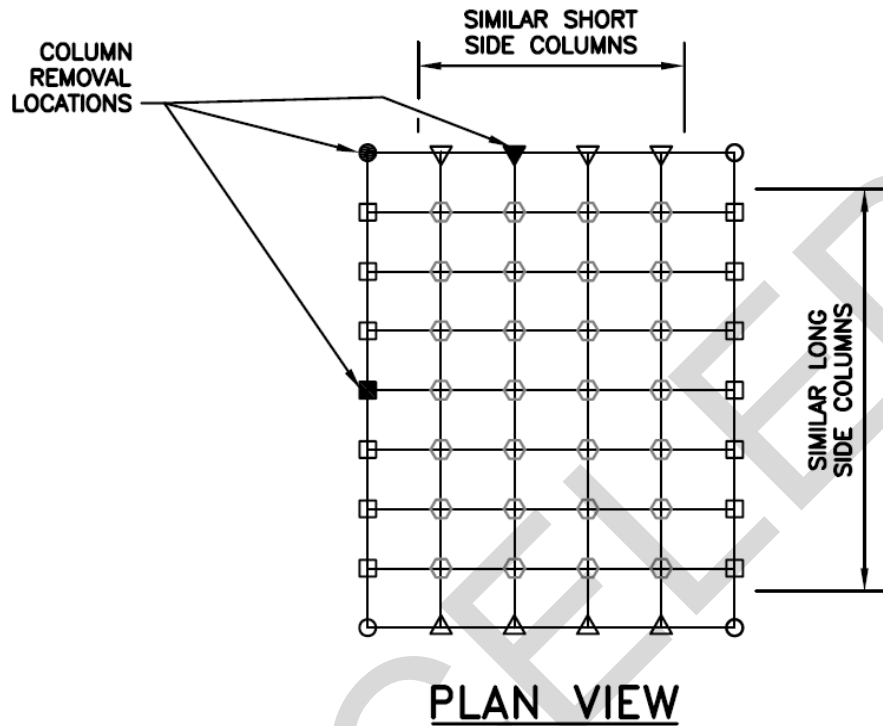


Figure 3-10. Location of Internal Column Removal for RC II Option 2, RC III and IV Structures

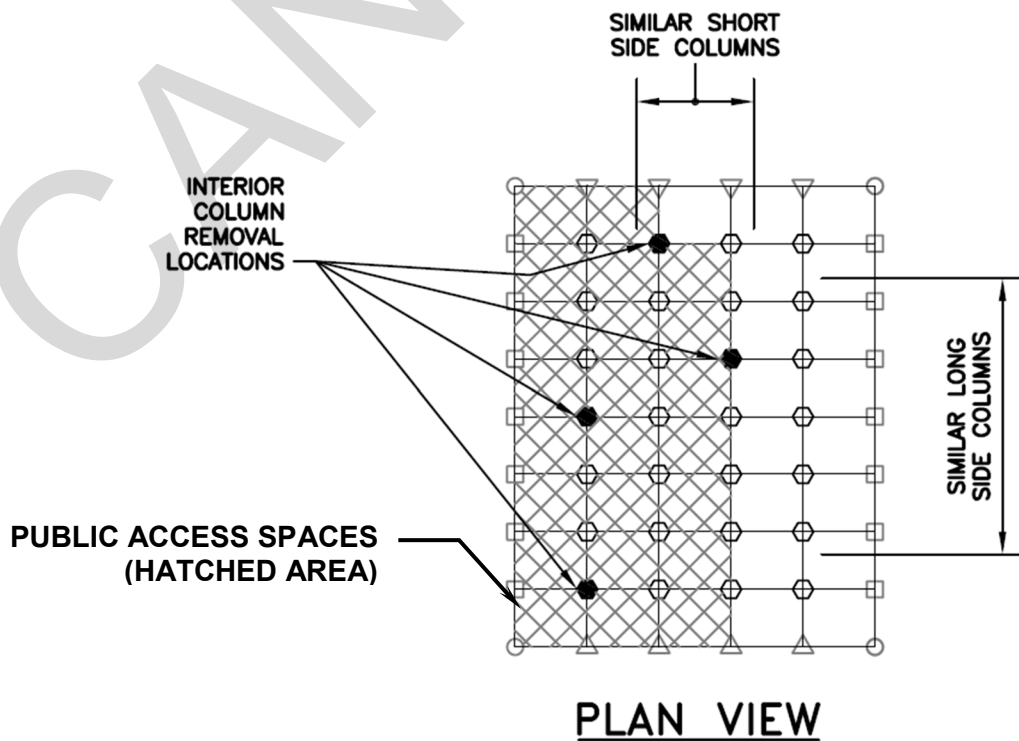
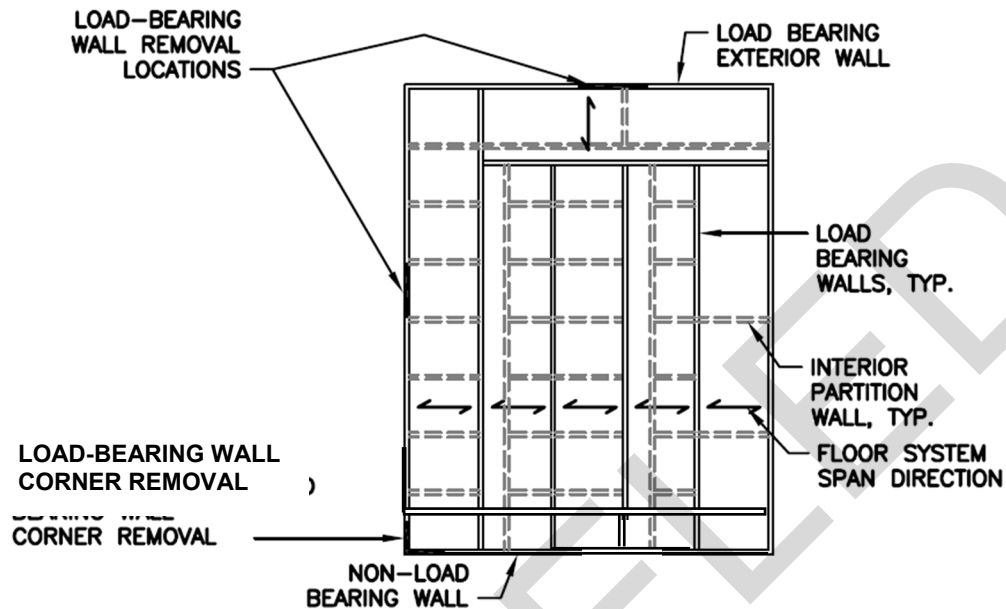
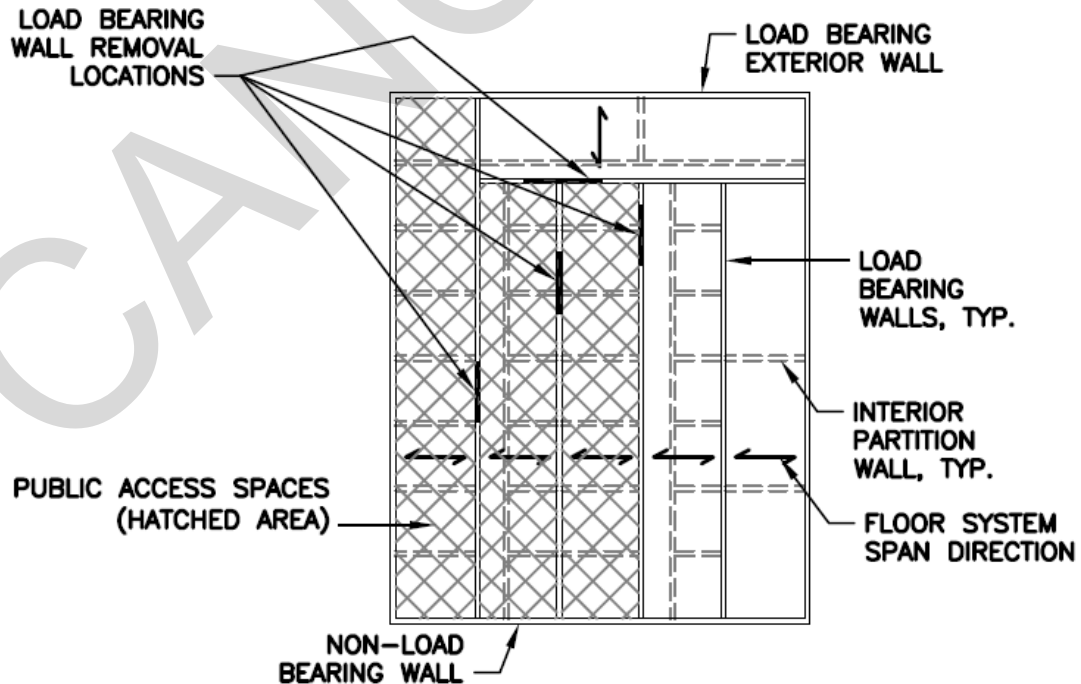


Figure 3-11. Location of External Load-Bearing Wall Removal for RC II Option 2, RC III and RC IV Structures



### PLAN VIEW

Figure 3-12. Location of Internal Load-Bearing Wall Removal for RC II Option 2, RC III and RC IV Structures





### **3-2.9.2.5 RC II Option 2, RC III and RC IV Internal Load-Bearing Walls.**

For structures with underground parking or areas of uncontrolled public access, remove internal load-bearing walls near the middle of the short side, near the middle of the long side and at the corner of the uncontrolled space, as shown in Figure 3-12; see Section 1-2.3 for a definition of controlled public access. For internal corners, where one or both of the intersecting walls is load-bearing, remove a length of wall equal to the clear story height  $H$  in each direction. The removed wall extends from the floor of the underground parking area or uncontrolled public floor area to the next floor (i.e., a one story height must be removed). Also remove internal load-bearing walls at other critical locations within the uncontrolled public access area, as determined with engineering judgment. For each plan location, the AP analyses are only performed for the load-bearing walls at the story with parking area or uncontrolled public space, and not for all stories in the structure. The length of the removed wall section is specified in Section 3-2.9.1. The designer must use engineering judgment to shift the location of the removed wall section by a maximum of the wall height if that creates a worst-case scenario. If any other load-bearing element such as column is within a distance of 30% of the clear story height ( $H$ ) behind or in front of the wall removal location, it must be removed simultaneously.

### **3-2.10 Structure Acceptance Criteria.**

For all three analysis types (LS, NS, and ND), the building is structurally adequate if none of the primary and secondary elements, components, or connections exceeds the acceptance criteria, in Paragraphs 3-2.11.7, 3-2.12.7, and 3-2.13.6, as appropriate. If the analysis predicts that any element, component, or connection does not meet these acceptance criteria, the building does not satisfy the progressive collapse requirements and must be re-designed or retrofitted to eliminate the non-conforming element.

### **3-2.11 Linear Static Procedure.**

The LSP and limitations to its use are provided in the following sub-sections.

#### **3-2.11.1 Limitations on the Use of LSP.**

The use of the LSP is limited to structures that meet the following requirements for irregularities and Demand-Capacity Ratios (DCRs).

If there are no structural irregularities as defined in Section 3-2.11.1.1, a linear static procedure may be performed and it is not necessary to calculate the DCRs defined in Section 3-2.11.1.2. If the structure is irregular, a linear static procedure may be performed if all of the component DCRs determined in the Section 3-2.11.1.1 are less than or equal to 2.0. If the structure is irregular and one or more of the DCRs exceed 2.0, then a linear static procedure cannot be used.

### 3-2.11.1.1 Irregularity Limitations.

A structure is considered irregular if any one of the following is true:

1. Significant discontinuities exist in the gravity-load carrying and lateral force-resisting systems of a building, including out-of-plane offsets of primary vertical elements, roof “belt-girders”, and transfer girders (i.e., non-stacking primary columns or load-bearing elements). Stepped back stories are not considered an irregularity.
2. At any exterior column except at the corners, at each story in a framed structure, the ratios of bay stiffness and/or strength from one side of the column to the other are less than 50%. Three examples are; a) the lengths of adjacent bays vary significantly, b) the beams on either side of the column vary significantly in depth and/or strength, and c) connection strength and/or stiffness vary significantly on either side of the column (e.g., for a steel frame building, a shear tab connection on one side of a column and a fully rigid connection on the other side shall be considered irregular).
3. For all external load-bearing walls, except at the corners, and for each story in a load-bearing wall structure, the ratios of wall stiffness and/or strength from one side of an intersecting wall to the other are less than 50%.
4. The vertical lateral-load resisting elements are not parallel to the major orthogonal axes of the lateral force-resisting system, such as the case of skewed or curved moment frames and load-bearing walls.

### 3-2.11.1.2 DCR Limitation.

To calculate the DCRs for either framed or load-bearing structures, create a linear model of the building as described in Section 3-2.11.2.2. The model will have all primary components with the exception of the removed wall or column. The deformation-controlled load case in Section 3-2.11.4.1 shall be applied, with gravity dead and live loads increased by the load increase factor  $\Omega_{LD}$  in Section 3-2.11.5. The resulting actions (internal forces and moments) are defined as  $Q_{UDLim}$ :

Use  $Q_{UDLim}$  to calculate the **DCRs** for the deformation controlled actions as:

$$DCR = Q_{UDLim}/Q_{CE} \quad \text{Equation (3-9)}$$

where  $Q_{CE}$  = Expected strength of the component or element, as specified in Chapters 4 to 8.

### **3-2.11.2 Analytical Modeling.**

To model, analyze, and evaluate a building, employ a three-dimensional assembly of elements and components. Two-dimensional models are not permitted. Note that hand or spreadsheet calculations can be used, as allowed in Section 3-2.2 Alternative Rational Analysis.

#### **3-2.11.2.1 Loads.**

Analyze the model with two separate load cases: 1) to calculate the deformation-controlled actions  $Q_{UD}$ , and 2) to calculate the force-controlled actions  $Q_{UF}$ . Apply the Gravity loads to the model using the load cases for deformation-controlled actions and force-controlled actions defined in Section 3-2.11.4.

#### **3-2.11.2.2 Required Model Elements.**

Include the stiffness and resistance of only the primary elements and components. Insure that the model includes a sufficient amount of structural detail to allow the correct transfer of vertical loads from the floor and roof system to the primary elements. Use the guidance of ASCE 41 Chapters 9 through 12 to create the model. Also, as discussed later, and after the analysis is performed, check the primary and secondary elements against the acceptance criteria for force-controlled and deformation-controlled actions. While secondary elements are not included in the model, their actions and deformations can either be estimated based on the deformations of the model with only primary elements or the model may be re-analyzed with the secondary components included. If the model is re-analyzed with the secondary components included, their stiffness and resistance must be set to zero, i.e., the advantage of including the secondary components is that the analyst may more easily check the secondary elements deformations rather than perform hand calculations of the original model.

If the building contains sections that are less than three stories and are attached to the sections with three or more stories, the designer shall perform an analysis to determine whether there is a possibility that the presence of the short section will affect the taller section in a negative manner; if so, then include the short section in the model.

#### **3-2.11.2.3 Limitations on Connection Strength.**

For models that incorporate connections between horizontal flexural elements (beams, slabs, girders, etc) and vertical load-bearing elements (columns and walls), the strength of the connection shall not be modeled as greater than the strength of the attached horizontal flexural element.

### 3-2.11.3 Stability/P-Δ Effects.

Note that overall vertical and lateral stability as well as local stability (i.e., lateral torsional buckling) must be considered. However, a P-Δ analysis is not required for the Linear Static approach due to the small deformations. /2/

### 3-2.11.4 Loading.

Due to the different methods by which deformation-controlled and force-controlled actions are calculated, two load cases will be applied and analyzed: one for the deformation-controlled actions, and one for the force-controlled actions, as specified here.

Live load reduction is allowed, if the requirements in Section 3-2.3 are met.

#### 3-2.11.4.1 Load Case for Deformation-Controlled Actions $Q_{UD}$ .

To calculate the deformation-controlled actions, simultaneously apply the following combination of gravity loads:

Increased Gravity Loads for Floor Areas Above Removed Column or Wall.  
Apply the following increased gravity load combination to those bays immediately adjacent to the removed element and at all floors above the removed element; see Figures 3-13 and 3-14:

$$G_{LD} = \Omega_{LD} [1.2 D + (0.5 L \text{ or } 0.2 S)] \quad \text{Equation (3-10)}$$

where

- $G_{LD}$  = Increased gravity loads for deformation-controlled actions for Linear Static Analysis
- $D$  = Dead load including façade loads (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)
- $L$  = Live load including live load reduction per Section 3-2.3 (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)
- $S$  = Snow load (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)
- $\Omega_{LD}$  = Load increase factor for calculating deformation-controlled actions for Linear Static analysis; use appropriate value for framed or load-bearing wall structures; see Section 3-2.11.5

Gravity Loads for Floor Areas Away From Removed Column or Wall. Apply the following gravity load combination to those bays not loaded with  $G_{LD}$ ; see Figures 3-13 and 3-14:

$$G = 1.2 D + (0.5 L \text{ or } 0.2 S) \quad \text{Equation (3-11)}$$

where  $G$  = Gravity loads

/2/ /2/

#### 3-2.11.4.2 Load Case for Force-Controlled Actions $Q_{UF}$ .

To calculate the force-controlled actions, simultaneously apply the following combination of gravity loads.

##### Increased Gravity Loads for Floor Areas Above Removed Column or Wall.

Apply the following increased gravity load combination to those bays immediately adjacent to the removed element and at all floors above the removed element; see Figures 3-13 and 3-14:

$$\sqrt{2} \ G_{LF} = \Omega_{LF} [1.2 D + (0.5 L \text{ or } 0.2 S)] / 2 \quad \text{Equation (3-12)}$$

where

- $G_{LF}$  = Increased gravity loads for force-controlled actions for Linear Static analysis
- $D$  = Dead load including façade loads (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)
- $L$  = Live load including live load reduction per Section 3-2.3 (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)
- $S$  = Snow load (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)
- $\Omega_{LF}$  = Load increase factor for calculating force-controlled actions for Linear Static analysis; use appropriate value for framed or load-bearing wall structures; see Section 3-2.11.5

Gravity Loads for Floor Areas Away From Removed Column or Wall. Use Equation 3-11 to determine the load  $G$  and apply as shown in Figures 3-13 and 3-14.

#### 3-2.11.5 Load Increase Factor.

The load increase factors for deformation-controlled and force-controlled actions for column and wall removal are provided in Table 3-4.

In Table 3-4,  $m_{LIF}$  is the smallest  $m$  of any primary beam, girder, spandrel or wall element that is directly connected to the columns or walls directly above the column or wall removal location. For each primary beam, girder, spandrel or wall element,  $m$  is the  $m$ -factor defined in Chapters 4 to 8 of this UFC, where  $m$  is either explicitly provided in each chapter or reference is made to ASCE 41 and a corresponding performance level (Collapse Prevention or Life Safety). Columns are omitted from the determination of  $m_{LIF}$ . The method behind this procedure is explained in Appendix C.

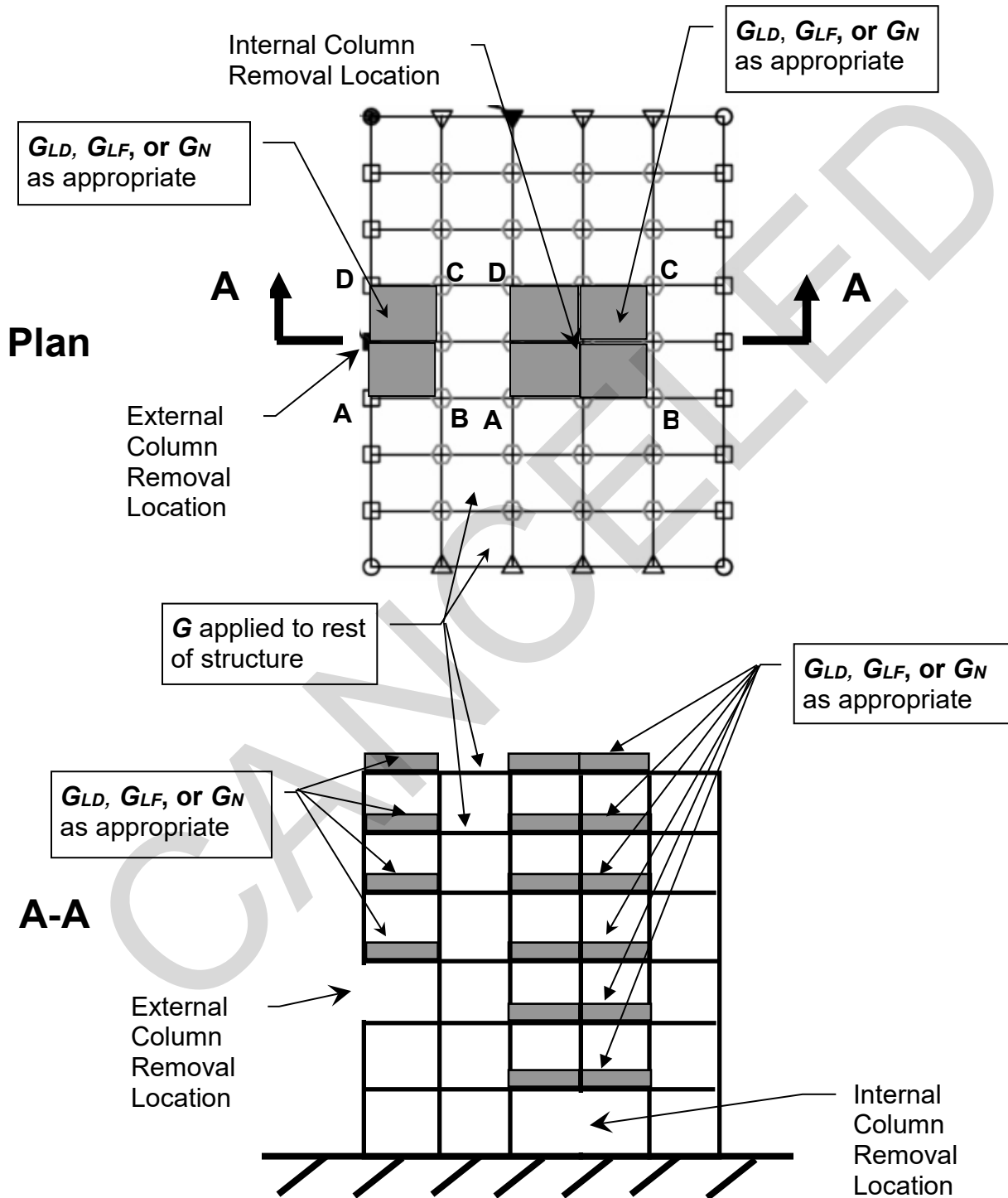
**Table 3-4. Load Increase Factors for Linear Static Analysis**

Material	Structure Type	$\Omega_{LD}$ , Deformation controlled	$\Omega_{LF}$ , Force controlled
Steel	Framed	$0.9 m_{LIF} + 1.1$	2.0
Reinforced Concrete	Framed <sup>A</sup>	$1.2 m_{LIF} + 0.80$	2.0
	Load-bearing Wall	$2.0 m_{LIF}$	2.0
Masonry	Load-bearing Wall	$2.0 m_{LIF}$	2.0
Wood	Load-bearing Wall	$2.0 m_{LIF}$	2.0
Cold-formed Steel	Load-bearing Wall	$2.0 m_{LIF}$	2.0

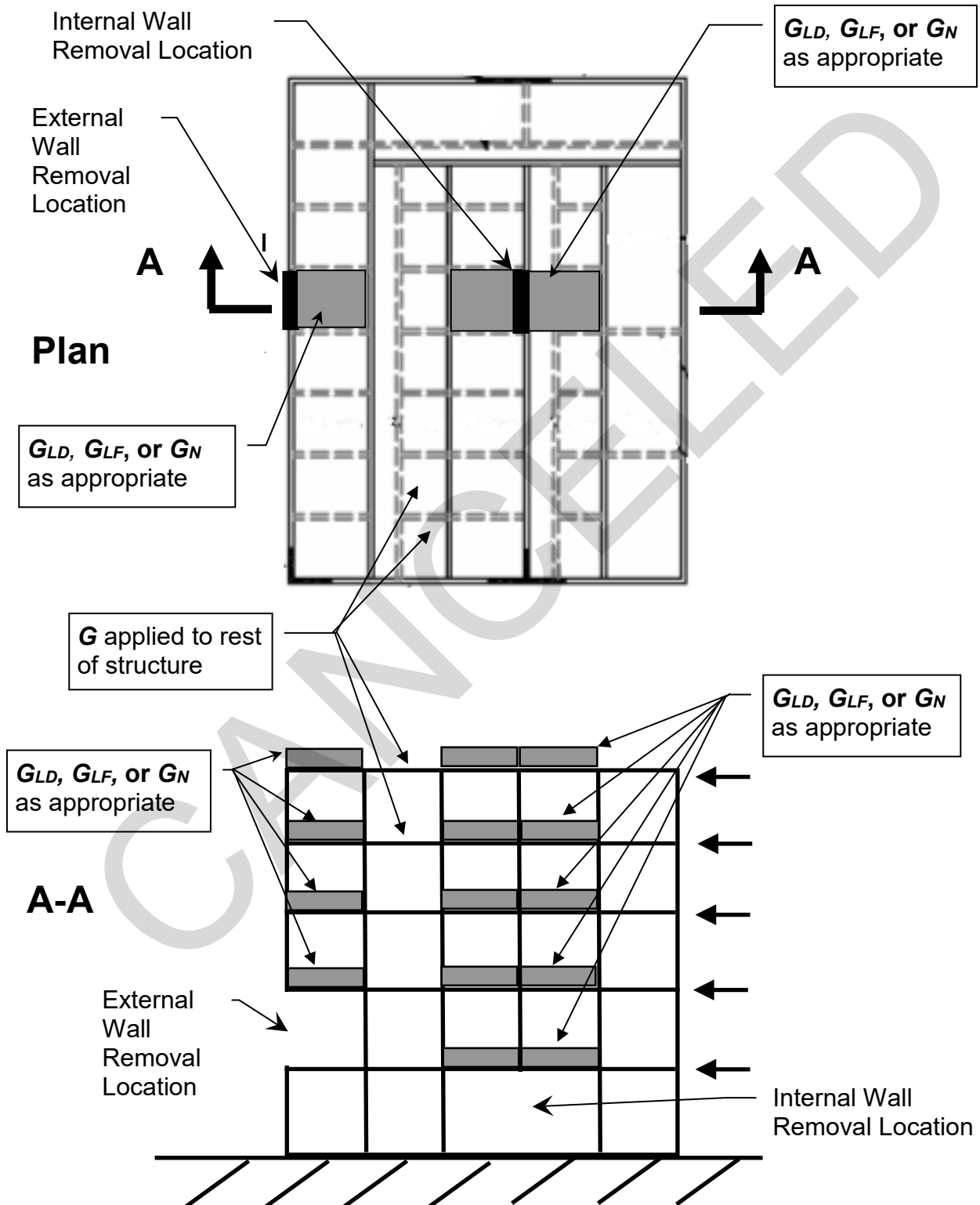
<sup>A</sup> Note that, per ASCE 41, reinforced concrete beam-column joints are treated as force-controlled; however, the hinges that form in the beam near the column are deformation-controlled and the appropriate m-factor from Chapter 4 of this UFC shall be applied to the calculation of the deformation-controlled load increase factor  $\Omega_{LD}$ .

\2\

**Figure 3-13. Loads and Load Locations for External and Internal Column Removal for Linear and Nonlinear Static Models (Left Side Demonstrates External Column Removal; Right Side Shows Internal Column Removal)**



**Figure 3-14. Loads and Load Locations for External and Internal Wall Removal for Linear and Nonlinear Static Models (Left Side Demonstrates External Wall Removal; Right Side Shows Internal Wall Removal)**





### 3-2.11.6 Design Forces and Deformations.

Calculate the deformation-controlled actions  $Q_{UD}$ , and force-controlled actions  $Q_{UF}$ , accordance with the linear analysis procedures of Sections 3-2.11.2 to 3-2.11.5.

### 3-2.11.7 Component and Element Acceptance Criteria.

Components and elements analyzed using the linear procedures of Section 3-2.11.2 to 3-2.11.5 shall satisfy the requirements of this section. Prior to selecting component acceptance criteria, classify components as primary or secondary, and classify actions as deformation-controlled or force-controlled, as defined in Section 3-2.5.

#### 3-2.11.7.1 Deformation Controlled Actions.

For deformation-controlled actions in all primary **and** secondary components, check that:

$$\phi m Q_{CE} \geq Q_{UD} \quad \text{Equation (3-13)}$$

where

- $Q_{UD}$  = Deformation-controlled action, from Linear Static model
- $m$  = Component or element demand modifier (m-factor) as defined in Chapters 4 to 8 of this document.
- $\phi$  = Strength reduction factor from the appropriate material specific code.
- $Q_{CE}$  = Expected strength of the component or element for deformation-controlled actions.

$Q_{CE}$ , the expected strength, shall be determined by considering all coexisting actions on the component under the design loading condition by procedures specified in ASCE 41 Chapters 9 through 12. Note that this includes interaction equations for shear, axial force, and moment and that these equations include force- and deformation-controlled actions, as well as expected and lower bound strengths.

Use the appropriate resistance factor for each action, as specified in the material specific design codes (i.e., the  $\phi$  factors in ACI 318, the AISC Steel Construction Manual, etc).

#### 3-2.11.7.2 Force-Controlled Actions.

For force-controlled actions in all primary **and** secondary components:

$$\phi Q_{CL} \geq Q_{UF} \quad \text{Equation (3-14)}$$

where

- $Q_{UF}$  = Force-controlled action, from Linear Static model
- $Q_{CL}$  = Lower-bound strength of a component or element

$\phi$  = Strength reduction factor from the appropriate material specific code.

$Q_{CL}$ , the lower-bound strength, shall be determined by considering all coexisting actions on the component under the design loading condition by procedures specified in ASCE 41 Chapters 9 through 12. Use the appropriate resistance factor for each action, as specified in the material specific design codes (i.e., the  $\phi$  factors in ACI 318, the AISC Steel Construction Manual, etc).

### 3-2.11.7.3 Secondary Elements and Components.

All secondary components and elements must be checked to ensure that they meet the acceptance criteria. Deformation-controlled actions are checked according to Equation 3-13 and force-controlled actions are checked according to Equation 3-14.

### 3-2.12 Nonlinear Static Procedure.

The NSP and limitations to its use are provided in the following sub-sections.

#### 3-2.12.1 Limitations on the Use of NSP.

There are no DCR or geometric irregularity limitations on the use of the NSP.

#### 3-2.12.2 Analytical Modeling.

To model, analyze, and evaluate a building, employ a three-dimensional assembly of elements and components. Two-dimensional models are not permitted. Create one model, as shown in Figures 3-13 and 3-14 for either framed or load-bearing wall structures, respectively. Inclusion of secondary components in the model is optional. However, if the secondary components are omitted, they must be checked after the analysis, against the allowable deformation-controlled criteria (e.g., to check the connections of gravity beams in a steel structure, compute the chord rotation and compare against the allowable plastic rotation angle for that connection). Include the stiffness and resistance of primary components. Note that the strength reduction factors are applied to the nonlinear strength models of the deformation controlled components (e.g., the nominal flexural strength of a beam or connection is multiplied by the appropriate  $\phi$  factor). Analyze the model for the Nonlinear Static load case defined in Section 3-2.12.4

Use the stiffness requirements of ASCE 41 Chapters 9 through 12 to create the model. Discretize the load-deformation response of each component along its length to identify locations of inelastic action. The force-displacement behavior of all components shall be explicitly modeled, including strength degradation and residual strength, if any. Model a connection explicitly if the connection is weaker or has less ductility than the connected components, or the flexibility of the connection results in a change in the connection forces or deformations greater than 10%.

If the building contains sections that are less than three stories and are attached to the sections with three or more stories, the designer shall use engineering judgment to include some or all of the shorter section if there is any possibility that the presence of the short section will affect the taller section in a negative manner.

### 3-2.12.3 Stability/P-Δ Effects.

Note that overall vertical and lateral stability as well as local stability (i.e., lateral torsional buckling) must be considered. /2/

### 3-2.12.4 Loading

Live load reduction is allowed, if the requirements in Section 3-2.3 are met. /2/

#### 3-2.12.4.1 Loads.

To calculate the deformation-controlled and force-controlled actions, simultaneously apply the following combination of gravity loads:

Increased Gravity Loads for Floor Areas Above Removed Column or Wall.  
Apply the following increased gravity load combination to those bays immediately adjacent to the removed element and at all floors above the removed element; see Figures 3-13 and 3-14:

$$G_N = \Omega_N [1.2 D + (0.5 L \text{ or } 0.2 S)] \quad \text{Equation (3-15)}$$

where  $G_N$  = Increased gravity loads for Nonlinear Static Analysis  
 $D$  = Dead load including façade loads (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)  
 $L$  = Live load including live load reduction per Section 3-2.3 (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)  
 $S$  = Snow load (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)  
 $\Omega_N$  = Dynamic increase factor for calculating deformation-controlled and force-controlled actions for Nonlinear Static analysis, use appropriate value for framed or load-bearing wall structures. see Section 3-2.12.5

Gravity Loads for Floor Areas Away From Removed Column or Wall. Apply the following gravity load combination to those bays not loaded with  $G_N$ ; see Figures 3-13 and 3-14.

$$G = 1.2 D + (0.5 L \text{ or } 0.2 S) \quad \text{Equation (3-16)}$$

where  $G$  = Gravity loads

/2/ /2/

#### 3-2.12.4.2 Loading Procedure.

Apply the loads using a load history that starts at zero and is increased to the final values. Apply at least 10 load steps to reach the total load. The software must be capable of incrementally increasing the load and iteratively reaching convergence before proceeding to the next load increment.

#### 3-2.12.5 Dynamic Increase Factor for NSP.

The Nonlinear Static dynamic increase factors are provided in Table 3-5.

In Table 3-5,  $\theta_{pra}$  is the plastic rotation angle given in the acceptance criteria tables in ASCE 41 and this UFC for the appropriate structural response level (Collapse Prevention or Life Safety, as specified in Chapters 4 to 8 of this UFC) for the particular element, component or connection;  $\theta_y$  is the yield rotation. For steel,  $\theta_y$  is given in Equation 9-1 in ASCE 41. For reinforced concrete,  $\theta_y$  is determined with the effective stiffness values provided in Table 10-5 in ASCE 41. Note that for connections,  $\theta_y$  is the yield rotation angle of the structural element that is being connected (beam, slab, etc) and  $\theta_{pra}$  is for the connection (determined from ASCE 41 and this UFC). Columns are omitted from the determination of the DIF.

To determine the DIF for the analysis of the entire structure, choose the smallest ratio of  $\theta_{pra}/\theta_y$  for any primary element, component, or connection in the model within or touching the area that is loaded with the increased gravity load, as shown in Figures 3-13 and 3-14. In other words, the DIF for every primary connection, beam, girder, wall element, etc that falls within or touches the perimeter marked as A-B-C-D must be determined and the largest value is used for the analysis. The method behind this procedure is explained in Appendix C.

**Table 3-5. Dynamic Increase Factors for Nonlinear Static Analysis**

Material	Structure Type	$\Omega_N$
Steel	Framed	$1.08 + 0.76/(\theta_{pra}/\theta_y + 0.83)$
Reinforced Concrete	Framed	$1.04 + 0.45/(\theta_{pra}/\theta_y + 0.48)$
	Load-Bearing Wall	2
Masonry	Load-bearing Wall	2
Wood	Load-bearing Wall	2
Cold-formed Steel	Load-bearing Wall	2

### 3-2.12.6 Design Forces and Deformations.

Calculate component design forces and deformations in accordance with the nonlinear analysis procedure of Sections 3-2.12.2 to 3-2.12.5.

### 3-2.12.7 Component and Element Acceptance Criteria.

Components and elements analyzed using the nonlinear procedures of Sections 3-2.12.2 to 3-2.12.5 shall satisfy the requirements of this section.

#### 3-2.12.7.1 Deformation-Controlled Actions.

Primary and secondary elements and components shall have expected deformation capacities greater than the maximum calculated deformation demands. Expected deformation capacities shall be determined considering all coexisting forces and deformations in accordance with Chapters 4 to 8 of this document.

#### 3-2.12.7.2 Force-Controlled Actions.

For force-controlled actions in all primary **and** secondary elements and components:

$$\phi Q_{CL} \geq Q_{UF} \quad \text{Equation (3-17)}$$

where  $Q_{UF}$  = Force-controlled action, from Nonlinear Static model  
 $Q_{CL}$  = Lower-bound strength of a component or element.  
 $\phi$  = Strength reduction factor from the appropriate material specific code.

$Q_{CL}$ , the lower-bound strength, shall be determined by considering all coexisting actions on the component under the design loading condition by procedures specified in ASCE 41 Chapters 9 through 12. Use the appropriate resistance factor for each action, as specified in the material specific design codes (i.e., the  $\phi$  factors in ACI 318, the AISC Steel Construction Manual, etc).

### 3-2.13 Nonlinear Dynamic Procedure.

The NDP and limitations to its use are provided in the following sub-sections.

#### 3-2.13.1 Limitations on the Use of NDP.

There are no DCR or geometric irregularity limitations on the use of the NDP.

### 3-2.13.2 Analytical Modeling.

To model, analyze, and evaluate a building, employ a three-dimensional assembly of elements and components. Two-dimensional models are not permitted. Create a model of the entire structure, including the wall section and column that are to be removed during the analysis. Include the stiffness and resistance of primary components. Note that the strength reduction factors are applied to the nonlinear strength models of the deformation controlled components (e.g., the nominal flexural strength of a beam or connection is multiplied by the appropriate  $\phi$  factor). Inclusion of secondary components in the model is optional. However, if the secondary components are omitted, they must be checked after the analysis, against the allowable deformation-controlled criteria (e.g., to check the connections of gravity beams in a steel structure, compute the chord rotation and compare against the allowable plastic rotation angle for that connection). Apply the loads per the loading procedure in Section 3-2.13.4.

Use the stiffness requirements of ASCE 41 Chapters 9 through 12 to create the model. Discretize the load-deformation response of each component along its length to identify locations of inelastic action. The force-displacement behavior of all components shall be explicitly modeled, including strength degradation and residual strength, if any. Model a connection explicitly if the connection is weaker or has less ductility than the connected components, or the flexibility of the connection results in a change in the connection forces or deformations greater than 10%.

If the building contains sections that are less than three stories and are attached to the sections with three or more stories, the designer shall use engineering judgment to include some or all of the shorter section if there is any possibility that the presence of the short section will affect the taller section in a negative manner.

### 3-2.13.3 Lateral Stability and P- $\Delta$ Effects.

\2\Note that overall vertical and lateral stability as well as local stability (i.e., lateral torsional buckling) must be considered./2/

### 3-2.13.4 Loading

\2\Live load reduction is allowed if the requirements in Section 3-2.3 are met.  
/2/

#### 3-2.13.4.1 Loads.

To calculate the deformation-controlled and force-controlled actions, apply the following gravity load per the loading procedure given in Section 3-2.13.4.2:

Gravity Loads for Entire Structure. Apply the following gravity load combination to the entire structure.

$$G_{ND} = 1.2 D + (0.5 L \text{ or } 0.2 S) \quad \text{Equation (3-18)}$$

where  $G_{ND}$  = Gravity loads for Nonlinear Dynamic Analysis  
 $D$  = Dead load including façade loads (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)  
 $L$  = Live load including live load reduction per Section 3-2.3 (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)  
 $S$  = Snow load (lb/ft<sup>2</sup> or kN/m<sup>2</sup>)

2/2/

#### 3-2.13.4.2 Loading Procedure.

Starting at zero load, monotonically and proportionately increase the gravity loads to the entire model (i.e., the column or wall section have not been removed yet) until equilibrium is reached.

After equilibrium is reached for the framed and load-bearing wall structures, remove the column or wall section. While it is preferable to remove the column or wall section instantaneously, the duration for removal must be less than one tenth of the period associated with the structural response mode for the vertical motion of the bays above the removed column, as determined from the analytical model with the column or wall section removed. The analysis shall continue until the maximum displacement is reached or one cycle of vertical motion occurs at the column or wall section removal location.

#### 3-2.13.5 Design Forces and Deformations.

Calculate component design forces and deformations in accordance with the nonlinear analysis procedure of Sections 3-2.13.2 to 3-2.13.4.

#### 3-2.13.6 Component and Element Acceptance Criteria.

Components and elements analyzed using the nonlinear procedures of Sections 3-2.13.2 to 3-2.13.4 shall satisfy the requirements of this section.

##### 3-2.13.6.1 Deformation-Controlled Actions.

Primary and secondary elements and components shall have expected deformation capacities greater than the maximum calculated deformation demands. Expected deformation capacities shall be determined considering all coexisting forces and deformations in accordance with Chapters 4 to 8 of this document.

### 3-2.13.6.2 Force-Controlled Actions.

For force-controlled actions in all primary **and** secondary components,

$$\phi Q_{CL} \geq Q_{UF} \quad \text{Equation (3-19)}$$

where  $Q_{UF}$  = Force-controlled action, from Nonlinear Dynamic model  
 $Q_{CL}$  = Lower-bound strength of a component or element.  
 $\phi$  = Strength reduction factor from the appropriate material specific code.

$Q_{CL}$ , the lower-bound strength, shall be determined by considering all coexisting actions on the component under the design loading condition by procedures specified in ASCE 41 Chapters 9 through 12. Use the appropriate resistance factor for each action, as specified in the material specific design codes (i.e., the  $\phi$  factors in ACI 318, the AISC Steel Construction Manual, etc).

\2\

## 3-3 ENHANCED LOCAL RESISTANCE

Enhanced Local Resistance (ELR) is required in three cases: RC II Option 1 (Tie Forces and ELR), RC III (Alternate Path and ELR), and RC IV (Tie Forces, Alternate Path and ELR). All three cases contain the same objective, which is to insure that a ductile failure mechanism can form when the column or wall is loaded laterally to failure. To meet this objective, the column or wall must not fail in shear prior to the development of the maximum flexural strength.

Two components must meet the ELR requirement: 1. The column or wall, and 2. The connections between the end of the column or wall and the lateral supports (floor slab, base plate, etc).

Note that design for ELR is not required if the wall or column has been designed for a specific design basis threat, providing that the design basis threat was developed with a risk assessment approach that was approved by the building owner, government agency or other responsible entity.

### 3-3.1 Load and Resistance Factor Design for Enhanced Local Resistance.

The LRFD approach is used for ELR design,

$$\phi R_n \geq R_u \quad \text{Equation (3-20)}$$

where  $\phi R_n$  = Design strength  
 $\phi$  = Strength reduction factor



$R_n$  = Nominal strength, including over-strength factors  
 $R_u$  = Required strength

The design strength provided by a component is taken as the product of the strength reduction factor  $\phi$  and the nominal strength  $R_n$ . For ELR, all strength reduction factors  $\phi$  shall be 1.0. In addition, the material strengths for shear, flexure and all other actions shall be the expected material strength (i.e., with the appropriate over-strength factor applied to the lower bound material strength).

### 3-3.1.1 Flexural Demand.

The flexural demand is the nominal flexural strength ( $R_n$ ) of a column or wall under the conditions specified in Sections 3-3.3, 3-3.4, and 3-3.5.

### 3-3.1.2 Shear Demand.

The shear demand is the required shear strength ( $R_u$ ) of the column or wall that is necessary to achieve the required flexural demand of the component, i.e. at the formation of a three-hinge mechanism or similar failure mode in the component. In calculating the shear demand, consider any effects (axial load, end conditions, etc) that may act to increase the nominal flexural strength; in no case shall the shear demand be less than that of the column or wall with zero axial load acting. Include any applicable material over-strength factors. The shear demand shall be determined for the horizontal out-of-plane direction (i.e., perpendicular to the building perimeter façade). Columns at building corners or re-entrant corners shall be evaluated in both directions normal to the building perimeter façade. Examples of this procedure are provided in Appendices D and E.

## 3-3.2 ELR Location and Extent Requirements.

### 3-3.2.1 RC II Option 1.

For RC II Option 1 and framed and two-way load-bearing wall buildings, ELR is applied to the perimeter corner and penultimate columns and load-bearing walls of the first story above grade. For one-way load-bearing wall buildings, ELR shall be applied to the entire length of the end wall and penultimate wall; it is recognized that the majority of the penultimate wall may be interior to the structure, but it must still be designed for ELR.

### 3-3.2.2 RC III.

For RC III and framed and two-way load-bearing wall buildings, ELR is applied to all perimeter columns and load-bearing walls of the first story above grade. For one-way load-bearing wall buildings, ELR shall be applied to the entire length of the end wall and penultimate wall; it is recognized that the majority of the penultimate wall may be interior to the structure, but it must still be designed for ELR.

### 3-3.2.3 RC IV.

For RC IV and framed and two-way load-bearing wall buildings, ELR is applied to all perimeter columns and load-bearing walls of the first story above grade. For one-way load-bearing wall buildings, ELR shall be applied to the entire length of the end wall and penultimate wall; it is recognized that the majority of the penultimate wall may be interior to the structure, but it must still be designed for ELR.

### 3-3.3 RC II Option 1 Shear and Flexural Requirements.

For RC II Option 1, the flexural demand for the columns and walls is the nominal flexural strength ( $R_n$ ) of the existing or as-designed structure and, therefore, no design modification is needed to meet the flexural requirement.

The shear demand is based on the flexural demand of the component as described above. Apply the procedure in Section 3-3.1.2 to the columns and walls to determine the shear demand. If Equation 3-20 is not satisfied, modify the column or wall design such that adequate design shear strength ( $\phi R_n$ ) is achieved. If the flexural demand is increased as a result of meeting the shear requirement, check that the design shear strength still exceeds the shear demand associated with this new flexural demand.

### 3-3.4 RC III Shear and Flexural Requirements.

4) An RC III building /4/ must first meet the Alternate Path requirement. The flexural demand for the columns and walls is the nominal flexural strength ( $R_n$ ) of the design after the Alternate Path requirement is applied and therefore, no design modification is needed to meet the flexural requirement.

The shear demand is based on the flexural demand of the component as described above. Apply the procedure in Section 3-3.1.2 to the columns and walls to determine the shear demand. If Equation 3-20 is not satisfied, modify the column or wall design such that adequate design shear strength ( $\phi R_n$ ) is achieved. If the flexural demand is increased as a result of meeting the shear requirement, check that the design shear strength still exceeds the shear demand associated with this new flexural demand.

### 3-3.5 RC IV Shear and Flexural Requirements.

#### 3-3.5.1 RC IV Flexural Requirements.

For RC IV, two nominal flexural strengths must first be determined and used to define the flexural demand. First, the **baseline** nominal flexural strength is determined using the design of the structure when only gravity loads are considered and after the AP procedure is applied. If such a design or analysis was not performed during the initial design and analysis of the new or existing structure, it must be performed to determine the column and load-bearing wall design.

Second, the **existing** nominal flexural strength is determined using the column and load-bearing wall design defined after the Alternate Path procedure was applied to the structural design that incorporated all applied loads (wind, earthquake, gravity, etc).

For columns in RC IV structures, the flexural demand is equal to the larger of: 1) the baseline nominal flexural strength multiplied by 2.0 and 2) the  $\phi M_n$  nominal flexural strength. If the flexural demand is controlled by condition 1), then redesign the column as required to meet the flexural demand of this controlling condition.

For load-bearing walls in RC IV structures, the flexural demand is equal to the larger of 1) the baseline nominal flexural strength multiplied by 1.5, and 2) the  $\phi M_n$  nominal flexural strength. If the flexural demand is controlled by condition 1), then redesign the wall as required to meet the flexural demand of this controlling condition.

### 3-3.5.2 RC IV Shear Requirements.

The shear demand for columns and walls is based on the flexural demand as calculated in the above section. Apply the procedure in Section 3-3.1.2 to the columns and walls to determine the shear demand. If Equation 3-20 is not satisfied, modify the column or wall design such that adequate design shear strength ( $\phi V_n$ ) is achieved. If the flexural demand is increased as a result of meeting the shear requirement, check that the design shear strength still exceeds the shear demand associated with this new flexural demand.

### 3-3.6 Connection Design for Rebound Reaction Forces.

In some dynamic loading scenarios, the column or wall will initially be pushed into the building and then rebound back; the connections must be designed to prevent failure during both phases of loading. Connections at the top and bottom of the columns and walls shall be designed for a rebound reaction force equal to 50% of the inbound value.

### 3-3.7 Conflicts Between ELR and Other Design Requirements.

The ELR requirements may be satisfied in a number of ways and as the designer desires, providing the ELR-modified design does not violate the standard building and material-specific design codes. If there is a conflict with the ELR modifications, the building or material-specific design code has precedence and the ELR design must be altered. /2/

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## CHAPTER 4 REINFORCED CONCRETE

This chapter provides the specific requirements for designing a reinforced concrete building to resist progressive collapse. Appendix D demonstrates the application of the reinforced concrete design requirements for a 7-story building.

If composite construction with other materials is employed, use the design guidance from the appropriate material chapter in this UFC for those structural elements or portions of the structure.

Note that the combination of design requirements (TF, AP, and ELR) will depend upon the Risk Category of the buildings, as defined in Section 2-2.

### 4-1 MATERIAL PROPERTIES FOR REINFORCED CONCRETE.

Apply the appropriate over-strength factors to the calculation of the design strengths for both Tie Forces and the Alternate Path method. The over-strength factors are provided in ASCE 41 in Table 10-4 Factors to Translate Lower-Bound Material Properties to Expected Strength Material Properties.

### 4-2 STRENGTH REDUCTION FACTOR $\phi$ FOR REINFORCED CONCRETE.

For the Alternate Path and Tie Force methods, use the appropriate strength reduction factor specified in ACI 318 *Building Code Requirements for Structural Concrete* for the component and behavior under consideration.

### 4-3 TIE FORCE REQUIREMENTS FOR REINFORCED CONCRETE.

Apply the Tie Force requirements in Section 3-1, when applicable, for concrete frame and load-bearing wall structural systems, mixed systems with concrete elements, and framed and load-bearing wall systems with precast concrete floors

The strength reduction factor  $\phi$  for properly anchored, embedded, or spliced steel reinforcement in tension shall be taken as 0.75 (based on ACI 318 for strut and tie models).

The structural integrity requirements of ACI 318 for cast-in-place and precast concrete construction must be satisfied, as well as the Tie Force requirements in this document. Use the largest or most stringent requirement when there is overlap between ACI 318 and this UFC.

#### **4-4 ALTERNATE PATH REQUIREMENTS FOR REINFORCED CONCRETE.**

##### **4-4.1 General.**

Use the Alternate Path method in Section 3-2 to verify that the structure can bridge over removed elements.

##### **4-4.2 Flexural Members and Joints.**

The design strength and rotational capacities of the beams and beam-to-column joints shall be determined with the guidance found in ASCE 41, as modified with the acceptance criteria provided in Paragraph 4-4.3.

##### **4-4.3 Modeling and Acceptance Criteria for Reinforced Concrete.**

With the exception of Tables 10-7, 10-11, 10-14, and 10-15 in ASCE 41, use the modeling parameters, nonlinear acceptance criteria and linear m-factors for the Life Safety condition from Chapter 10 of ASCE 41 for primary and secondary components. Use the ASCE 41 modeling parameters and guidance, including definitions of stiffness, to create the analytical model.

Replace Table 10-7 of ASCE 41 with Table 4-1, which contains the nonlinear modeling parameters and acceptance criteria for reinforced concrete beams. Replace Table 10-11 of ASCE 41 with Table 4-2, which contains the acceptance criteria for linear modeling of reinforced concrete beams.

Replace Table 10-14 of ASCE 41 with Table 4-3, which contains the nonlinear modeling parameters and acceptance criteria for two-way slabs and slab-column connections. Replace Table 10-15 of ASCE 41 with Table 4-4, which contains the acceptance criteria for linear modeling of two-way slabs and slab-column connections.

#### **4-5 ENHANCED LOCAL RESISTANCE REQUIREMENTS FOR REINFORCED CONCRETE.**

Apply the Enhanced Local Resistance requirements in Section 3-3, where applicable, for framed and load-bearing wall reinforced concrete buildings.

**Table 4-1. Nonlinear Modeling Parameters and Acceptance Criteria for Reinforced Concrete Beams (Replacement for Table 10-7 in ASCE 41)**

Conditions			Modeling Parameters <sup>1</sup>			Acceptance Criteria <sup>1,2</sup>		
			Plastic Rotations Angle, radians		Residual Strength Ratio	Plastic Rotations Angle, radians		
							Component Type	
			Primary		Secondary			
a			b		c			

i. Beams controlled by flexure <sup>3</sup>								
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf. <sup>4</sup>	$\frac{V}{b_w d \sqrt{f'_c}}$						
≤ 0.0	C	≤ 3	0.063	0.10	0.2		0.063	0.10
≤ 0.0	C	≥ 6	0.05	0.08	0.2		0.05	0.08
≥ 0.5	C	≤ 3	0.05	0.06	0.2		0.05	0.06
≥ 0.5	C	≥ 6	0.038	0.04	0.2		0.038	0.04
≤ 0.0	NC	≤ 3	0.05	0.06	0.2		0.05	0.06
≤ 0.0	NC	≥ 6	0.025	0.03	0.2		0.025	0.03
≥ 0.5	NC	≤ 3	0.025	0.03	0.2		0.025	0.03
≥ 0.5	NC	≥ 6	0.013	0.02	0.2		0.013	0.02

ii. Beams controlled by shear <sup>3</sup>								
Stirrup spacing ≤ d / 2			0.0030	0.02	0.2		0.002	0.01
Stirrup spacing > d / 2			0.0030	0.01	0.2		0.002	0.005

iii. Beams controlled by inadequate development or splicing along the span <sup>3</sup>								
Stirrup spacing ≤ d / 2			0.0030	0.02	0.0		0.002	0.01
Stirrup spacing > d / 2			0.0030	0.01	0.0		0.002	0.005

iv. Beams controlled by inadequate embedment into beam-column joint <sup>3</sup>								
			0.015	0.03	0.2		0.01	0.02

1. Linear interpolation between values listed in the table shall be permitted. See Section 3-2.4 for definition of primary and secondary components and Figure 3-7 for definition of nonlinear modeling parameters *a*, *b*, and *c*.
2. Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength, in accordance with Section 7.4.3.2 of ASCE 41.
3. Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.
4. "C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ *d*/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (*V<sub>s</sub>*) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.

**Table 4-2. Acceptance Criteria for Linear Models of Reinforced Concrete Beams  
(Replacement for Table 10-11 in ASCE 41)**

Conditions			<i>m</i> -factors <sup>1</sup>	
			Component Type	
			Primary	Secondary
<b>i. Beams controlled by flexure<sup>2</sup></b>				
$\frac{\rho - \rho'}{\rho_{bal}}$	Trans. Reinf. <sup>3</sup>	$\frac{V}{b_w d \sqrt{f'_c}}$ <sup>4</sup>		
≤ 0.0	C	≤ 3	16	19
≤ 0.0	C	≥ 6	9	9
≥ 0.5	C	≤ 3	9	9
≥ 0.5	C	≥ 6	6	7
≤ 0.0	NC	≤ 3	9	9
≤ 0.0	NC	≥ 6	6	7
≥ 0.5	NC	≤ 3	6	7
≥ 0.5	NC	≥ 6	4	5
<b>ii. Beams controlled by shear<sup>2</sup></b>				
Stirrup spacing ≤ <i>d</i> / 2			1.5	3
Stirrup spacing > <i>d</i> / 2			1.5	2
<b>iii. Beams controlled by inadequate development or splicing along the span<sup>2</sup></b>				
Stirrup spacing ≤ <i>d</i> / 2			1.5	3
Stirrup spacing > <i>d</i> / 2			1.5	2
<b>iv. Beams controlled by inadequate embedment into beam-column joint<sup>2</sup></b>				
			2	3
<ol style="list-style-type: none"> <li>Linear interpolation between values listed in the table shall be permitted. See Section 3-2.4 for definition of primary and secondary components.</li> <li>Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.</li> <li>"C" and "NC" are abbreviations for conforming and nonconforming transverse reinforcement. A component is conforming if, within the flexural plastic hinge region, hoops are spaced at ≤ <i>d</i>/3, and if, for components of moderate and high ductility demand, the strength provided by the hoops (<i>V<sub>s</sub></i>) is at least three-fourths of the design shear. Otherwise, the component is considered nonconforming.</li> <li><i>V</i> is the design shear force calculated using limit-state analysis procedures in accordance with Section 10.4.2.4.1 of ASCE 41.</li> </ol>				



**Table 4-3. Modeling Parameters and Acceptance Criteria for Nonlinear Models of Two-Way Slabs and Slab-Column Connections (Replacement for Table 6-14 in ASCE 41)**

Conditions	Modeling Parameters <sup>1</sup>			Acceptance Criteria <sup>1,2</sup>		
	Plastic Rotations Angle, radians		Residual Strength Ratio	Plastic Rotations Angle, radians		
				Component Type		
				Primary	Secondary	
	a	b	c			
<b>i. Slabs controlled by flexure, and slab-column connections<sup>3</sup></b>						
$\frac{V_g}{V_o}$ <sup>2</sup>	Continuity Reinforcement <sup>3</sup>					
≤ 0.2	Yes	0.05	0.10	0.2	0.05	0.10
≥ 0.4	Yes	0.0	0.04	0.2	0.0	0.08
≤ 0.2	No	0.02	0.02	-	0.015	0.015
≥ 0.4	No	0.0	0.0	-	0.0	0.0
<b>ii. Slabs controlled by inadequate development or splicing along the span<sup>3</sup></b>						
		0.0	0.02	0.0	0.0	0.01
<b>iii. Slabs controlled by inadequate embedment into slab-column joint<sup>3</sup></b>						
		0.015	0.03	0.2	0.01	0.02
<ol style="list-style-type: none"> <li>Linear interpolation between values listed in the table shall be permitted. See Section 3-2.4 for definition of primary and secondary components and Figure 3-7 for definition of nonlinear modeling parameters <i>a</i>, <i>b</i>, and <i>c</i>.</li> <li>Primary and secondary component demands shall be within secondary component acceptance criteria where the full backbone curve is explicitly modeled including strength degradation and residual strength, in accordance with Section 7.4.3.2 of ASCE 41.</li> <li>Where more than one of the conditions i, ii, iii, and iv occurs for a given component, use the minimum appropriate numerical value from the table.</li> <li><math>V_g</math> = the gravity shear acting on the slab critical section as defined by ACI 318; <math>V_o</math> = the direct punching shear strength as defined by ACI 318.</li> <li>Under the heading “Continuity Reinforcement,” use “Yes” where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, use “Yes” where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, use “No.”</li> </ol>						

**Table 4-4. Acceptance Criteria for Linear Models of Two-Way Slabs and Slab-Column Connections (Replacement for Table 6-15 in ASCE 41)**

		<i>m</i> -factors <sup>1</sup>		
Conditions			Component Type	
			Primary	Secondary
<b>i. Slabs controlled by flexure, and slab-column connections<sup>2</sup></b>				
$\frac{V_g}{V_o}$ <sup>3</sup>	Continuity Reinforcement <sup>4</sup>			
≤ 0.2	Yes		6	7
≥ 0.4	Yes		1	5
≤ 0.2	No		2	2
≥ 0.4	No		1	1
<b>ii. Slabs controlled by inadequate development or splicing along the span<sup>2</sup></b>				
			-	4
<b>iii. Slabs controlled by inadequate embedment into slab-column joint<sup>2</sup></b>				
			3	4

1. Linear interpolation between values listed in the table shall be permitted. See Section 3-2.4 for definition of primary and secondary components.
2. Where more than one of the conditions i, ii, and iii occurs for a given component, use the minimum appropriate numerical value from the table.
3.  $V_g$  = the gravity shear acting on the slab critical section as defined by ACI 318;  $V_o$  = the direct punching shear strength as defined by ACI 318.
4. Under the heading "Continuity Reinforcement," use "Yes" where at least one of the main bottom bars in each direction is effectively continuous through the column cage. Where the slab is post-tensioned, use "Yes" where at least one of the post-tensioning tendons in each direction passes through the column cage. Otherwise, use "No."

## CHAPTER 5 STRUCTURAL STEEL

This chapter provides the specific requirements for designing a structural steel building to resist progressive collapse. Appendix E demonstrates the application of the structural steel design requirements for a 4-story building.

If composite construction with other materials is employed, use the design guidance from the appropriate material chapter in this UFC for those structural elements or portions of the structure.

Note that the combination of design requirements (TF, AP, and ELR) will depend upon the Risk Category of the buildings, as defined in Section 2-2.

### 5-1 MATERIAL PROPERTIES FOR STRUCTURAL STEEL.

Apply the appropriate over-strength factors to the calculation of the design strengths for both Tie Forces and the Alternate Path method. The over-strength factors are provided in ASCE 41 in Table 5-3 Factors to Translate Lower-Bound Steel Properties to Expected Strength Steel Properties.

### 5-2 STRENGTH REDUCTION FACTOR $\phi$ FOR STRUCTURAL STEEL.

For Alternate Path and Tie Force methods, use the appropriate strength reduction factor  $\phi$  specified in ANSI/AISC 360 *Specifications for Structural Steel Buildings* for the component and behavior under consideration. If steel components can be proven capable of carrying the required longitudinal, transverse, and peripheral tie strength while undergoing rotations of 0.20-rad (11.3-deg), use the appropriate strength reduction factor  $\phi$  for each limit state considered.

### 5-3 TIE FORCE REQUIREMENTS FOR STEEL.

Apply the Tie Force requirements in Section 3-1, where applicable, for framed steel buildings.

### 5-4 ALTERNATE PATH METHOD FOR STEEL.

#### 5-4.1 General.

Use the Alternate Path method in Section 3-2, where applicable, to verify that the structure can bridge over removed elements.

#### 5-4.2 Connection Rotational Capacity.

The design strength and rotational capacities of the beams and beam-to-column connections shall be determined with the guidance found in ASCE 41, as modified with the acceptance criteria provided in Paragraph 5-4.3.

### 5-4.3 Modeling and Acceptance Criteria for Structural Steel.

With the exception of the connections and elements discussed later in this section, use the modeling parameters, nonlinear acceptance criteria and linear  $m$ -factors for the Life Safety condition from Chapter 9 of ASCE 41 for primary and secondary components. Use the modeling parameters and guidance, including definitions of stiffness, to create the analytical model.

Columns under high axial load ( $P/P_{CL} > 0.5$ ) shall be considered force-controlled, with the considered loads ( $P$  and  $M$ ) equal to the maximum loads from the analysis. The  $P$ - $M$  interaction equation shall not exceed unity. For  $P/P_{CL} \leq 0.5$ , the interaction equation shall be used with the moment considered as deformation-controlled and the axial force as force-controlled.

Nonlinear and linear acceptance criteria for structural steel components shall meet the Life Safety condition for primary and secondary elements provided in Tables 9-5, 9-6 and 9-7 of ASCE 41, except as follows:

1. For beams subjected to flexure or flexure plus axial tension, use the Collapse Prevention values for primary and secondary elements.
2. For the Fully Restrained (FR) and Partially Restrained (PR) connections listed in Tables 5-1 and 5-2 in this UFC, use the specified plastic rotations, modeling parameters and  $m$ -factors, as given.

For the Double Angles PR connection, the expected flexural strength shall be determined for each of the three limit states listed in Tables 5-1 and 5-2, using accepted analytical procedures. For the Simple Shear Tab, the expected flexural strength will be taken as the smallest flexural strength determined with limit state analysis for bolt shear, weld failure, block shear, bearing, plate flexure or other limit states as appropriate.

### 5-5 ENHANCED LOCAL RESISTANCE REQUIREMENTS FOR STEEL.

Apply the Enhanced Local Resistance requirements in Section 3-3, where applicable, for framed and load-bearing wall steel buildings.

**Table 5-1. Acceptance Criteria for Linear Static Modeling of Steel Frame Connections**

Connection Type	Linear Acceptance Criteria	
	<i>m</i> -factors	
	Primary <sup>(1)</sup>	Secondary <sup>(1)</sup>
<b>Fully Restrained Moment Connections</b>		
Improved WUF with Bolted Web	$2.3 - 0.021d$	$4.9 - 0.048d$
Reduced Beam Section (RBS)	$4.9 - 0.025d$	$6.5 - 0.025d$
WUF	$4.3 - 0.083d$	$4.3 - 0.048d$
SidePlate®	$6.7 - 0.039d^{(2)}$	$11.1 - 0.062d$
<b>Partially Restrained Moment Connections (Relatively Stiff)</b>		
Double Split Tee		
a. Shear in Bolt	4	6
b. Tension in Bolt	1.5	4
c. Tension in Tee	1.5	4
d. Flexure in Tee	5	7
<b>Partially Restrained Simple Connections (Flexible)</b>		
Double Angles		
a. Shear in Bolt	$5.8 - 0.107d_{bg}^{(3)}$	$8.7 - 0.161d_{bg}$
b. Tension in Bolt	1.5	4
c. Flexure in Angles	$8.9 - 0.193d_{bg}$	$13.0 - 0.290d_{bg}$
Simple Shear Tab	$5.8 - 0.107d_{bg}$	$8.7 - 0.161d_{bg}$

<sup>(1)</sup> Refer to Section 3-2.4 for determination of Primary and Secondary classification

<sup>(2)</sup>  $d$  = depth of beam, inch

<sup>(3)</sup>  $d_{bg}$  = depth of bolt group, inch

**Table 5-2. Modeling Parameters and Acceptance Criteria for Nonlinear Modeling of Steel Frame Connections**

Connection Type	Nonlinear Modeling Parameters <sup>(1)</sup>			Nonlinear Acceptance Criteria	
	Plastic Rotation Angle, radians		Residual Strength Ratio	Plastic Rotation Angle, radians	
	<i>a</i>	<i>b</i>	<i>c</i>	Primary <sup>(2)</sup>	Secondary <sup>(2)</sup>
<b>Fully Restrained Moment Connections</b>					
Improved WUF with Bolted Web	0.021 - 0.0003d	0.050 - 0.0006d	0.2	0.021 - 0.0003d	0.050 - 0.0006d
Reduced Beam Section (RBS)	0.050 - 0.0003d	0.070 - 0.0003d	0.2	0.050 - 0.0003d	0.070 - 0.0003d
WUF	0.0284 - 0.0004d	0.043 - 0.0006d	0.2	0.0284 - 0.0004d	0.043 - 0.0006d
SidePlate®	0.089 - 0.0005d <sup>(3)</sup>	0.169 - 0.0001d	0.6	0.089 - 0.0005d	0.169 - 0.0001d
<b>Partially Restrained Moment Connections (Relatively Stiff)</b>					
Double Split Tee					
a. Shear in Bolt	0.036	0.048	0.2	0.03	0.040
b. Tension in Bolt	0.016	0.024	0.8	0.013	0.020
c. Tension in Tee	0.012	0.018	0.8	0.010	0.015
d. Flexure in Tee	0.042	0.084	0.2	0.035	0.070
<b>Partially Restrained Simple Connections (Flexible)</b>					
Double Angles					
a. Shear in Bolt	0.0502 - 0.0015d <sub>bg</sub> <sup>(4)</sup>	0.072 - 0.0022d <sub>bg</sub>	0.2	0.0502 - 0.0015d <sub>bg</sub>	0.0503 - 0.0011d <sub>bg</sub>
b. Tension in Bolt	0.0502 - 0.0015d <sub>bg</sub>	0.072 - 0.0022d <sub>bg</sub>	0.2	0.0502 - 0.0015d <sub>bg</sub>	0.0503 - 0.0011d <sub>bg</sub>
c. Flexure in Angles	0.1125 - 0.0027d <sub>bg</sub>	0.150 - 0.0036d <sub>bg</sub>	0.4	0.1125 - 0.0027d <sub>bg</sub>	0.150 - 0.0036d <sub>bg</sub>
Simple Shear Tab	0.0502 - 0.0015d <sub>bg</sub>	0.1125 - 0.0027d <sub>bg</sub>	0.2	0.0502 - 0.0015d <sub>bg</sub>	0.1125 - 0.0027d <sub>bg</sub>

- (1) Refer to Figure 3-7 for definition of nonlinear modeling parameters *a*, *b*, and *c*  
(2) Refer to Section 3-2.4 for determination of Primary and Secondary classification  
(3) *d* = depth of beam, inch  
(4) *d*<sub>bg</sub> = depth of bolt group, inch

## CHAPTER 6 MASONRY

This chapter provides the specific requirements for designing a masonry building to resist progressive collapse.

If composite construction with other materials is employed, use the design guidance from the appropriate material chapter in this UFC for those structural elements or portions of the structure.

Note that the combination of design requirements (TF, AP, and ELR) will depend upon the Risk Category of the buildings, as defined in Section 2-2.

### 6-1 MATERIAL PROPERTIES FOR MASONRY.

Apply the appropriate over-strength factors to the calculation of the design strengths for both Tie Forces and the Alternate Path method. The over-strength factors are provided in ASCE 41 in Table 11-1 Factors to Translate Lower-Bound Masonry Properties to Expected Strength Masonry Properties.

### 6-2 STRENGTH REDUCTION FACTOR $\Phi$ FOR MASONRY.

For Tie Force and Alternate Path methods, use the appropriate strength reduction factor specified in ACI 530 *Building Code Requirements for Masonry Structures* for the component and behavior under consideration. If masonry components can be proven capable of carrying the required longitudinal, transverse, and peripheral tie strength while undergoing rotations of 0.20-rad (11.3-deg), use the appropriate strength reduction factor  $\Phi$  for each limit state considered.

### 6-3 TIE FORCE REQUIREMENTS FOR MASONRY.

Apply the Tie Force requirements in Section 3-1, where applicable, for load-bearing masonry buildings.

### 6-4 ALTERNATE PATH METHOD FOR MASONRY.

#### 6-4.1 General.

Use the Alternate Path method in Section 3-2, where applicable, to verify that the structure can bridge over removed elements.

#### 6-4.2 Modeling and Acceptance Criteria for Masonry.

Use the modeling parameters, nonlinear acceptance criteria and linear m-factors for the Life Safety condition from Chapter 11 of ASCE 41 for primary and secondary components. Use the modeling parameters and guidance, including definitions of stiffness, to create the analytical model.

**6-5      ENHANCED LOCAL RESISTANCE REQUIREMENTS FOR MASONRY.**

Apply the Enhanced Local Resistance requirements in Section 3-3, where applicable, for framed and load-bearing wall masonry buildings.

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## CHAPTER 7 WOOD

This chapter provides the specific requirements for designing a wood building to resist progressive collapse. Appendix F demonstrates the application of the wood design requirements for a 3-story load-bearing wall building

Wood construction takes several forms in current practice. As described in the 1996 version of AF&PA/ASCE 16, *Load and Resistance Factor Design Manual for Engineered Wood Construction*, wood construction can be categorized as wood frame, noncombustible wall-wood joist, and heavy timber. As most wood construction used for DoD facilities falls under the wood frame category, this is the focus of these provisions. If composite construction with other materials is employed, use the design guidance from the appropriate material chapter in this UFC for those structural elements or portions of the structure.

Note that the combination of design requirements (TF, AP, and ELR) will depend upon the Risk Category of the buildings, as defined in Section 2-2.

### 7-1 MATERIAL PROPERTIES FOR WOOD.

Per ASCE 41, default expected strength values for wood materials shall be based on design resistance values from AF&PA/ASCE 16. In addition, ASCE 41 provides default expected strength values for shear walls and wood diaphragms. When default lower bound strength values are needed, multiply the expected strength values by 0.85.

### 7-2 STRENGTH REDUCTION FACTOR $\phi$ FOR WOOD.

For Tie Force and Alternate Path methods, use the appropriate strength reduction factor specified in ANSI/AF&PA *National Design Specification for Wood Construction* for the component and behavior under consideration. If wood components can be proven capable of carrying the required longitudinal, transverse, and peripheral tie strength while undergoing rotations of 0.20-rad (11.3-deg), use the appropriate strength reduction factor  $\phi$  for each limit state considered.

### 7-3 TIME EFFECT FACTOR $\lambda$ FOR WOOD.

The time effect factor  $\lambda$  for wood is 1.0.

### 7-4 TIE FORCE REQUIREMENTS FOR WOOD.

Apply the Tie Force requirements in Section 3-1, where applicable, for load-bearing wood buildings.

## **7-5 ALTERNATE PATH METHOD FOR WOOD.**

### **7-5.1 General.**

Use the Alternate Path method in Section 3-2, where applicable, to verify that the structure can bridge over removed elements.

### **7-5.2 Modeling and Acceptance Criteria for Wood.**

Use the modeling parameters, nonlinear acceptance criteria and linear m-factors for the Life Safety condition from Chapter 12 of ASCE 41 for primary and secondary components. Use the modeling parameters and guidance, including definitions of stiffness, to create the analytical model.

## **7-6 ENHANCED LOCAL RESISTANCE REQUIREMENTS FOR WOOD.**

Apply the Enhanced Local Resistance requirements in Section 3-3, where applicable, for framed and load-bearing wall wood buildings.

## CHAPTER 8 COLD-FORMED STEEL

This chapter provides the specific requirements for designing a cold-formed steel building to resist progressive collapse.

If composite construction with other materials is employed, use the design guidance from the appropriate material chapter in this UFC for those structural elements or portions of the structure.

Note that the combination of design requirements (TF, AP, and ELR) will depend upon the Risk Category of the buildings, as defined in Section 2-2.

### 8-1 MATERIAL PROPERTIES FOR COLD-FORMED STEEL.

ASCE 41 provides default expected strength values for light metal framing shear walls. When default lower bound strength values are needed, multiply the expected strength values by 0.85.

### 8-2 STRENGTH REDUCTION FACTOR $\phi$ FOR COLD-FORMED STEEL.

For Tie Force and Alternate Path methods, use the appropriate strength reduction factor specified in AISI/COS/NASPEC *AISI Standard North American Specification for the Design of Cold-Formed Steel Structural Members* for the component and behavior under consideration. If cold formed steel components can be proven capable of carrying the required longitudinal, transverse, and peripheral tie strength while undergoing rotations of 0.20-rad (11.3-deg), use the appropriate strength reduction factor  $\phi$  for each limit state considered.

### 8-3 TIE FORCE REQUIREMENTS FOR COLD-FORMED STEEL.

Apply the Tie Force requirements in Section 3-1, where applicable, for load-bearing cold-formed steel buildings.

### 8-4 ALTERNATE PATH METHOD FOR COLD-FORMED STEEL.

#### 8-4.1 General.

Use the Alternate Path method in Section 3-2, where applicable, to verify that the structure can bridge over removed elements.

#### 8-4.2 Modeling and Acceptance Criteria for Cold-Formed Steel.

Use the modeling parameters, nonlinear acceptance criteria and linear m-factors for the Life Safety condition from Chapter 12 of ASCE 41 for primary and secondary components. Use the modeling parameters and guidance, including definitions of stiffness, to create the analytical model.

**8-5      ENHANCED LOCAL RESISTANCE REQUIREMENTS FOR COLD-  
FORMED STEEL.**

Apply the Enhanced Local Resistance requirements in Section 3-3, where applicable, for framed and load-bearing wall cold-formed steel buildings.

CANCELLED

## APPENDIX A

## REFERENCES

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3. General Services Administration  
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GSA Steel Frame Bomb Blast &  
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4. US Army Corps of Engineers  
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Omaha District  
215 N. 17<sup>th</sup> St.  
Omaha, NE 68102-4978  
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Antiterrorism/Force Protection  
Construction Standards, Guidance on  
Structural Requirements  
2001  
Methodology Manual for the Single-  
Degree-of-Freedom Blast Effects  
Design Spreadsheets (SBEDS)  
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5. Department of Defense  
Washington, DC  
UFC 3-301-01 Structural Engineering  
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7. National Institute of Standards and  
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Buildings  
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P.O. Box 9094  
Farmington Hills, MI 48333-9094

ACI 318 Building Code  
Requirements for Structural Concrete

ACI 530 Building Code  
Requirements for Masonry Structures

2. American Forest & Paper  
Association  
American Wood Council  
1111 19<sup>th</sup> Street NW, Suite 800  
Washington, DC 20036

National Design Specification for Wood  
Construction

3. American Institute of Steel  
Construction  
One E. Wacker Dr., Suite 3100  
Chicago, IL 60601-2000

Manual of Steel Construction, Load  
and Resistance Factor Design

AISC 341 Seismic Provisions  
for Structural Steel Buildings

ANSI/AISC 360 Specification for  
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4. American Iron and Steel Institute  
1101 Seventeenth Street NW  
Suite 1300  
Washington, DC 20036-4700

AISI Standard North American  
Specification for the Design of Cold-  
Formed Steel Structural Members

- |   |   |
|---|---|
| 5. American Society of Civil Engineers<br>1801 Alexander Bell Drive<br>Reston, VA 20191-4400                | ASCE 7 Minimum Design Loads for<br>Buildings and Other Structures   |
|   | ASCE 41 Seismic Rehabilitation of<br>Existing Buildings   |
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| 7. International Code Council<br>500 New Jersey Avenue, NW<br>Washington, DC 20001-2070                     | International Building Code   |
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## APPENDIX B DEFINITIONS

### B-1 INTRODUCTION.

Definitions for terminology and for structural analysis concepts are provided in this appendix. Many of the terms in this UFC are provided in other DoD UFCs, instructions, directives, standards, and manuals, as well as in typical non-government standards, such as ASCE 41, AISC Manual of Steel Construction, etc. Those terms of significance to this UFC are included in Section B-2. The definitions for structural analysis procedures are given in Section B-3.

### B-2 TERMINOLOGY.

**Deformation Controlled Action.** A deformation controlled action provides a resistance that is proportional to the imposed deformation until the peak strength is reached, after which the resistance remains at a significant level, as the deformation increases. Classification as a deformation-controlled action is not based on engineering judgment and must follow the guidance presented in Section 3-2.5.

**Expected Strength.** The expected strength of a component is the statistical mean value of yield strengths for a population of similar components, and includes consideration of the variability in material strengths as well as strain hardening and plastic section development. If a statistically determined value for the expected strength is not available, the expected strength can be obtained by multiplying the lower bound strength (i.e., the nominal strength or strength specified in the construction documents) by the appropriate factor from Chapters 9 to 12 in ASCE 41.

**Enhanced Local Resistance (ELR).** ELR is an indirect design approach that provides a prescribed level of out-of-plane flexural and shear resistance of perimeter building columns (including their connections, splices and base plates) and load bearing wall elements, such that the shear resistance exceeds the shear associated with the required out-of-plane enhanced flexural resistance of the columns and wall elements. When the shear strength is reached before the flexural strength, the possibility exists of a sudden, non-ductile failure of the element, which may lead to progressive collapse.

**Force Controlled Action.** A force controlled action provides a resistance that is proportional to the imposed deformation until the peak strength is reached, after which the resistance drops to zero. Classification as a force-controlled action is not based on engineering judgment and must follow the guidance presented in Section 3-2.5.

**Linear Static Procedure.** In a linear static procedure, the structural analysis incorporates only linear elastic materials and small deformation theory; buckling phenomena are not included in the model but are assessed through examination of the output. Inertial forces are not considered. The analysis consists of a single step, in which the deformations and internal forces are solved based on the applied loads and geometry and materials.

**Lower Bound Strength.** The lower bound strength of a component is the statistical mean minus one standard deviation of the yield strengths for a population of similar components. If a statistically determined value for the lower bound strength is not available, the nominal strength or strength specified in the construction documents may be used.

**Nonlinear Dynamic Procedure.** In a nonlinear dynamic procedure, inertial effects and material and geometric nonlinearities are included. A time integration procedure is used to determine the structural response as a function of time.

**Nonlinear Static Procedure.** In a nonlinear static procedure, the structural model incorporates material and geometric nonlinearities. Inertial effects are not included. An incremental or iterative approach is typically used to solve for the structural response as a function of the applied loading.

**Penultimate Column or Wall.** The column or wall that is next to the corner column or corner wall on the exterior surface, i.e., the next-to-last wall or column along the exterior of the building.

**Secondary Component.** Any component that is not a primary component is classified as secondary.

**Story.** That portion of a building between the surface of any one floor and the surface of the floor above it or, if there is no floor above it, then that portion of the building included between the surface of any floor and the ceiling or roof above it.

**Tie Forces.** A tie force is the tensile resistance that is used to transfer the loads from the damaged region of the structure to the undamaged portion. Tie forces can be provided by the existing structural elements that have been designed using conventional design methods to carry the standard loads imposed upon the structure. If an existing structure or a new conventional structure design does not meet the tie force requirements, then new members must be added, or the structure must be redesigned.

### **B-3 DEFINITIONS FOR STRUCTURAL ANALYSIS PROCEDURES.**

**Joint and Joint Rotation.** From ASCE 41, a joint is an area where ends, surfaces, or edges of two or more components are attached; categorized by type of fastener or weld used and method of force transfer. As shown in Figure B-1, a joint is the central region to which the structural members are attached. A joint possesses size, geometry, and material and, as such, the joint can rotate as a rigid body, as shown in Figure B-2. The joint in Figure B-2 is shown as a "+" shape, to facilitate visualization of the joint rotation,  $\Gamma$ .

Typically, deformations within the joint are ignored and only rigid body rotation is considered. However, shear deformations within the panel zone of structural steel and reinforced concrete joints can occur, as defined later.

Figure B-1. Joint and Connection Definition

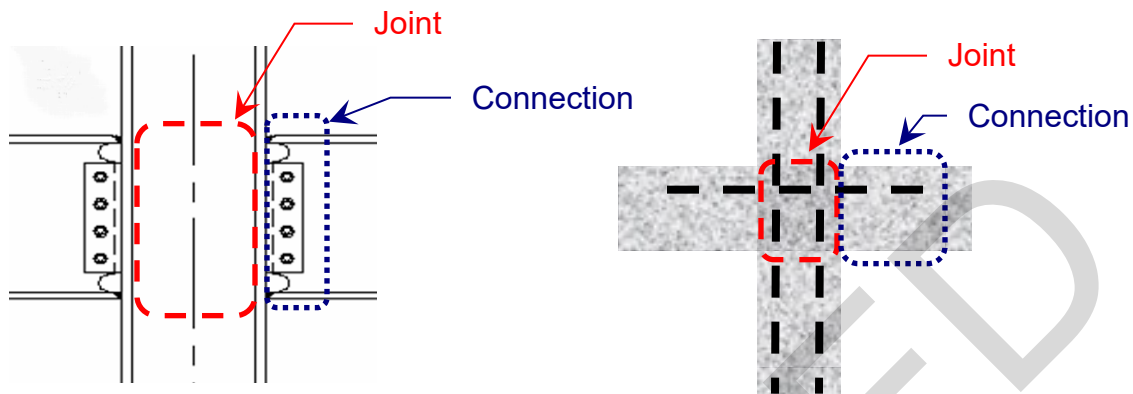
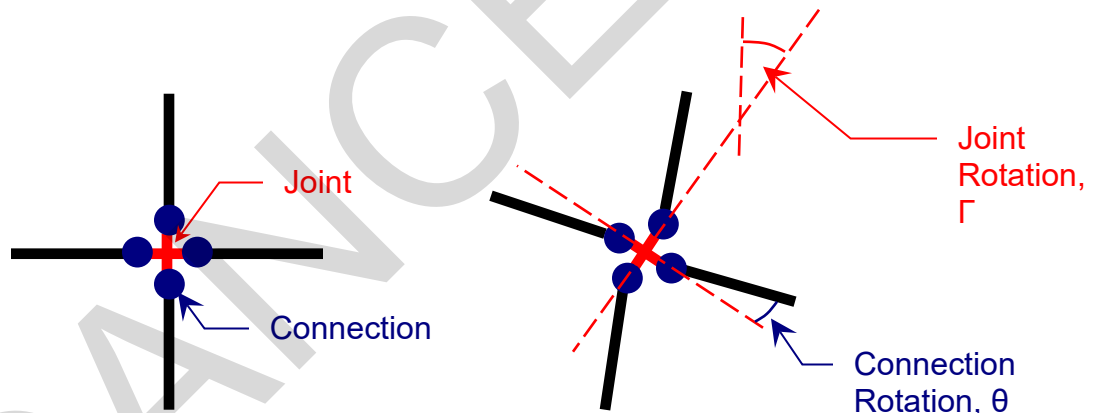


Figure B-2. Joint and Connection Rotations

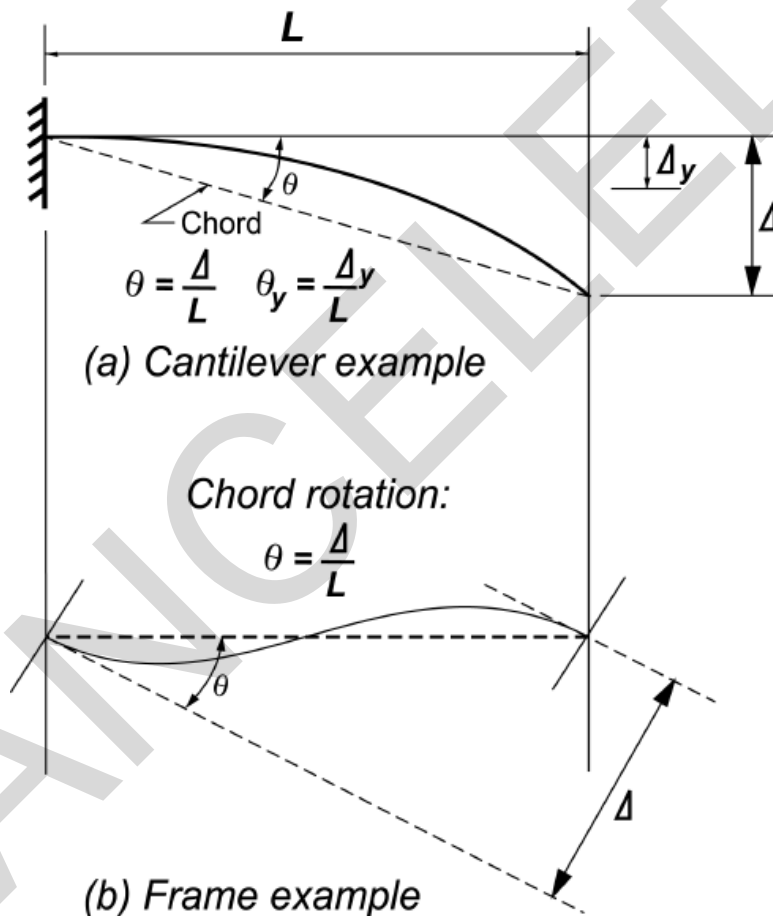


**Connection and Connection Rotation.** A connection is defined as a link that transmits actions from one component or element to another component or element, categorized by type of action (moment, shear, or axial) (ASCE 41). Steel moment and reinforced concrete connections are shown in Figure B-1. The rotation of the connection is shown in the sketches in Figure B-2. Rotation can occur through shear and flexural deformations in the connection and may be elastic (recoverable) or plastic (permanent). The connection rotation is measured relative to the rigid body rotation of the joint as shown in Figure B-2.

In a frame, calculation of the connection rotation is often determined via the chord rotation. In the case shown in Figure B-3, the chord rotation and connection rotation  $\theta$  are identical; however, joint rotation must also be considered. The total connection rotation is the sum of the elastic and plastic rotations, defined later.

In numerical models and design software, connections are typically modeled with discrete “plastic hinges”, which exhibit a linear elastic behavior until the yield plateau is reached; in some models, the elastic rotations are ignored, due to their small value. In this case, the rotation of the discrete plastic hinge model is the connection rotation; care must be taken to insure that the rotation of the plastic hinge model only considers the connection rotation  $\theta$  and does not also include the joint rotation  $\Gamma$ .

**Figure B-3. Definition of Chord Rotation (from ASCE 41)**



**Yield Rotation.** Many flexural elements will deform elastically until the extreme fibers of the element reach their yield capacity and the response becomes nonlinear. While the depth of the yielded material in the cross section will gradually increase as the moment is increased, this portion of the response is typically assumed as a finite change in the slope of the moment vs. rotation curve, as shown in Figure B-4. The yield rotation  $\theta_y$  corresponds to the flexural rotation at which the extreme fibers of the structural elements reach their yield capacity  $f_y$ . This is also called the elastic rotation as it corresponds to the end of the elastic region.

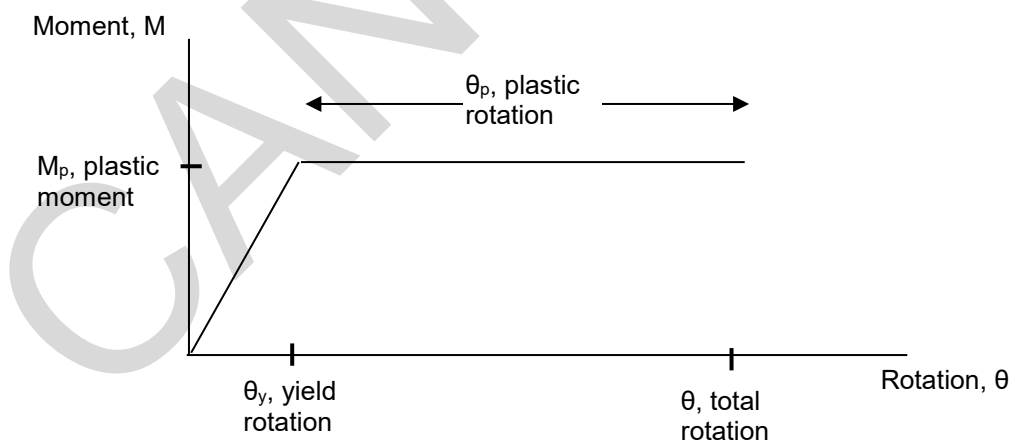
For steel beams and columns, ASCE 41 allows  $\theta_y$  to be calculated as follows, where it has been assumed that the point of contraflexure occurs at the mid-length of the beam or column.

$$\text{Beams: } \theta_y = \frac{ZF_{ye}I_b}{6EI_b}$$

$$\text{Columns: } \theta_y = \frac{ZF_{ye}I_c}{6EI_c} \left(1 - \frac{P}{P_{ye}}\right)$$

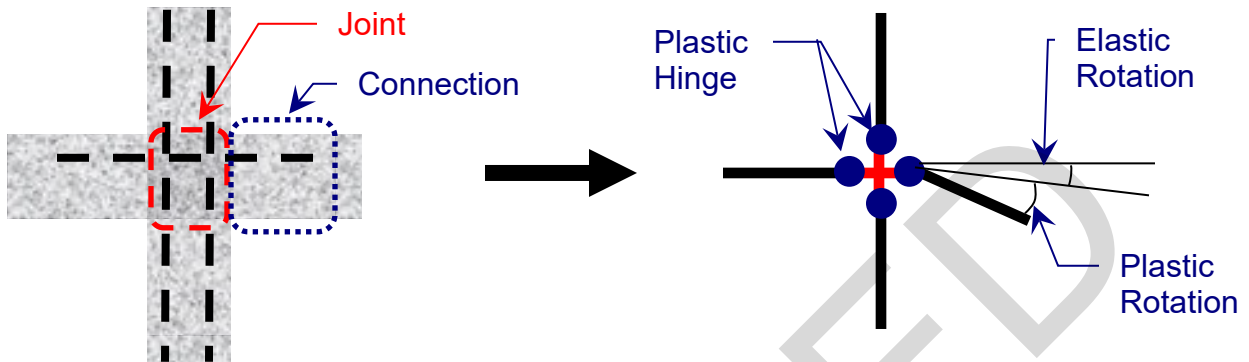
For steel structures, in ASCE 41, multiples of the yield rotation  $\theta_y$  are used to define the acceptance criteria and modeling parameters in terms of plastic rotation for a number of elements (beams, columns, shear walls).

**Figure B-4. Definition of Yield Rotation, Plastic Rotation, and Total Rotation**



**Plastic Rotation and Plastic Hinge.** The plastic rotation  $\theta_p$  is the inelastic or non-recoverable rotation that occurs after the yield rotation is reached and the entire cross section has yielded; see Figure B-4. The plastic rotation  $\theta_p$  is typically associated with a discrete plastic hinge that is inserted into a numerical frame model, as shown in Figure B-5. The plastic hinge measures both elastic and plastic rotations, although for simplicity, the elastic portion is often ignored due to its small size.

Figure B-5. Plastic Hinge and Rotation



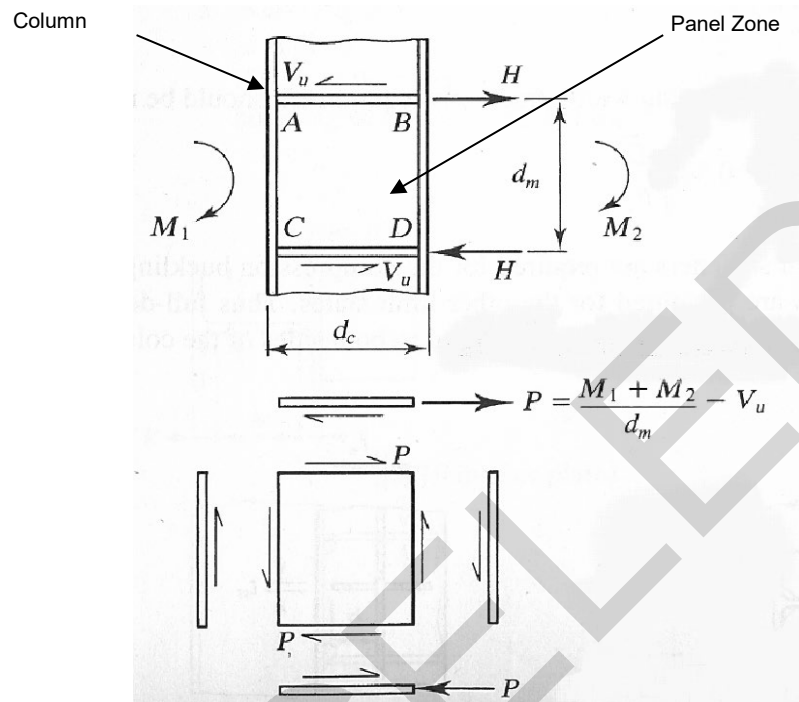
For both steel and concrete, ASCE 41 specifies the acceptance criteria and the modeling parameters in terms of plastic rotation. For some steel structural elements, the criteria parameters are given in terms of multiples of the yield rotation  $\theta_y$ ; for concrete and the remainder of the structural steel elements, a numerical value for the plastic rotation is given, in units of radians.

**Total Rotation.** The total rotation  $\theta$  is the sum of the yield rotation  $\theta_y$  and the plastic rotation  $\theta_p$ .

**Panel Zone.** In steel frame structures, the panel zone is the region of high shear stress in the column web within the boundaries of the joint, which results from the large moment transferred to the column joint from a fully restrained connection; see Figure B-6. The panel zone is an integral part of the steel frame beam-to-column moment connection. The deformation measure is the plastic angular shear rotation. Guidance for including or excluding the panel zone in steel models is given in Sections 9.5.2.2.1 and 9.5.2.2.2 in ASCE 41.

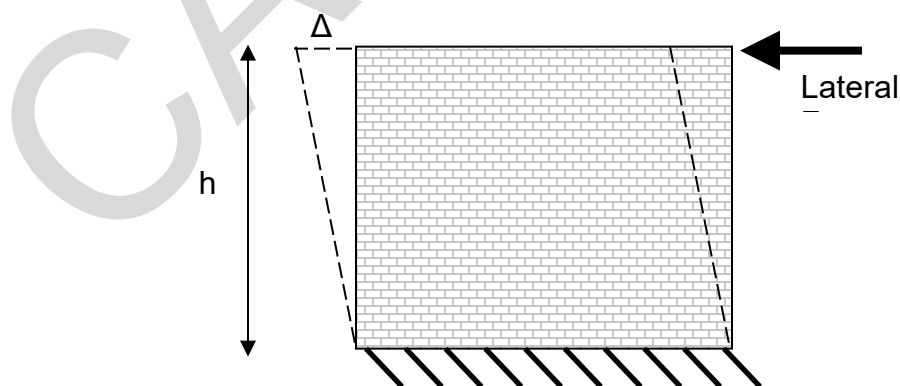
Similarly, for beam-column joints in reinforced concrete framed structures, the plastic shear rotation is the deformation parameter used in the acceptance criteria; in ASCE 41, only the secondary beam-column joints must be checked for shear rotation.

Figure B-6. Panel Zone



**Story Drift (Wall Structures).** In ASCE 41, story drift is used as the nonlinear deformation measure for load-bearing wall structures (masonry, wood, and cold formed steel). The story drift is defined as the ratio of the lateral deflection at the top of a wall segment  $\Delta$  to the overall height of the wall segment, as shown in Figure B-7.

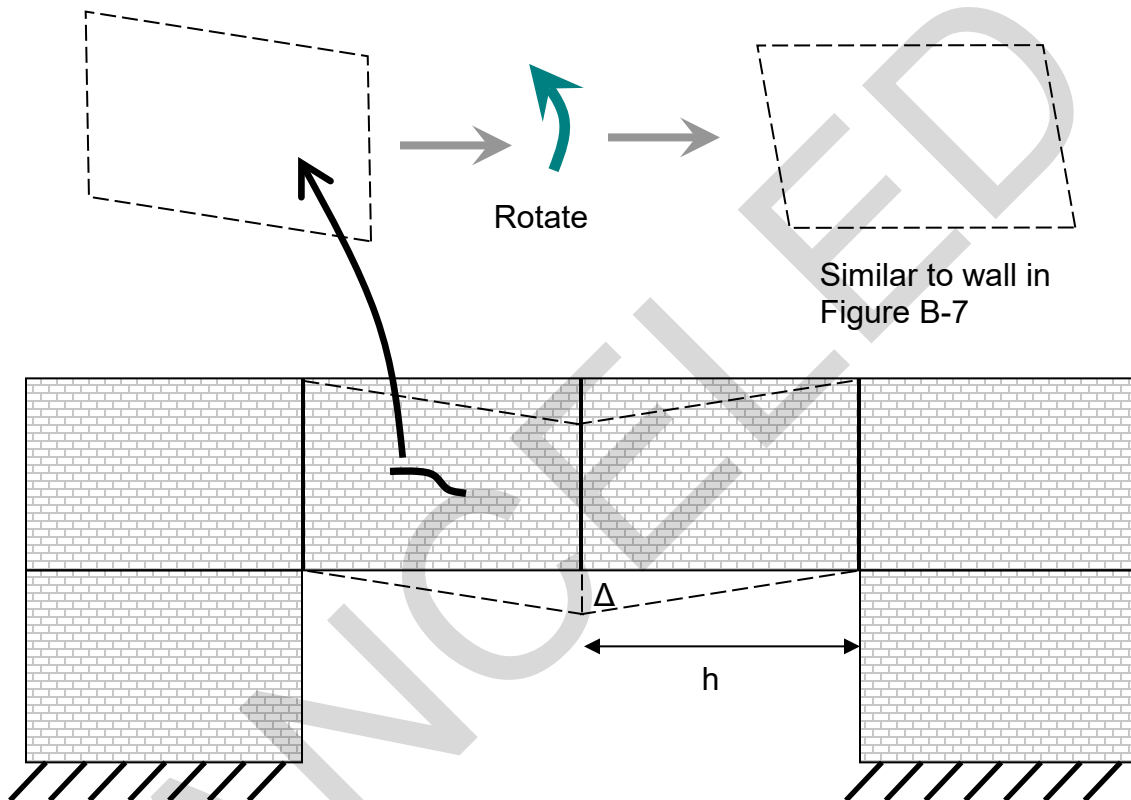
Figure B-7. Story Drift



While the story drift deformation criteria in ASCE 41 are applied to horizontal deformations due to lateral earthquake loads, this information can be used directly for

progressive collapse analysis with vertical deformations due to removed wall sections, as shown in Figure B-8.

**Figure B-8. Vertical Wall Deflection (Drift)**





## **APPENDIX C      COMMENTARY**

### **C-1      INTRODUCTION.**

The goal of these design requirements is to provide a rational and uniform level of resistance to progressive or disproportionate collapse. \4\ /4/ These requirements are threat-independent and are not intended to provide resistance to the local damage that may initiate the progressive collapse. Discussion and justification for the applicability requirements, design approaches, modeling techniques, and acceptance criteria are provided in this Appendix.

### **C-2      APPLICABILITY.**

#### **C-2.1      Three Story Requirement and Story Definition.**

The required minimum height of 3 stories for progressive collapse design is taken from the original DoD guidance (DoD 2001). This requirement was based on a minimum threshold of 12 casualties in a progressive collapse event where it was assumed that the 2 bays on either side of a removed column or wall would collapse on each of 3 floors and that each bay/room would house 2 persons. Thus, the justification for setting the limit at 3 stories was determined by the level of casualties and not by the mechanics of progressive collapse as a function of structural characteristics.

As casualties are the key metric, a basement or penthouse structure is defined to be a story if it is occupied. The definition of “occupied” in the International Building Code (IBC) is: “A room or enclosed space designed for human occupancy in which individuals congregate for amusement, educational or similar purposes or in which occupants are engaged at labor, and which is equipped with means of egress and light and ventilation facilities.” This definition was adopted in Section 1-2.1. Further, as noted in Section 1-2.1, any story that will not be occupied does not count towards the limit of 3 stories; this may include floors that house mechanical equipment or are used for storage.

Any portion of a building that is less than 3 stories is not required to meet the progressive collapse design requirements of this UFC. However, any deleterious effect from the attachment of a short section of the building to the 3 story or higher section must be considered. In particular, peripheral tie forces must be placed in the 3+ story section of the building, at the boundaries between the short section and 3+ story section. For Alternate Path, the structural elements of the short section must be considered in the analysis and design of the 3+ story section if there is any possibility that the presence of the short section will affect the 3+ story section in a negative manner.

## **C-2.2 Clarification for Partial Occupancy.**

UFC 4-010-01 *DoD Minimum Antiterrorism Standards for Buildings* requires that: “These standards only apply where DoD personnel occupy leased or assigned space constituting at least 25% of the net interior useable area or the area as defined in the lease, and they only apply to that portion of the building that is occupied by DoD personnel.” This 25% space threshold might be met by lease of entire stories in a multi-story building, e.g., DoD might lease the 3<sup>rd</sup> and 4<sup>th</sup> story in a 7-story building. As it is impractical to design or retrofit a building to resist collapse on only certain stories, this requirement from UFC 4-010-01 has been superseded by the requirement in Section 1-2-2.

## **C-3 RISK CATEGORIES.**

In the 2005 UFC4-023-03, the level of progressive collapse design was based on the level of protection (LOP), which, in turn, was based on the asset value of the building, as calculated with UFC 4-020-01 *DoD Security Engineering Facilities Planning Manual*. The asset value was a function of different asset categories, including General Population, Critical Infrastructure and Operations and Activities, Sensitive Information, and All Other Assets, including Mission Critical Personnel. Thus, there was a strong dependence upon the level of occupancy and the criticality to the user. In essence, this is a “consequence approach” in that probability of occurrence and the associated risk for progressive collapse cannot be explicitly considered due to the very small database of progressive collapse events. Thus, the level of casualties and the degradation of function are the key considerations.

Beginning with the 2009 UFC 4-023-03, different levels of design requirements are specified, depending upon the Risk Category (RC). The RC is based on the Risk categories defined in Table 2-2 Risk Category of Buildings and Other Structures in UFC 3-301-01 *Structural Engineering*. The descriptions for “Nature of Occupancy” in Table 2-2 are very similar to those in ASCE 7 Minimum Design Loads for Buildings and Other Structures, however, some modifications specific to DoD have been made.

It is noted that the RC is independent of threat or initiating event, and, as with the previous LOP approach, this is consequence-based where Risk level and function are key parameters in defining the level of progressive collapse design.

## **C-4 DESIGN REQUIREMENTS.**

The design requirements for RC I through RC IV are listed in Table 2-2 and briefly summarized in the following sections.

#### **C-4.1 RC I Design Requirement.**

These buildings present little risk to human life and no progressive collapse design is required providing the buildings were designed to the extant building code.

#### **C-4.2 RC II Design Requirement.**

For RC II structures, one of two options must be chosen: Option 1, Internal, peripheral and vertical Tie Forces with Enhanced Local Resistance for the corner and penultimate columns or walls at the first story **OR** Option 2, Alternate Path applied to specific locations. For load-bearing wall structures, the AP method may be the best choice, as the designer can take advantage of the building's inherent redundancy as well as the ability to develop deep beam or arching action.

In the 2005 UFC 4-023-03, only tie forces were used for LLOP (i.e., RC II) buildings. Tie Forces can be very difficult to implement in existing buildings and even for some new types of load-bearing wall construction. Since many load-bearing wall buildings are very redundant and may meet the Alternate Path requirements while staying elastic, Option 2 (as suggested in the Eurocode) was added. Thus, this provides some relief for existing buildings. For many load-bearing buildings, the walls are identical and a single set of calculations for a typical wall may be sufficient.

##### **C-4.2.1 RC II Option 1, Tie Forces and Enhanced Local Resistance.**

The goal of the Tie Force requirement is to enhance the structural integrity evenly throughout the structure, by prescriptively defining the magnitude, location, and distribution of the Tie Forces and without requiring significant design or analysis effort. While the Tie Forces are distributed uniformly throughout the structure, the response and performance of the structure varies with the location at which the initial damage occurs. As discussed in Stevens 2008, the removal of a corner column or wall or a penultimate corner or wall can lead to local collapse of a portion of the bay since the lateral support to anchor the Tie Forces has been removed or reduced. This damage will extend to the height of the building, but is unlikely to progress horizontally. While this damage is spatially limited and does not threaten the rest of the building, it is a limitation of the Tie Force approach. Therefore, to reduce the possibility that the corner or penultimate column or wall will be damaged, the Enhanced Local Resistance approach is applied to these elements at the first story above grade, as discussed in Section 3-3.

##### **C-4.2.2 RC II Option 2, Alternate Path.**

While the Tie Force requirement can be easily implemented in new construction for some material types, it can be difficult to apply to existing buildings and to non-ductile floor systems. The option to use the Alternate Path method provides

another approach by which to evaluate an existing structure. It also allows the designer to account for the inherent and often substantial collapse resistance due to the natural redundancy and available load paths, commonly found in load-bearing wall structures. Many of the structures in RC II will be short (5 stories or less) load-bearing wall buildings with a uniform or regular layout. Hand calculations can be used to demonstrate bridging by deep beam action or arching over removed wall sections for a typical wall and those results applied to the similar walls in the structure.

#### **C-4.3 RC III Design Requirement.**

For RC III, two requirements must be satisfied: Alternate Path and Enhanced Local Resistance. The consequence of collapse is greater for this Risk Category III, which also increases the (unknown and unquantifiable) probability of a deliberate attack. Thus, a specified level of resistance to loss of a column or wall is provided by the Alternate Path method. Additional protection is provided by minimizing the likelihood of a non-ductile failure of the columns and walls at the building perimeter, in the first story above grade, through the Enhanced Local Resistance requirement.

For RC III (and IV), the buildings will tend to be large, framed structures, and the specified locations for column or wall removal are only the minimal locations that must be considered; the engineer must also consider locations where the geometry of the structure changes significantly. Since the regular portion of the structure should be covered by the minimum cases and all unusual portions of the structure should be identified by the engineer, this requirement applied to all columns or load-bearing walls in the structure. However, note that for RC III (and RC IV) structures without underground parking or other areas of uncontrolled public access, internal column removal does not need to be considered.

#### **C-4.4 RC IV Design Requirement.**

For RC IV, three requirements must be satisfied: Alternate Path, Tie Forces, and Enhanced Local Resistance. The addition of the Tie Force requirement to those of RC III provides another layer of resistance to collapse and will supplement the flexural resistance developed through the AP method. In addition, the ELR requirement is applied to all perimeter walls and columns, over the first story above grade and the level of flexural resistance is increased, to minimize the possibility that two columns or walls at the same level will be removed in the same event.

### **C-5 TIE FORCES.**

#### **C-5.1 General.**

The Tie Force requirement is designed to enhance the structural integrity of the building by prescriptively defining tensile force strength of the members and connections, in terms of strength, location, and distribution. This prescriptive method is simple in that detailed or complicated models and analyses are not required, yet it must

also be based on mechanical principles, such as equilibrium and deformation compatibility, as applied to a damaged structure.

### **C-5.2 Previous Requirements.**

The Tie Force requirements in the 2005 UFC 4-023-03 were based on the British Building Standards (before the Eurocodes were introduced) for reinforced concrete, structural steel, and masonry. The previous Tie Force requirements were

material-specific, with limited similarity across the different materials in terms of location, distribution, magnitude, and other details. As discussed in the 2005 UFC 4-023-03, the Tie Forces for Reinforced Concrete could be related to an assumed catenary behavior of the floor system, but similar justification could not be found for the other materials.

One common and justified criticism of the previous Tie Force approach was that the great majority of steel connections as well as some RC connections are not capable of providing the magnitudes of rotation that are needed to develop the typically small Tie Forces that were specified. Thus, the connections would fail before the beams, girders, and spandrels could develop axial force. This is also true for Tie Forces distributed in floor systems with limited ductility, such as plywood on engineered I-joists or precast planks with limited continuity across connections.

### **C-5.3 New Tie Force Approach.**

Due to the inability of many connections to sustain large rotations, a new approach was proposed and employed beginning with the 2009 UFC 4-023-03. In this new approach, the floor system now provides and carries the internal Tie Forces, thus removing these Tie Forces from the beams, girders, and spandrels. In essence, the floor system will transfer the vertical loads from the damaged section, via catenary or membrane action, to the undamaged horizontal members, which, in turn, will transfer the load into the vertical load carrying elements, as shown in Figure C-1.

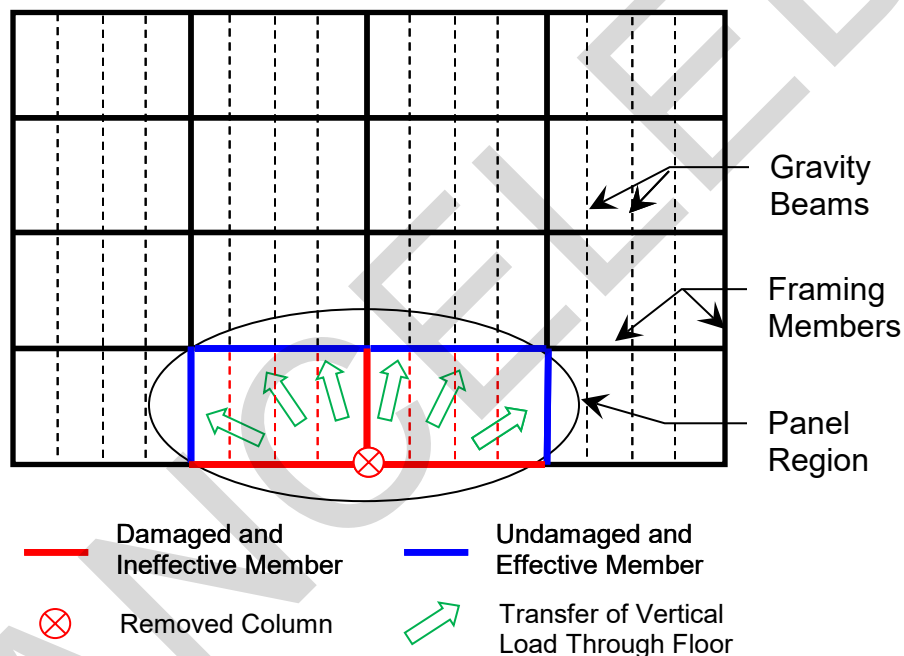
While the internal and peripheral ties are now placed in the floor system, a designer is allowed to use the members and connections for steel, reinforced concrete and other materials, if it can be shown that the connections can carry the tensile forces due to a removed column/wall without failure due to large deformations and rotations.

Additional modifications from the previous UFC include:

- Requirements are now material-independent,
- Explicit tying to external and corner walls and columns is removed,
- Provisions to address openings in the floor system (stairwells, elevators, atria) are included,
- Provisions are added to account for large variations in floor loads over the plan geometry of a single floor.

#### C-5.4 Justification for the Tie Force Approach.

In the development of the 2009 UFC 4-023-03, analytical and numerical methods were used to derive reasonable tie force requirements that can be used for different column and wall loss locations for braced frames, moment frames, and load-bearing wall structures, with floor systems that are capable of developing membrane or catenary response; the details can be found in Stevens 2008. These floor systems include reinforced concrete (RC) floors with integral slabs, composite construction with steel decks and RC, and floor systems that incorporate a grid of rebar or welded wire fabric.



**Figure C-1. Damaged and Undamaged Structural Elements**

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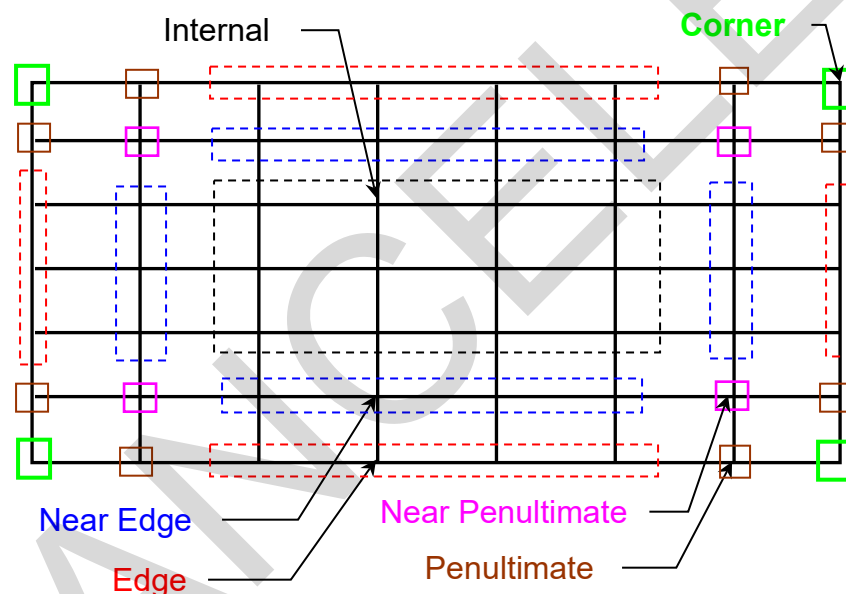
##### C-5.4.1 Tie Forces for Framed and Two-way Load-bearing Wall Buildings.

To develop the internal and peripheral tie force requirements for framed structures, 6 different column scenario removals were assessed, using simple catenary theory, membrane theory, and finite element analysis; see Stevens 2008. The removal locations included the corner column, penultimate column, internal column, near penultimate column, edge column, and near edge column, as shown in Figure C-2. In addition to assessing the tie force magnitudes required to carry the loads in catenary or membrane action, the transfer of the vertical force from the damaged panel to the undamaged structures was assessed. Finite element analyses were also performed to determine the dynamic effects created by the sudden loss of column support; the results of these analyses were used to modify the Tie Force equations. Load-bearing wall

structures were also considered. As the floor system is the critical element for developing and supplying the internal and peripheral tie forces, many of the findings from the framed structure assessment are directly applicable to load-bearing wall structures.

The application of peripheral ties around the perimeter of openings was also investigated with numerical methods and the loads were shown to be adequately transferred from the damaged area to the peripheral ties and to the undamaged elements. Care must be taken to develop, lap or anchor the peripheral ties at openings

in the floor system (stairways, elevators, atria, etc), such that the strength can be developed. This same approach was used to develop the sub-areas and peripheral ties at the boundary between sub-areas with different floor loads.



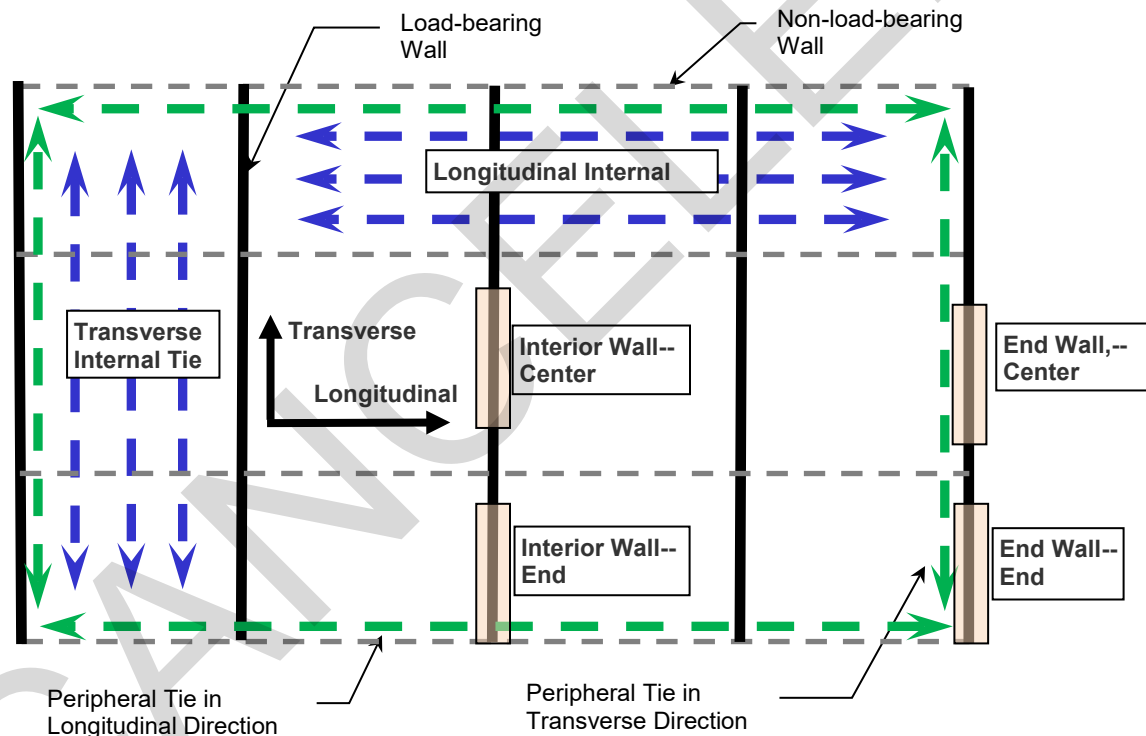
**Figure C-2. Column Removal Locations**

#### **C-5.4.2 One-way Load-bearing Wall Buildings.**

The internal tie force equations for one-way load-bearing wall structures in the 2009 version of UFC 4-023-03 were based on the Eurocode. A series of finite element analyses were subsequently performed to investigate 1) the use of  $5h_w$  in determining the internal tie forces in the direction parallel to the load-bearing walls; and 2) the effect of loads from the removed section of the load-bearing wall on the internal and peripheral ties. As noted in Paragraph 2-2.2.2 design of this structural system to resist progressive collapse is often best addressed with the alternate path approach.

Figure C-3 shows the plan view of a one-way load-bearing wall building. To determine the required magnitude of the internal and peripheral tie forces, different wall removal locations must be analyzed; in all cases, the length of the removed wall is  $2h_w$

where  $h_w$  is the clear story height. Four wall removal locations were considered, shown by the shaded boxes in Figure C-3: End wall--center; End wall--end; Interior wall--center; and, Interior wall--end. The results of the numerical simulations for the first two wall removal locations verified that tie forces are not an effective mechanism for resisting progressive collapse when the removed wall sections are at the end of the building. The end walls pull into the building and the wall and floor fail, as there is no means for resisting the lateral forces created by the tie force members; this fact was the motivation behind the requirement to apply Enhanced Local Resistance to the end and penultimate load-bearing walls for \3\ Risk Category /3/ 2 Option 1, in the 2009 version of UFC 4-023-03. A similar result will occur for the removal of the end and center wall sections for the penultimate load-bearing walls. Therefore, the numerical simulations were used to assess the second two cases: interior wall-center and interior wall-end.

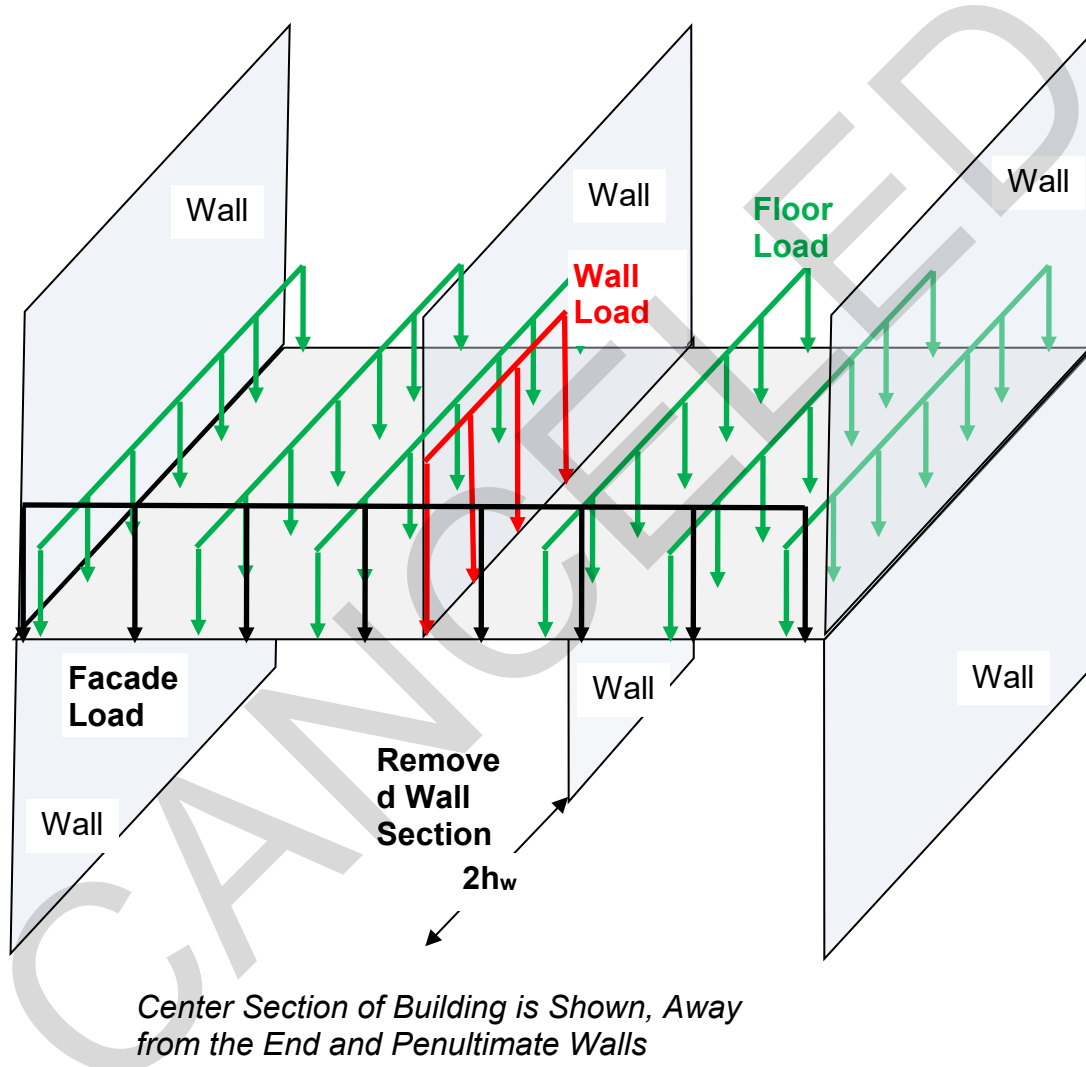


**Figure C-3. Plan View of One-way Load-bearing Wall Structure**

The purpose of the internal and peripheral tie forces is to transfer the loads of the floor section that is no longer vertically supported to the portions of the structure that are undamaged. For the case of framed structures, this means that each floor must carry its own dead and live load and, thus, each floor can be considered as a separate entity. This same approach holds for load-bearing wall structures, with the addition that the dead load must now include the weight of the walls above the removed wall sections as well as the façade load, if the façade is tied into the floor system. Thus, an idealized representation of the loads on a typical floor above the wall removal location is as shown in Figure C-4.



A series of finite element simulations were performed, using a similar approach as discussed for frame buildings. As reported in Stevens 2008, two levels of façade load (500-plf and 2000-plf) and two levels of wall loads (300-plf and 1500-plf) were considered. The deformation criteria were based on a 10% sag over a double span length; for these analyses, the double span length is in the longitudinal direction as shown in Figure C-4. The results of the analyses were used to create Equation (3-7); details can be found in Stevens 2008.



**Figure C-4. Loads in a One-way Load-bearing Wall Structure**

/2/

### C-5.5 Tie Forces in Roof Systems

The roof system must meet the Tie Force requirements in Section 3-1. As with floor systems, these requirements will be more easily met with some types of roof systems, such as reinforced concrete slabs and composite decks.

For lightweight systems such as steel deck and joist roof systems, sufficient strength and ductility should be available in the direction of the joist, providing that the bottom chord of the joist does not connect to the column or beam; see Figure C-5. If so, then the top chord of the joist can be used to supply the Tie Force, providing that the load path is continuous across each vertical support. Two approaches can be used in the transverse direction. In the first, steel shapes (rods, angles, bars, etc.) could be placed through the open webs. Attachments of these internal ties to the peripheral ties must be capable of developing the longitudinal and transverse tie forces. Second, a steel deck that spans in the transverse direction to the joist could be used to supply the internal Tie Force provided that the welds or other connections between the steel deck sections are sufficient to develop the Tie Force. While there may be crushing of the steel deck where it crosses the joist or other vertical support, the upper surface of the deck should remain intact and capable of supplying tension up to 0.20-rad; see Figure C-6.

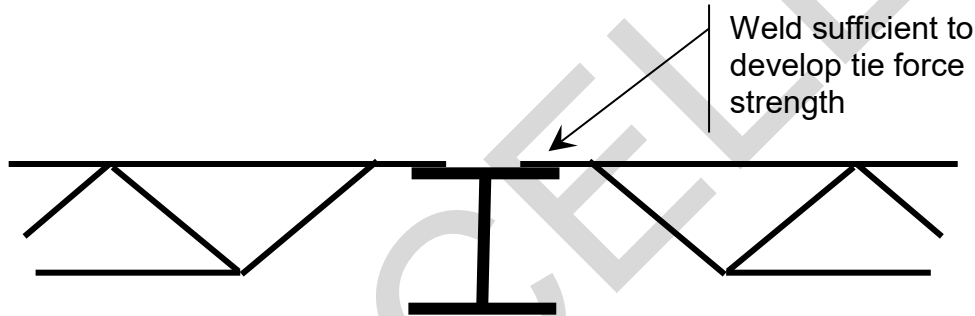


Figure C-5. Tie Force in Upper Chord of Roof Joist

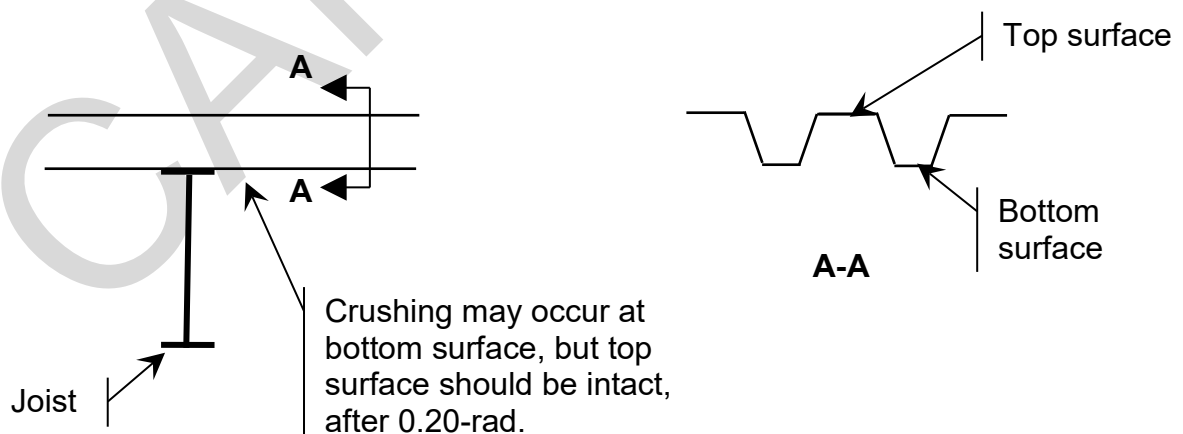


Figure C-6. Tie Force in Steel Deck

### **C-5.6 Location Restrictions on Internal and Peripheral Ties.**

As mentioned, one goal of the revised Tie Force approach is to remove the Tie Forces from the flexural members, which typically are not capable of sustaining the large amount of deformation associated with catenary and diaphragm action. For example, if a peripheral tie was placed in a deep edge beam with limited ductility, the peripheral tie may be damaged or failed by the resulting motion of the edge beam after a column is removed. Within the floor plan, the internal ties can be shifted to either side of the beam, girder or spandrel for framed structures; for flat plate or flat slab structures without edge beams or internal beams, the tie forces can be placed on the column lines and pass through the columns. However, the peripheral ties also need to be close to the edge of the structure and therefore the peripheral ties are permitted to be close to the inner edge of the beam, girder or spandrel. While the portion of the slab/floor next to the beam may respond as a flange of the beam early in the deflection, at some point the beam and its action will be separated from the slab and the peripheral tie will function as intended.

### **C-5.7 Consideration for Non-Uniform Load Over Floor Area.**

Since the load magnitude may vary significantly over the plan area of a given story, e.g. manufacturing activities may be located in one section of the floor and office space in another, the concept of sub-areas is used to accommodate the differences in longitudinal, transverse, and peripheral ties that result from the load variation.

The approach in Paragraph 3-1.3 is illustrated in Figure 3-2 for the case of two sub-areas. In principle, multiple sub-areas can be used across a floor system, but the designer is encouraged to minimize the number of sub-areas, to reduce the number of unique rebar layouts as well as the potential for errors in construction.

The peripheral ties between the sub-areas must be satisfactorily anchored or embedded such that the full tensile strength can be developed.

## **C-6 ALTERNATE PATH METHOD.**

### **C-6.1 General.**

In the Alternate Path (AP) method, the designer must show that the structure is capable of bridging over a removed column or section of wall and that the resulting deformations and internal actions do not exceed the acceptance criteria. Three analysis procedures are permitted: Linear Static, Nonlinear Static, and Nonlinear Dynamic.

These procedures were re-evaluated for the 2009 UFC 4-023-03. An assessment of analysis methods in the related field of seismic design revealed that the procedures specified in ASCE 41 *Seismic Rehabilitation of Existing Buildings* could be adopted and modified for application in the 2009 UFC 4-023-03. While progressive

collapse design and seismic design are distinctly different, the general ASCE 41 approach was adopted for the following reasons:

- ASCE 41 and UFC 4-023-03 deal with extreme events that severely damage structures which must not collapse or otherwise imperil the occupants.
- The ASCE 41 methodology was developed and vetted by a panel of structural engineering experts over many years of effort and could be modified in a straightforward manner for progressive collapse design.
- Five materials are considered: steel, RC, masonry, wood, and cold formed steel, in ASCE 41 and UFC 4-023-03.
- Explicit requirements and guidance for analyzing and designing multiple building types for each material are provided in ASCE 41.
- Careful attention is given in ASCE 41 to deformation- and force-controlled actions, as well as primary and secondary components.
- The acceptance criteria and modeling parameters in ASCE 41 can be scaled for different structural performance levels.

The most significant differences between the physics, intent, and approaches underlying UFC 4-023-03 and ASCE 41 are:

- **Extent.** The seismic event involves the entire structure, whereas, for progressive collapse, the initial event is localized to the column/wall removal area.
- **Load Types.** Seismic loads are horizontal and temporary; for progressive collapse, the loads are vertical and permanent.
- **Damage Distribution.** For earthquake design, it is accepted that the damage will be distributed throughout the structure. For progressive collapse, the initial damage is localized and the goal is to keep the damage from progressing.
- **Connection and Member Response.** In typical tests to evaluate the seismic performance of connections and members, cyclic loads with increasing magnitude are applied, without axial loading, and the resulting curves are used to develop “backbone” curves. In progressive collapse, the connection and member experiences one half cycle of loading, often in conjunction with a significant axial load, due to large deformations and catenary response.

These differences have been accommodated in the adaptation of ASCE 41 procedures and criteria to Alternate Path modeling and design for progressive collapse. The significant elements of the Alternate Path method are presented in the following paragraphs.

## **C-6.2 Peer Review.**

In the 2005 UFC 4-023-03, a peer review was required for Alternate Path design of medium and high level of protection buildings. In both the 2009 UFC 4-023-03 and in this UFC, a peer review is no longer required as this is a policy issue that is best addressed by the building owner. However, peer reviews are strongly recommended, for Alternate Path design in any \3\ Risk Category /3/.

## **C-6.3 Alternative Rational Analysis.**

Any rational alternative analysis procedure that is based on fundamental principles of engineering mechanics and dynamics may be used. For load-bearing wall structures with uniform and regular wall layouts or simple frame structures, hand calculations or spreadsheet applications may be appropriate and more efficient. New software design and analysis tools, based on novel analytical formulations, may be used as well. However, any alternative rational analyses must incorporate or satisfy the following:

- the acceptance criteria contained in Section 3-2.10 and in Chapters 4 through 8.
- the specified locations and sizes of removed columns and load-bearing walls in Section 3-2.9.
- the ASCE 7 extreme event load combination.
- the load increase factors and dynamic increase factors in Sections 3-2.11.5 and 3-2.12.5 for linear static and nonlinear static analyses, respectively.
- the requirements of Section 3-2.11.1 must be met for a Linear Static analysis.

All projects using alternative rational analysis procedures shall be reviewed and approved by an independent third-party engineer or by an authorized representative of the facility owner.

## **C-6.4 Load and Resistance Factor Design.**

Load and Resistance Factor Design (LRFD) continues to be used in this version of UFC 4-023-03 as well as the ASCE 7 extraordinary event load combination is employed. Also, unlike ASCE 41, strength reduction factors are employed in determining the design strength. The strength reduction factors account for deficient material strength, construction errors, design flaws and other uncertainties that can act to reduce the strength of the building; all of these uncertainties are “locked” into the building when it is constructed and will still be there when a progressive collapse event occurs. Therefore, the strength reduction factors, load factors, and the LRFD approach continue to be employed in this version of UFC 4-023-03.

## **C-6.5 Primary and Secondary Components.**

The designation of elements, components and connections as primary or secondary is left to the judgment of the engineer; however, in all cases, the engineer must verify that the structure and its elements, components and connections are capable of meeting the structural acceptance criteria in Paragraph 3-2.10.

For evaluation of existing buildings, the engineer may wish to include elements that are typically considered secondary, i.e., gravity beams, slabs, infill walls, etc. If such elements are included as part of the system that resists the vertical loads and collapse, they become primary components by definition and must meet the primary component acceptance criteria.

### **C-6.5.1 Secondary Components.**

While secondary components are designated by the engineer as not contributing to the resistance of gravity loads and progressive collapse, they are a critical part of the load path for the vertical loads and they pose a risk to the building occupants if they drop into the space below, potentially creating additional damage and collapse. As an example, the gravity beams in a bay supporting heavy mechanical equipment could be treated as secondary components; however, the shear tab connections with a deep bolt group could have reduced allowable rotations/m-factors such that the rotations from the column removal could be sufficient to fail the shear tab connections. Secondary components are not included as part of the models in the linear or nonlinear procedures but must be checked against the acceptance criteria given in this UFC and in ASCE 41.

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### **C-6.5.2 Linear Static Secondary Component Acceptance Criteria.**

While secondary components such as gravity beams and simple shear tab connections are not included in the linear static model, these members may fail if subjected to large dynamic loads and/or significant deflections due to column/wall removal and may pose a hazard to occupants or lead to additional structural failure if they detach and impact the floor below. These members must be checked with the same level of attention as the primary members. For linear procedures, the secondary component must meet the force- and deformation-controlled criteria of Equations 3-13 and 3-14.

Before the column or wall is removed, the secondary component will be initially stressed and deformed due to the deformation-controlled or force-controlled load combinations given in Equations 3-10 and 3-12, respectively. When the column or wall is removed, additional stresses and deformations are created. As a linear static procedure is being used, these two sets of demands can be directly added. Two steps are required. First, for each secondary component or connection, the force- or deformation controlled load case is applied to the secondary component in the un-

deformed configuration and the demand (internal shear, moment, axial force) due to this load is calculated. Second, the displacements and rotations from the linear static model with the removed column/wall are then applied to the secondary component (without the gravity load) and the resulting demand is calculated. The two demands (due to load and due to deformed structure) are added together to determine the total demand. This approach is used in the steel example in Appendix E.

As Appendix E shows, acceptance checks of gravity beams and simple shear tab connections (secondary components) in steel frame structures present a unique challenge. While force- and deformation-controlled actions can be checked in a straight-forward manner with nonlinear procedures, the linear static procedure and criteria are based on m-factors applied to the moments and other deformation-controlled actions and thus moments must be determined to perform the checks, even at the simple connections and the ends of gravity beams which are often considered to be pinned. While allowable plastic rotations are available for nonlinear static and nonlinear dynamic procedures, they cannot be used for checks in the linear static method, as the analysis procedures are different, in terms of load adjustment factors, explicit consideration of hinges, fidelity of the nonlinear models, etc. Thus, the linear static acceptance criteria must be based on moments, shears, and other forces. As simple shear tab connections can be considered partially restrained (PR) connections, their flexural strength can be calculated with an approximate rotational stiffness and the overall rotations, for comparison to the flexural demand, as shown in Appendix E. Similar approaches must be devised and used for reinforced concrete, masonry, wood, and cold-formed steel structures./2/

## **C-6.6 Analysis Procedures.**

### **C-6.6.1 Linear Static.**

The Linear Static approach in the 2005 UFC 4-023-03 had been replaced with an “m-factor” procedure, very similar to that defined in ASCE 41. The two significant departures from the ASCE 41 procedure are in the definition of the “Irregularity Limitations” in Paragraph 3-2.11.1.1 and the use of a load increase factor appropriate for progressive collapse loading. The irregularity limitations have been adjusted due to the inherent difference between lateral/seismic loading and vertical/progressive collapse loading and the related criticality of different building geometric and strength features. As discussed in Section C-6.8, a new load increase factor to account for nonlinearity and dynamic effects has been implemented.

### **C-6.6.2 Nonlinear Static.**

The Nonlinear Static procedure is similar to that specified in the 2005 UFC 4-023-03 and in ASCE 41. Two exceptions are the modeling parameters and the acceptance criteria, which are now taken from ASCE 41, unless specifically modified in Chapters 4 to 8 of this UFC. One advantage of ASCE 41 is that guidance is provided for the development of analytical and numerical models for a number of distinct structural systems, including the determination of connection and member properties

One significant difference from ASCE 41 and the 2005 UFC 4-023-03 is the specification of a dynamic increase factor that is applied to the loads on the bays above the removed column or wall location to account for dynamic effects. In the 2005 UFC 4-023-03 and in the 2003 *GSA Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects* ("GSA Guidelines"), the load factor was set at 2, as for the Linear Static analysis, despite the explicit incorporation of nonlinear effects in the Nonlinear Static procedure. The dynamic increase factor is discussed in Section C-6.8.

### **C-6.6.3 Nonlinear Dynamic.**

The Nonlinear Dynamic procedure is essentially unchanged from the 2005 UFC 4-023-03, with the exception of the incorporation of the modeling parameters and acceptance criteria from ASCE 41.

### **C-6.7 Loads.**

The ASCE 7 extraordinary event load combination is employed, with the exception that the lateral load has been removed. In alternate path analyses, the initial and primary damage is limited to the column or removal location, with the rest of the structure being intact and providing the majority of its original lateral load resistance; the original lateral load resistance would be based on peak wind forces much higher than  $0.2W$ ,  $0.002\Sigma P$  or seismic loads if present. It is highly unlikely that the loss of a column or 2H wall section would destabilize the building laterally and cases where applying the lateral load uncovered any instability of the building and motivated any changes to the building design are unknown. Therefore, the lateral load requirement has been removed. /2/

### **C-6.8 Load and Dynamic Increase Factors.**

Three analytical procedures may be employed: Linear Static, Nonlinear Static, and Nonlinear Dynamic. As progressive collapse is a dynamic and nonlinear event, the applied load cases for the static procedures require the use of load increase factors or dynamic increase factors, which approximately account for inertial and nonlinear effects. For both Linear Static and Nonlinear Static, the 2005 UFC 4-023-03 and the GSA Guidelines use a load multiplier of 2.0, applied directly to the progressive collapse load combination.

Three issues with the use of a fixed factor of 2 have been identified. First, the same load multiplier is used for Linear Static and Nonlinear Static analyses, although the Nonlinear Static analysis incorporates nonlinearity. Second, an increase factor of 2.0 is not appropriate for the majority of LS and NS cases. The maximum dynamic displacement of an instantaneously applied and sustained load in a linear analysis is twice the displacement achieved when the load is applied statically. If a structure is designed to remain elastic, a factor of 2.0 would be appropriate. However, in extreme loading events, it is typical to design structures to respond in the nonlinear



range. Thus, the dynamic increase factor (DIF) that allows a Nonlinear Static solution to approximate a Nonlinear Dynamic solution, is typically less than 2. On the other hand, the load increase factor (LIF) for a Linear Static analysis must be greater than 2, since dynamic and nonlinear effects are present. Third, the load enhancement factor did not vary with the structural performance level, i.e., a structure is assigned a load enhancement factor of 2.0 regardless of whether the designer wants to allow significant structural damage or very little damage.

A study was undertaken to investigate the factors needed to better match the results of the LS and NS static procedures to the ND results; see McKay et al. 2008. As in ASCE 41, structural deformation was considered to be the best metric for approximating structural damage. To match the ND deformation levels, SAP2000 models of reinforced concrete and steel multi-story models were developed and analyzed with LS, NS, and ND procedures. For the LS and ND models, the loads were varied until agreement with the NS model was reached. The ASCE 7 extreme event load case was used for all analyses.

The range of nonlinear structural deformations used in this study was based primarily on the acceptance criteria in ASCE 41, with some modifications for reinforced concrete, for which the Life Safety values were increased by a factor of 3.5. For reinforced concrete, the allowable deformation criteria in ASCE 41 are much smaller than indicated by test data from blast- and impact-loaded RC structural members. In addition, the conservative ASCE 41 RC criteria are based on backbone curves derived from cyclic testing of members and joints, whereas only one-half cycle is applied in a progressive collapse event.

As an example, the results of this procedure for the Dynamic Increase Factor (DIF), used for Nonlinear Static analyses of steel structures, are shown in Figure C-7, where the DIF is shown as a function of the normalized rotation (allowable plastic rotation divided by the rotation at yield of the cross section). The data points in this plot were obtained by analyzing a range of buildings with various heights, bay dimensions and structural details. With this plot the DIF can be chosen as a function of the level of nonlinear behavior (i.e., structural performance level) that the designer wishes to employ or, else, the level of nonlinear behavior can be assigned, resulting in a specific DIF. In this UFC, the designer must find the smallest normalized rotation for any structural component or connection within the region of the structure affected by the column removal and will use this value to determine the DIF from the recommended equation in Figure C-7.

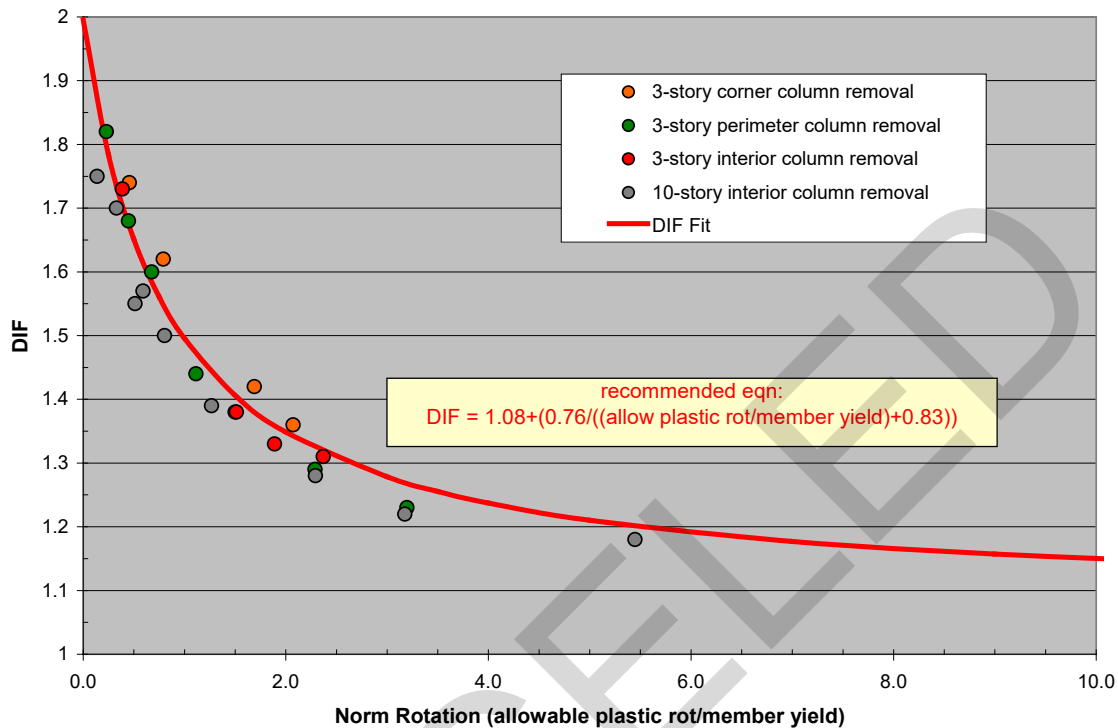


Figure C-7. Dynamic Increase Factor for Structural Steel

### C-6.9 Structural Damage Limits.

In the previous UFC, the structural damage limits were set at 15% and 30% for the floor area above the removed column or wall at an external or internal column or wall, respectively. In this UFC, no damage to the floor is allowed and these criteria have been removed, as the floor system, beams, and girders in the bays directly above the removed column can be designed to not fail, as is done for the bays in the floors above the removed column location.

### C-6.10 Modeling and Acceptance Criteria.

With a few notable exceptions, the acceptance criteria for linear and nonlinear approaches and the modeling criteria for nonlinear approaches from ASCE 41 are employed in the updated UFC 4-023-03. The ASCE 41 criteria are considered to be conservative when applied to progressive collapse design as they have been developed for repeated load cycles (i.e., backbone curves) whereas only one-half load cycle is applied in progressive collapse. As specified in each material specific chapter of this UFC, either the Collapse Prevention or Life Safety structural performance levels in ASCE 41 are used for many of the components; see Chapters 4 to 8.

The notable exceptions/modifications to the acceptance and modeling criteria include RC beams and slabs and a number of steel connections. These changes are motivated and justified by experimental data and numerical analysis results, as discussed later in this Appendix.

## **C-7      ENHANCED LOCAL RESISTANCE.**

The second direct design approach is Specific Local Resistance (SLR) or structural hardening, in which key or critical elements of the structure are designed for a specific load, such as blast or vehicle impact. This approach reduces the likelihood or extent of the initial damage and can be effective, for those cases where the threat can be quantified through risk analysis or specified through prescriptive design requirements. SLR can be a cost-effective method for providing resistance to collapse prevention, particularly for existing structures. The main shortcoming to this method is the requirement to define the threat or design load, as this information could be used to plan a deliberate attack on the structure; because of this, the threat information may be considered classified, restricting its use by the general public. Also, philosophically, progressive collapse design is typically considered to be threat-independent and if a specific threat such as an explosive device is specified, separate design guidance for hardening buildings is available.

In the 2005 UFC 4-023-03, a version of SLR was implemented in the Additional Ductility Requirement (ADR), which specified that the shear strength of a ground story column or wall exceed the flexural strength. With this requirement, the columns or walls will fail in flexure, which provides a more ductile and controlled response than the sudden failure associated with shear. This requirement can be implemented cost-effectively in new construction and provides a significant benefit.

A modified or enhanced version of SLR is implemented in this UFC to provide a nominal level of protection for perimeter columns and walls. This procedure is also threat independent and is referred to as Enhanced Local Resistance (ELR) to differentiate it from SLR and ADR. As ELR is intended to provide resistance against severe dynamic loadings, the same philosophy as is used for blast resistant design is employed, wherein all strength reduction factors are set to 1.0 and the expected strength is employed for shear, flexure and all other actions. The development and design approach of this method is “tuned” to the inherent structural robustness of the system as discussed in Marchand and Stevens 2008. A charge weight that will destroy multiple columns or wall sections, depending upon standoff and location, was determined for a variety of representative structures. An analysis of the data resulted in required enhancement factors of 2 and 1.5 for the flexural and shear demands of columns and walls, respectively. The design shear strength of the column or wall and the connections to the lateral force resisting elements is also stipulated to be greater than the shear demand, which is based on the flexural demand, to provide a ductile and more controlled failure mode. \2\

### C-7.1 ELR Design Requirements.

For RC II Option 1, the shear demand of the columns and walls is determined based on the existing or as-designed structure. For RC III, the shear demand is based on the structure design after the AP method has been applied.

A higher level of resistance is required for RC IV structures; in this case, two flexural demands are compared and used to determine the final shear demand. Because an RC IV building in a seismic region will have significantly larger columns than the same RC IV building in a non-seismic region, the seismic structure will also have significantly higher shear demand. However, both buildings should provide the same protection to the occupants. To accomplish this and to provide a reasonable shear demand, the design of the building based on only gravity loads and the AP requirements is used to define the baseline nominal flexural strength. If a gravity loads only design was not accomplished during the design effort, it must be performed. It is anticipated that mathematical models will be created for RC IV buildings and the effort to use them for a gravity loads only design and AP analysis should not be significant. After the baseline nominal flexural strength is determined with the column design from the gravity loads-only AP structural design, it is multiplied by 2.0 and compared to the existing nominal flexural strength. The existing nominal flexural strength is based on the column design after the Alternate Path method is applied to the final structure (i.e., the structural design based on wind, seismic, snow, and gravity loads). The larger of 2.0 times the baseline nominal flexural strength and the existing nominal flexural strength is defined as the flexural demand. If the flexural demand is greater than the existing nominal flexural strength, then the design of the column must be upgraded to match the flexural demand. The same procedure applies for load-bearing walls but in this case, the baseline flexural strength is multiplied by 1.5.

### C-7.2 Connection Design for Rebound.

Rebound is most likely to occur for explosive loadings, not vehicle impact and the magnitude of the rebound reaction is a function of the column properties, end conditions, façade design, charge weight and standoff, etc. Guidance for rebound forces can be found in Section 4-1.2.5 of PDC TR-06-01 *Methodology Manual for the Single-Degree-of-Freedom Blast Effects Design Spreadsheets (SBEDS)*, which states that “An equivalent static design load for rebound equal to one-half the inbound ultimate resistance is often used as a rule of thumb for blast design.” This is the basis for the requirement in Section 3-3.6 that the connection be designed for a rebound reaction equal to 50% of the inbound reaction force. It should be noted that this requirement is based on analytical modeling, engineering judgment, and qualitative observations. However, there is limited experimental evidence that neither supports nor discredits this requirement for the wide range of construction types and scenarios covered in this UFC.

/2/

## **C-8 REINFORCED CONCRETE.**

### **C-8.1 Reinforced Concrete Beams and Joints.**

4/4/ The design strength and rotational capacities of the beams and beam-to-column-to-beam joints shall be determined with the guidance found in ASCE 41, as modified with the acceptance criteria provided in Paragraph 4-4.3.

### **C-8.2 Structural Performance Levels.**

To determine the appropriate structural performance level and the corresponding modeling and acceptance criteria from ASCE 41, the types of loading and resulting performance of reinforced concrete beams and beam-column joints were assessed. In seismic events, the structural elements and connections are subjected to primarily shear and bending with little axial tension. Under progressive collapse conditions, axial tension is developed in the beam, as they undergo large displacements in a double-span condition and the beam starts to behave as a catenary.

Conceptually, the damage states reflected in the Life Safety category make sense for progressive collapse. In seismic design, it is assumed that the primary components can sustain significant degradation to their lateral load resisting behavior as long as they can still support the gravity loads; for progressive collapse, the primary components must resist gravity loads during and after the event. As defined in ASCE 41, Collapse Prevention results in a damage state for which there is little additional deformation capacity and the stability of the system has been severely compromised. Life Safety provides a greater reserve in terms of nonlinear deformation and strength and thus is used for the majority of the steel acceptance criteria.

### **C-8.3 Modeling and Acceptance Criteria for Reinforced Concrete.**

The majority of the modeling parameters, acceptance criteria and linear m-factors for reinforced concrete are chosen as the Life Safety values in Chapter 6 of ASCE 41 for primary and secondary components. Modifications to the modeling and acceptance criteria for beams and slabs were made based on data from blast- and impact-loaded beams and other flexural members. For RC beams and slabs controlled by flexure, the modeling and acceptance criteria values for Collapse Prevention were multiplied by a factor of 2.5 for primary members and 2.0 for secondary members. For all other conditions and cases of beams and slabs, the Life Safety values are used.

### **C-8.4 Best Practice Recommendation.**

To insure ductile and energy absorbing response in new construction of reinforced concrete structures, it is recommended that the primary reinforced concrete beams and beam-to-column-to-beam joints comply with the provisions for special moment frames in ACI 318. These code provisions include ductile detailing requirements for longitudinal reinforcement, transverse reinforcement, required shear strength, and development length of bars in tension.

## **C-9 STRUCTURAL STEEL.**

### **C-9.1 Structural Steel Connections.**

A variety of steel frame connection types are listed in Table C-1 and illustrated in Figures C-8 through C-10. This list constitutes an inventory of connection types that have been used either in the past and/or present for standard building code design applications (gravity, wind, and earthquake loads).

Proprietary connections have been evaluated and found to be acceptable for specific projects and/or for general application. Inclusion of these connections in this UFC does not constitute an endorsement. The Kaiser Bolted Bracket®, SidePlate® and SlottedWeb™ are shown schematically in Figures C-11 through C-13, respectively. Details of the performance and geometry can be obtained from the vendors.

### **C-9.2 Steel Connection Requirements.**

4/4/ The design strength and rotational capacities of the beams and beam-to-column connections shall be determined with the guidance found in ASCE 41, as modified with the acceptance criteria provided in Paragraph 5-4.3 in this UFC.

### **C-9.3 Structural Performance Levels.**

To determine the appropriate structural performance level and the corresponding modeling and acceptance criteria from ASCE 41, the types of loading and resulting performance of structural steel connections were assessed as there is a fundamental difference between seismic and progressive collapse events. The seismic modeling and acceptance criteria in ASCE 41 are based upon cyclic tests in which the end of a cantilever beam is subjected to ever-increasing amplitudes; the beams and connections experience shear and bending moment with no axial tension. Under progressive collapse conditions, axial tension is developed as the beam experiences large displacements in a double-span condition and the beam starts to behave more like a cable than a beam.

**Table C-1. Steel Frame Beam-to-Column Connection Types**

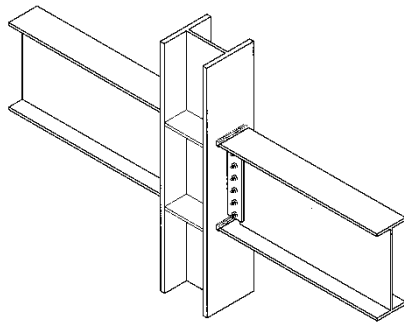
Connection	Description	Type	Figure
Welded Unreinforced Flange (WUF)	Full-penetration welds between beams and columns, flanges, bolted or welded web, designed prior to code changes following the Northridge earthquake.	FR	C-8(a)
Welded Flange Plates (WFP)	Flange plate with full-penetration weld at column and fillet welded to beam flange	FR	C-8(b)
Welded Cover-Plated Flanges	Beam flange and cover-plate are welded to column flange	FR	C-8(c)
Bolted Flange Plates (BFP)	Flange plate with full-penetration weld at column and field bolted to beam flange	FR or PR	C-8(d)
Improved WUF-Bolted Web <sup>(1)</sup>	Full-penetration welds between beam and column flanges, bolted web, developed after Northridge Earthquake	FR	C-8(a)
Improved WUF-Welded Web	Full-penetration welds between beam and column flanges, welded web developed after Northridge Earthquake	FR	C-8(a)
Free Flange	Web is coped at ends of beam to separate flanges, welded web tab resists shear and bending moment due to eccentricity due to coped web developed after Northridge Earthquake	FR	C-8(e)
Welded Top and Bottom Haunches	Haunched connection at top and bottom flanges developed after Northridge Earthquake	FR	C-8(f)
Reduced Beam Section (RBS) <sup>(2)</sup>	Connection in which net area of beam flange is reduced to force plastic hinging away from column face developed after Northridge Earthquake	FR	C 8(g)
Top and Bottom Clip Angles	Clip angle bolted or riveted to beam flange and column flange	PR	C-9(a)
Bolted Double Split Tee <sup>(2)</sup>	Split tees bolted or riveted to beam flange and column flange	PR	C-9(b)
Composite Top and Clip Angle Bottom	Clip angle bolted or riveted to column flange and beam bottom flange with composite slab	PR	C-9(a) similar
Bolted Flange Plates	Flange plate with full-penetration weld at column and bolted to beam flange	PR	C-8(d)
Bolted End Plate	Stiffened or unstiffened end plate welded to beam and bolted to column flange	PR	C-8(c)
Shear Tab Connection with or without <sup>(2)</sup> floor deck	Simple gravity connection with shear tab, may have composite floor deck	PR	C-8(d)
Kaiser Bolted Bracket <sup>®</sup> ,	SMF moment connection with fastened cast steel haunch brackets that are bolted to the column flange and either fillet-welded or bolted to both beam flanges.	FR	C-11
SidePlate <sup>®</sup>	SMF moment connection with full-depth side plates and fillet welds, developed following the 1994 Northridge earthquake.	FR	C-12
SlottedWeb <sup>™</sup>	SMF moment connection similar to WUF with extended web slots at weld access holes to separating the beam flanges from the beam web in the region of the connection.	FR	C13

Note: PR = Partially Restrained Moment Connection or Shear Connection

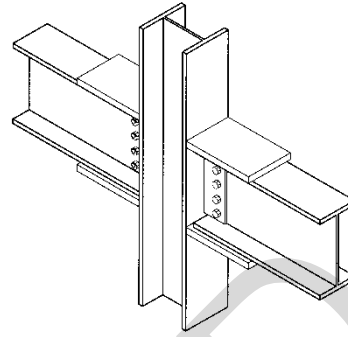
FR = Fully Restrained Moment Connection

<sup>(1)</sup> Testing and predictive analysis information is provided in Karns and Houghton 2008.

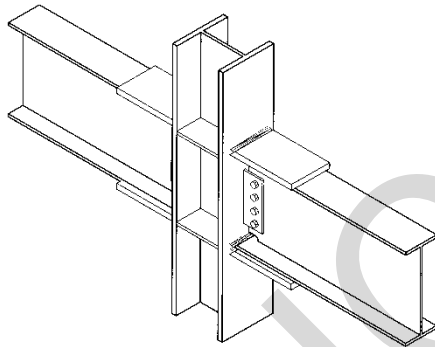
<sup>(2)</sup> Predictive Analysis only information is provided in Karns and Houghton 2008.



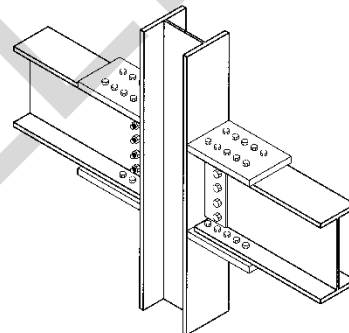
(a) WUF Connection



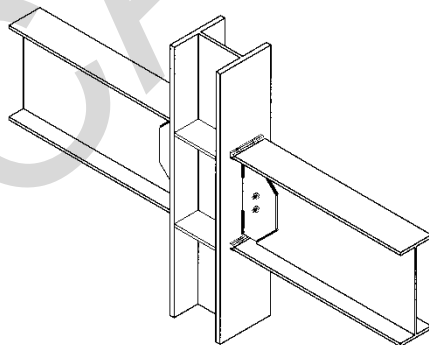
(b) Welded Flange Plate



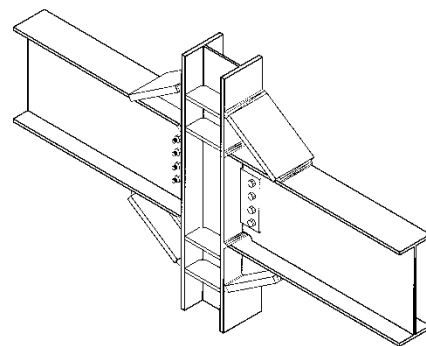
(c) Welded Cover Plated Flanges



(d) Bolted Flange Plate



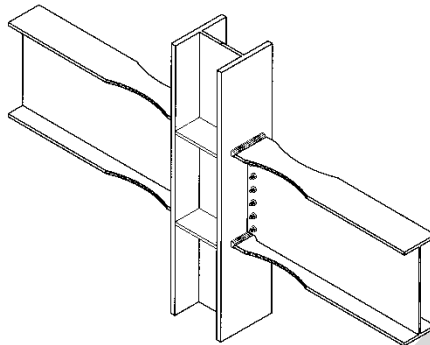
(e) Free Flange



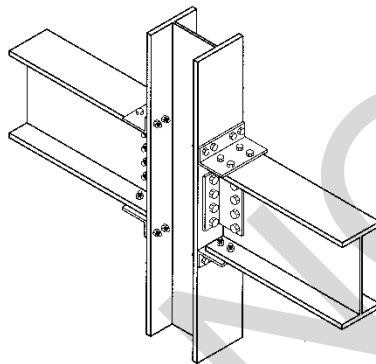
(f) Top and Bottom Haunch

**Figure C-8. Fully Restrained Moment Connections**

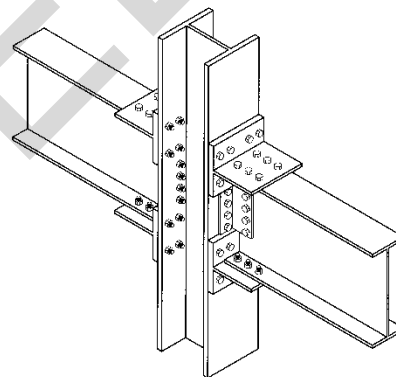




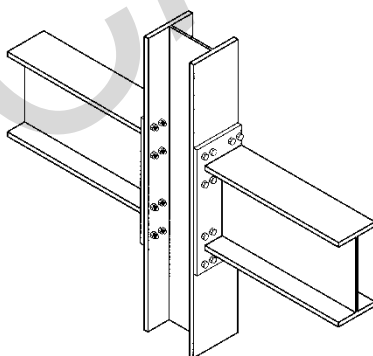
**Figure C-8 (continued) Fully Restrained Moment Connections**



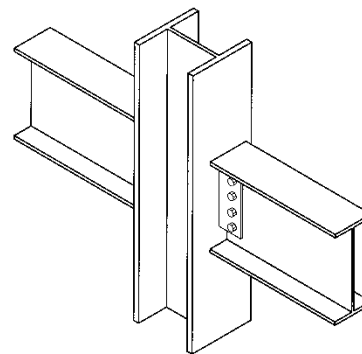
(a) Bolted or Riveted Angle



(b) Bolted Double Split Tee

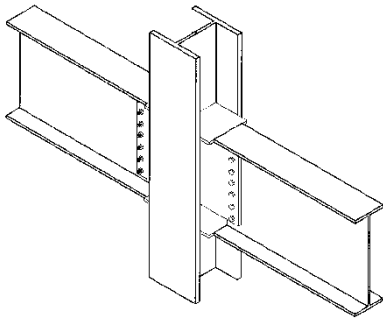


(c) End Plate (Unstiffened)

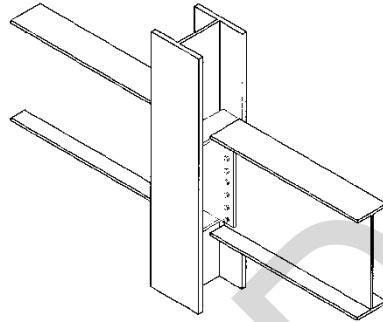


(d) Simple Shear Tab Connection

**Figure C-9. Partially Restrained Moment Connections or Shear Connections**

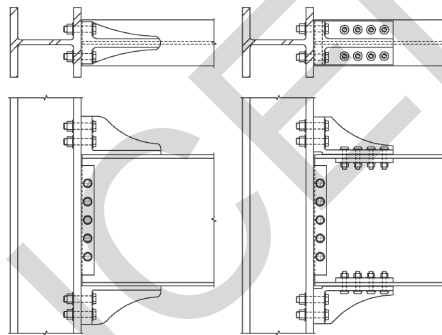


(a) Fully Restrained Connection

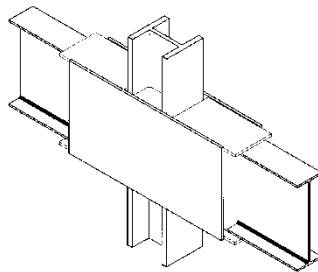


(b) Typical Shear Only Connection

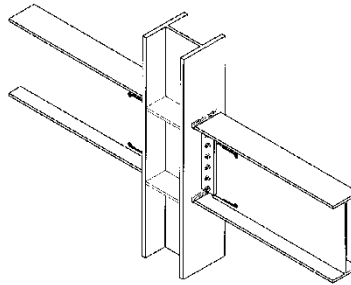
**Figure C-10. Weak Axis Moment Connection or Shear Connection**



**Figure C-11. Kaiser Bolted Bracket® Fully Restrained Connection**



**Figure C-12. SidePlate® Fully Restrained Moment Connection**



**Figure C-13. SlottedWeb™ Fully Restrained Connection**

The rotational capacity values for connections in the 2005 UFC 4-023-03 were based upon the 2003 GSA Guidelines, and reasonably agree with those in ASCE 41 for primary elements and the Life Safety structural response level. Conceptually, the damage states reflected in the Life Safety category make sense for progressive collapse. The governing assumption for seismic design is that the primary components can sustain significant degradation to their lateral load resisting behavior as long as reserve capacity is maintained to support the gravity loads. For progressive collapse, the primary components must resist gravity loads during and after the event. Since gravity loads are constant, damage can result in instabilities and failure. Per the definitions of structural performance levels in ASCE 41, Collapse Prevention results in a damage state for which there is little additional deformation capacity and the stability of the system has been severely compromised. Life Safety provides a greater reserve in terms of nonlinear deformation and strength and thus is used for the majority of the steel acceptance criteria.

#### **C-9.4 Modeling Parameters and Acceptance Criteria.**

In developing the modeling procedures and acceptance criteria, a comparison was made between the deformation limits contained in ASCE 41, the Eurocode, and the 2005 UFC 4-023-03. These limits were also compared to the rotational capacities reported in the *GSA Steel Frame Bomb Blast & Progressive Collapse Test Program Report (2004-2007)* ("GSA Test Program Report") as summarized in Karns and Houghton 2008. The progressive collapse test configurations in the GSA Test Program were designed to capture both bending and axial tension to determine the effect of their interaction on the rotational capacity of the connection investigated.

Models and acceptance criteria in ASCE 41 are based upon cyclic loadings with bending moment only and rotational capacities are often limited because of

degradation and premature loss of strength due to low cycle fatigue. In contrast, recent progressive collapse research with monotonic loading conditions has demonstrated that rotational capacities are most often higher than for cyclic loading. However, with the addition of axial loads, the progressive collapse rotational capacities may be limited, as some connections are unable to develop significant axial tension load upon reaching the

ultimate moment capacity of the beam. Thus, the majority of the modeling and acceptance criteria in Chapter 5 are specified either as Life Safety justified by the behavioral differences associated with the effects of loading (monotonic vs. cyclic) and the ultimate state of strain (moment only vs. moment-axial tension interaction).

Where appropriate, some modifications to the modeling parameters and acceptance criteria have been applied. In some cases, little or no criteria were available and new acceptance criteria were created, using the existing literature and recent tests and numerical simulations, as detailed in Karns and Houghton 2008. The results were used to determine the modeling and acceptance criteria provided in Tables 5-1 and 5-2 of this document.

#### **C-9.5 Best Practice Recommendation.**

For new construction, it is recommended that all primary steel frame beam-to-column moment connections be one of the special moment frame (SMF) connections identified in FEMA 350 under Section 3.5 (welded), Section 3.6 (bolted) or Section 3.8 (proprietary), and/or ANSI/AISC 358 (including Supplements), and/or prequalified under ICC-ES AC129. The use of an SMF connection type should not be construed to include the SMF seismic detailing provisions specified in national building codes for higher seismic regions, except for the case where a particular building design is subject to those code provisions.

The additional cost for SMF connections should be minimal, as the use of notch-tough weld wire, continuity plates, and high strength bolts, etc, is common practice. The primary reason for using an SMF connection is to secure the connection characteristics that provide a minimum threshold of rotational capacity. It is important to note that the “seismic detailing” provisions of the IBC Building Code are not required for progressive collapse design applications, unless the seismic region for a particular building design is subject to those earthquake code provisions anyway.

Acceptable SMF-type connections include:

- Welded Unreinforced Flanges with Welded Web (WUF-W)
- Bolted Flange Plate (BFP)
- Bolted Unstiffened End Plate (BUEP)
- Bolted Stiffened End Plate (BSEP)
- Reduced Beam section (RBS)
- Kaiser Bolted Bracket®
- SidePlate®

- Slotted Web™

Two common connections that do not meet the SMF requirements are:

- Double Split Tee (DST)
- Welded Unreinforced Flanges with Bolted Web (WUF-B).

For the WUF-B connection, welding of its bolted web-to-shear tab connection is all that is required for it to become a WUF-W connection, for which there is a significant improvement in rotational performance, including increased reliability.

## **C-10 MASONRY, WOOD, AND COLD-FORMED STEEL.**

As discussed for steel and reinforced concrete, the modeling parameters, nonlinear acceptance criteria and linear m-factors for the Life Safety performance level in ASCE 41 are appropriate for Alternate Path analysis and design of masonry, wood, and cold-formed steel structures.

### **C-10.1 Time Effect Factor $\lambda$ for Wood.**

Note that for wood construction, the time effect factor  $\lambda$  must be included in the determination of strength for the Tie Force and Alternate Path requirements. As discussed in AFPA/AWC "LRFD Manual for Engineering Wood Construction", the time effect factors,  $\lambda$ , were derived based on reliability analysis that considered variability in strength properties, stochastic load process modeling and cumulative damage effects. The time effect factors are applied to the reference strengths used in the code, which are based on short-term loading test values. Time effect factors range in value from 1.25 for a load combination controlled by impact loading to 0.6 for a load combination controlled by permanent dead load. Common building applications will likely be designed for time effect factors of 0.80 for gravity load design and 1.0 for lateral load design. Further ANSI/ASCE 16-95 indicates time effect factors of 0.7 when the live load in the basic gravity load design combination is for storage, 0.8 when the live load is from occupancy, and 1.25 when the live load is from impact. It is desirable that the structure is stable following local damage to allow for rescue operations and the installation of temporary shoring, however stability in the damaged state is not a permanent condition. Therefore a time effect factor greater than that associated with permanent occupancy and less than that associated with impact is warranted. For this reason and to avoid overly conservative values for such an extreme loading, a time effect factor of 1.0, consistent with the time effect factors used for gravity-lateral load combinations, is specified.

\3\

**C-11 RECOMMENDED ADDITIONAL SPECIFICATION PARAGRAPHS.**

**C-11.1 Recommendation 1. Work Experience.**

The resume of the licensed Professional Engineer (P.E.) with a minimum of 3 years of experience with the Alternate Path Analysis per UFC 4-023-03 shall be submitted.

Reference projects.

Provide the list of projects involving progressive collapse design. Each reference project should include the design method used such as Tie Force Method or Alternate Path Method. For Alternate Path Method, specify the design procedure such as linear static, non-linear static or non-linear dynamic.

**C-11.2 Resume for ATPF and Progressive Collapse Design Experience.**

Specify the engineer(s) responsible for the progressive collapse design and/or the ATPF, blast design analysis for the project.

/3/

## APPENDIX D REINFORCED CONCRETE EXAMPLE

### D-1 INTRODUCTION.

A typical reinforced concrete frame commercial building design and analysis example has been prepared to illustrate tie force calculations. The structure is assumed to have an occupancy less than 500 people and is classified as \3\ Risk Category /3/ II per UFC 3-301-01.

The example has been prepared using tools and techniques commonly applied by structural engineering firms in the US. Computer software that is typical of that used for structural design was employed for preliminary design. Per the option given in the UFC, as specified in Section 2-2, tie forces and enhanced local resistance are applied to provide resistance to progressive collapse.

### D-2 BASELINE PRELIMINARY DESIGN.

The structure is a seven-story concrete moment frame. The intended function of the building is office use, with occupancy of less than 500 people. See Figures D-1 and D-2 for drawings of the building. The preliminary design, shown in Figures D-1 and D-2, and described below, has been sized to meet the requirements of IBC2006.

#### D-2.1 Modeling Assumptions.

Systems:

Gravity:

Floor system: Pan formed beams

Vertical support: Columns

Lateral:

Moment frames

Foundation:

Shallow spread footings

Elevation:

Foundation to L1: 16'-0"

Typical: 13'-0"

Roof: 14'-0"

Parapet: 4'-0"

Plan:

E-W dimension: 227'-0"

N-W dimension: 97'-0"

Concrete:

All concrete shall be normal weight concrete and shall have specified 28-day compressive strength as shown below:

f'c columns: 5000 psi  
f'c floor: 4000 psi  
f'c foundation: 4000 psi

Rebar:

A615 grade 60 ksi.

Details of pan formed beams and slab:

Slab thickness = 5" (fire rating)

Per ACI table 9.5 (a) for beams spanning 37.5' (clear span approximately 34.5') the required depth  $\sim 34.5 \times 12 / 18.5 = 22.3"$ . 20" deep beams with 5" slab (overall depth of 25") are provided. Rib width of 6" and spacing of 6'-0" utilizes pan form system that is very common.

Pans have 1" to 12" side slope.

Therefore Eq. thickness =  $(6" + 9.33") / 2 \times 20" / (72") = 2.13" + 5" \text{ slab} = 7.13"$

Therefore Eq. uniform weight = 89 psf

Details of girders:

Width of the beam: 36"

Depth of beam: 25"

Since the slab and pan formed beam weight is considered for the entire floor area, there is common area of concrete between girders and pan formed beams.

Eq. beam depth =  $25 - 7.13" = 17.87"$

There are 7 girders over the width of 227' in N-S direction and therefore eq. weight =  $(36 \times 17.87) / 144 \times 150 \times 7 / 227 = 20.66 \text{ psf}$

There are 2 girders over the width of 97' in E-W direction and therefore eq. weight =  $(36 \times 17.87) / 144 \times 150 \times 2 / 97 = 13.82 \text{ psf}$

Therefore total weight of beams other than pan formed beams =  $20.66 + 13.82 = 34.48 \text{ psf}$ . Say 35 psf

Details of columns:

Typical interior column:

$P_u = 1.2 (89 + 35 + 10) + 1.6 (50 + 10 + 20) = 288 \text{ psf}$

$P_u = 0.288 \times 37.5' \times 37.5' / 2 \times 7 = 1421 \text{ kips}$ .

Axial stress =  $1421 / (24 \times 24) = 2.46 \text{ ksi}$

For f'c of 5 ksi use 24' square column.

There are 28 columns at each floor.

Weight of column at typical floor =  $(24" \times 24") / 144 \times 13' \times 28 \times 150 / (97' \times 227') = 9.91 \text{ psf}$  say 10 psf.



## D-2.2 Loading Assumptions.

Dead loads (equivalent uniform loads) (D):

Self weight:

Slab and pan formed beams → 89 psf

Beams → 35 psf

Columns → 10 psf

Super imposed dead load (SDL):

Ceiling, MEP → 10 psf

Roofing → 20 psf

Cladding (CL) → 60 psf (wall area)

Live loads:

Office floor area (LL) → 50 psf + 20 psf allowance for partitions

Storage/Mechanical floor area (LL) → 125 psf

Corridors → (LL) 80 psf

Roof (Lr) → 20 psf

Wind Load (W) was determined per IBC 2006 using 110 mph with exposure = B and importance factor = 1.0

Earthquake Load (E) is assumed not to control the design because the building is in a non-seismic region.

Other Loads: Snow Loads (S), Rain Loads (R) are assumed to not control the design.

## D-2.3 Design Information.

Column reinforcement at first level above grade:

Corner columns: 8-#8, 3 each face

Long side columns: 14-#11, 4X-5Y

Short side columns: 8-#8, 3 each face

Interior columns: 12-#10, 4 each face

Slab reinforcement at first level above grade:

Each direction: #3 at 12"

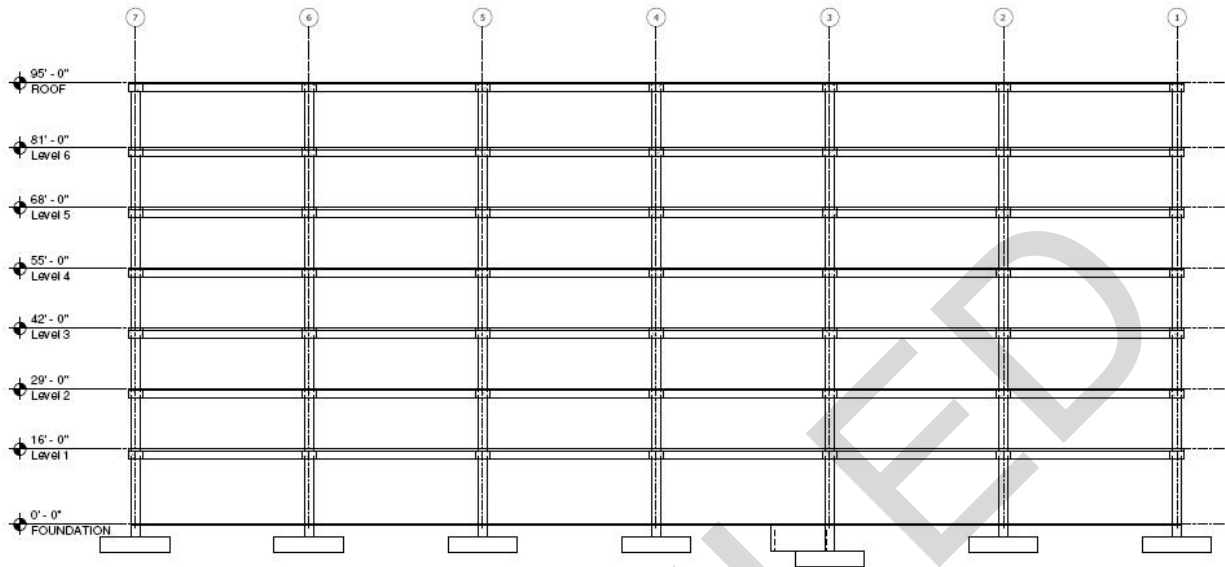


Figure D-1. Concrete Building Elevation

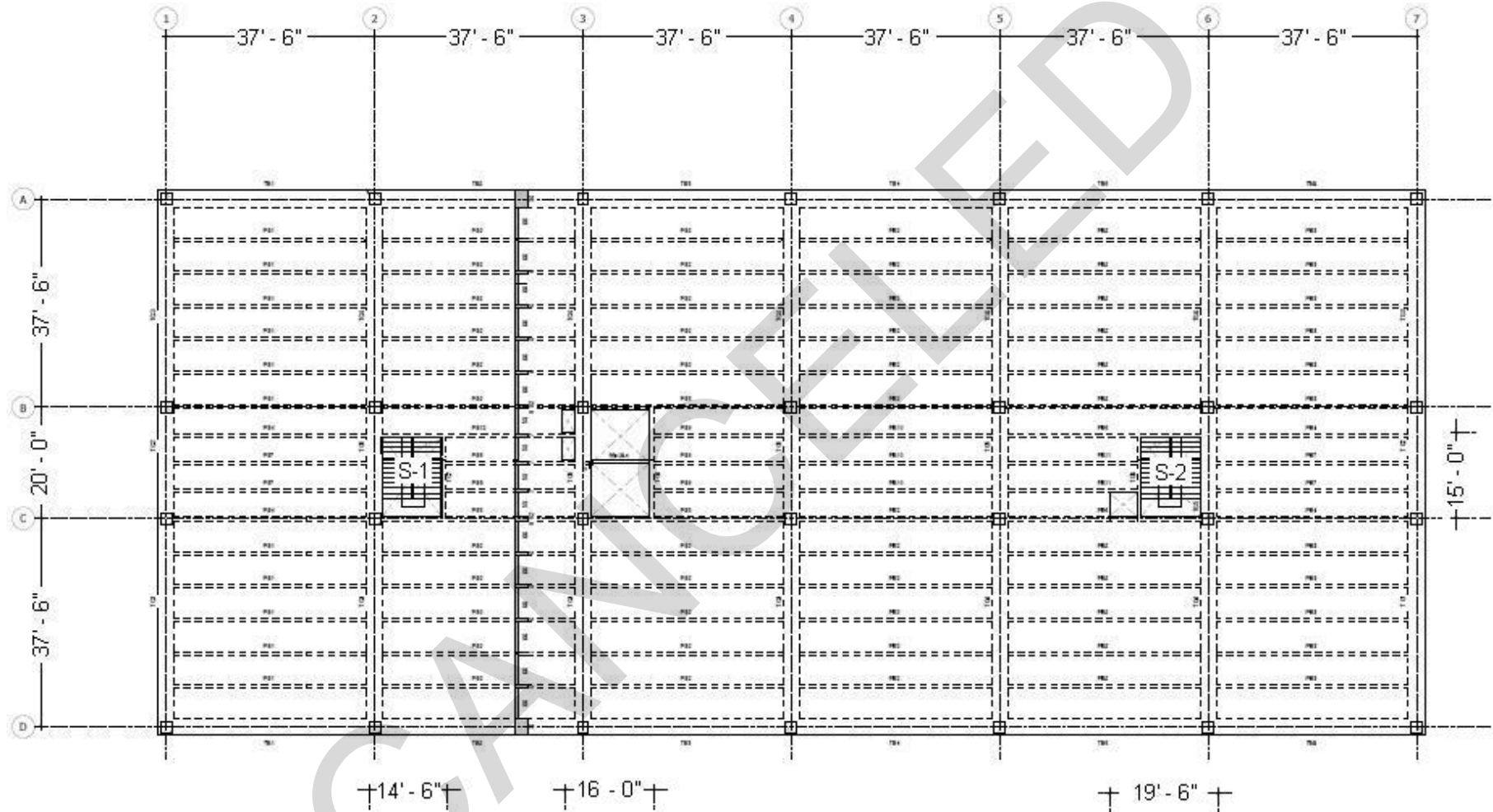


Figure D-2. Concrete Building Plan

### D-3 TIE FORCE DESIGN.

The procedure and requirements for Tie Forces for framed buildings are presented in Section 3-1. In the Tie Force approach, the building is mechanically tied together, enhancing continuity, ductility, and development of alternate load paths. There are three horizontal ties that must be provided: longitudinal, transverse, and peripheral. Vertical ties are required in columns. Figure 3-1 illustrates these ties for frame construction.

#### D-3.1 Calculating $w_F$ .

Section 3-1.3 presents Equation 3-2 for determination of the floor load used in tie force calculations. The corridor load is applied over 25% of bay B-C and storage/mechanical load is provided over the 75% of bay B-C over the length of the building. Office live load is applied over the remainder of the floor. Based on these loads in combination with those presented in Section D-2.2, the effective  $w_F$  for transverse and longitudinal ties is 214.5-psf (from Section 3-1.3.2.2, with the difference between the minimum and maximum floor load in the bays on the floor plan being less than or equal to 25% of the minimum floor load and with the area associated with the maximum floor load less than or equal to 25% of the total floor plan area). The calculation is shown in Table D-1.

**Table D-1. Load Calculations for  $w_F$**

Dead, psf	Live, psf			w <sub>F</sub> (1.2D+0.5L), psf		
Slab and pan formed beams, beams, columns, ceiling, MEP	Office	Storage/ Mech	Corr.	Office	Storage/ Mech	Corr.
144	70	125	80	207.8	235.3	212.8
				Difference = 235.3-207.8 = 27.5 psf, less than 25% of smallest w <sub>F</sub>		
Areas, sf	Office	Storage/ Mech	Corridor	Total		
	16875	3375	1125	21375		
	Storage/Mech area is less than 25% of total area					
Since max difference in w <sub>F</sub> is less than 25% of smallest w <sub>F</sub> and area of largest load is less than 25% of total area, use average w <sub>F</sub> for the entire floor						
w <sub>F</sub> , total load over total area, psf	= (16875 * 207.8 + 3375 * 235.3 + 1125 * 212.8)/(21375) = 214.5-psf					

The cladding load  $W_C$  is calculated as:

$$W_C = 1.2 * 60\text{-psf} * 13\text{-ft height} * 37.5\text{-ft span} = 35.1\text{-kip}$$

For vertical ties on the perimeter, the cladding load is averaged over the bays in which it is present and added to the floor distributed loading, to determine the effective  $w_F$ . For example, for column A1:

$$\begin{aligned}
 w_F &= \text{Floor distributed load} + \text{Cladding load/area} \\
 &= 214.5\text{-psf} + 1.2 (18.75\text{-ft} + 18.75\text{-ft}) (13\text{-ft}) (60\text{-psf}) / (18.75\text{-ft})^2 \\
 &= 314.3\text{-psf}
 \end{aligned}$$

### D-3.2 Tie Force Calculations.

The required tie forces and rebar area can be calculated with the loads from the previous section and the geometric properties of the building. As an example, the peripheral tie force in the transverse (N-S) direction is determined with Equation 3-6:

$$\begin{aligned} F_p &= 6 w_F L_1 L_p + 3 W_C \\ &= 6 (214.5\text{-psf}) 37.5\text{-ft} (3\text{-ft}) + 3 (35.1\text{-kip}) \\ &= 250.1\text{-kips} \end{aligned}$$

The required area is determined with Equation 3-1:

$$\begin{aligned} \Phi R_n &\geq \Sigma \gamma_i Q_i \\ 0.75 (1.25 \cdot 60\text{-ksi}) A_{s \text{ req'd}} &\geq 250.1\text{-kips} \\ A_{s \text{ req'd}} &= 4.45\text{-in}^2 \end{aligned}$$

where 0.75 is the LRFD strength reduction factor per Section 4-3 of this document and 1.25 is the over-strength factor for rebar, per ASCE 41.

The data for tie force calculations and the resulting reinforcement are summarized in Table D-2.

**Table D-2. Tie Force Calculations**

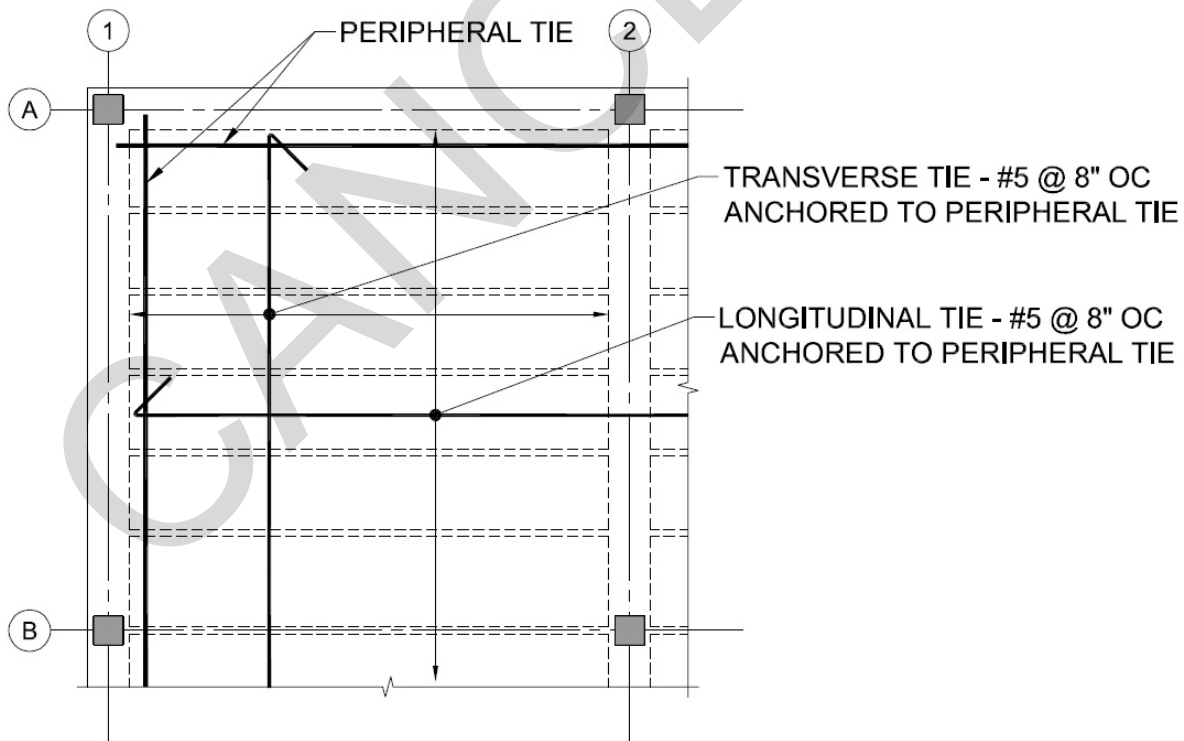
Tie Type	Location	Length (ft)	$w_F$ (psf)	$W_C$ (kips)	F (kips)	$A_{s \text{ req'd}}$ (sq in) <sup>A</sup>	Reinfm't
Peripheral	Transverse	37.5	214.5	35.1	250.1	4.45	8 - #7
Peripheral	Longitudinal	37.5	214.5	35.1	250.1	4.45	8 - #7
Peripheral	Stair 1 (S1) Transverse	15	214.5	0	57.9	1.03	6 - #4
Peripheral	Stair 1 (S1) Longitudinal	14.5	214.5	0	56.0	1.00	5 - #4
Peripheral	Stair 2 (S2) Transverse	15	214.5	0	57.9	1.03	6 - #4
Peripheral	Stair 2 (S2) Longitudinal	19.5	214.5	0	74.3	1.32	7 - #4
Peripheral	Elevator Transverse	21	214.5	0	81.08	1.44	8 - #4
Peripheral	Elevator Longitudinal	16	214.5	0	61.8	1.10	6 - #4
Tie Type	Location	Length (ft)	$w_F$ (psf)		F (kips/ft)	$A_{s \text{ req'd}}$ (sq in/ ft)	Reinfm't
Transverse	Distributed	37.5	214.5		24.13	0.429	#5 @ 8" O.C. <sup>B</sup>
Longitudinal	Distributed	37.5	214.5		24.13	0.429	#5 @ 8" O.C. <sup>B</sup>
Tie Type	Location	Area (sq ft)	$w_F$ (psf)		F (kips)	$A_{s \text{ req'd}}$ (sq in)	Reinfm't
Vertical	A1	351.6	314.3		110.5	1.96	No Additional
Vertical	A2	703.2	264.4		185.9	3.31	No Additional
Vertical	B1	539.1	264.4		142.5	2.53	No Additional
Vertical	B4	1078.2	214.5		231.3	4.11	No Additional

<sup>A</sup> The over-strength factor is 1.25, for rebar, per ASCE 41 and  $\Phi$  is 0.75, per Section 4-3 of this UFC.

<sup>B</sup> Replace #3 at 12" O.C slab reinforcement with #5 at 8" O.C.

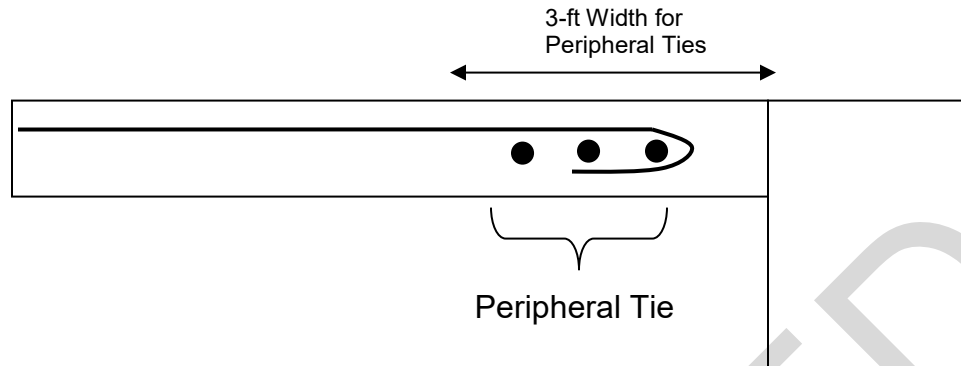
Reinforcement already present from the baseline design may be used to satisfy the tie force requirements in Table D-1 provided it is not within or directly above flexural members. Splices, development lengths and connections of reinforcement must be per Section 3-1.6. Note that the required tie forces at each of the stairs are different. This difference is to account for the additional MEP opening located immediately adjacent to Stair 2. In the case of Stair 2 and the adjacent MEP opening, these openings are separated by a flexural member. Since tie reinforcement cannot be located directly above a flexural member unless it can be shown to meet Section 3-1 rotation requirements, these openings were combined for the purposes of tie force calculation and placement of opening peripheral ties. A similar concept was used for the elevator and adjacent MEP openings. Note the dimension extents shown in Figure D-2.

Also note that the reinforcement shown in Table D-1 for internal longitudinal and transverse ties is to replace the preliminary #3 bars at 12". A diagram of tie layout for an exterior bay is shown in Figure D-3. The internal ties may be connected to peripheral ties as shown in Figure D-4. Note that only the portion of the internal tie steel needed for the baseline design (the #3 bars at 12") must extend to the beam top steel; the remainder of the internal ties may be directly connected to the peripheral tie (i.e., the supplemental hook is not needed for the internal ties that are directly connected to the peripheral tie).



**Figure D-3. Typical Layout of Internal Ties**





**Figure D-4. Typical Anchorage of Internal Ties to Peripheral Ties**

### D-3.3 Enhanced Local Resistance.

ELR provisions for Risk Category II require that corner and penultimate perimeter columns at the first floor above grade achieve design shear strengths that exceed the shear demand associated with the flexural demands of the columns. For Risk Category II Option 1, the flexural demand is determined from the building design after it meets the TF requirements. For the purposes of ELR evaluation, the columns are considered fixed at the first level above grade due to continuity of the column above the first level and the rigid support provided by the floor diaphragm and pinned at the base. The flexural demand is based on the development of a three hinged mechanism in the column, with one hinge present as the pinned end of the column, a plastic hinge that develops at the fixed end at the first level above grade and a plastic hinge that develops within the column.

The required shear strength (i.e., the shear demand) can be found using methods in plastic structural design references or in Table 4-4 of PDC TR-06-01 *Methodology Manual for the Single-Degree-of-Freedom Blast Effects Design Spreadsheets (SBEDS)*. From PDC TR-06-01, the largest reaction/shear for a pinned-fixed, uniformly loaded beam is  $5 r_u L/8$ , where  $r_u$  is the ultimate resistance and equal to  $12 M_p/L^2$ .  $M_p$  is the plastic moment of the column cross-section and thus is the nominal flexural strength of the penultimate and corner columns, which incorporate vertical tie forces. Thus,

$$\begin{aligned} V_u &= 5 r_u L/8 \\ r_u &= 12 M_n/L^2 \end{aligned}$$

and re-arranging yields

$$V_u = 7.5 M_n / L \quad \text{Equation (D-1)}$$

where  $V_u$  = Shear demand (maximum shear and reaction in the beam)  
 $M_n$  = Nominal flexural strength, accounting for axial load

$r_u$  = Ultimate resistance,  
 $L$  = Column height

The corner column is considered in this example. The axial load on the corner column is based on the loads given in Section 3-2.2, bay sizes of 37.5-ft by 37.5-ft and the 7-story height.

$$\begin{aligned} P_{\text{axial}} &= 7 [1.2D + 0.5L] 37.5\text{-ft } 37.5\text{-ft}/4 \\ &= 7 [1.2(89 + 35 + 10) + 0.5 (50 + 10 + 20)] 37.5\text{-ft } 37.5\text{-ft}/4 \\ &= 494\text{-kip} \end{aligned}$$

With over-strength factors of 1.5 for concrete and 1.25 for steel (from ASCE 41), the concrete and steel strengths are 7500-psi and 75-ksi, respectively. For a 24-in x 24-in column with 8 #8s (3 each face) and an axial load of 494-kip, the interaction equation shows that  $M_n = 9,400\text{-in-kip} = 783\text{-ft-kip}$ .

$$\begin{aligned} V_u &= 7.5 M_n/L = 7.5 (783\text{-ft-kip})/16\text{-ft} \\ &= 367\text{-kip} \end{aligned}$$

The shear force carried by the concrete is

$$\begin{aligned} V_c &= 2 (1 + N_u/2000A_g) (f'_c)^{1/2} b_w d \\ &= 2 [1 + 494,000\text{-lb}/2000(24\text{-in} \times 24\text{-in})] (7500\text{-psi})^{1/2} 21.5\text{-in } 24\text{-in} \\ &= 128\text{-kip} \end{aligned}$$

The shear force carried by the steel is

$$\phi (V_n) = \phi (V_c + V_s) \geq V_u$$

$$\begin{aligned} V_s &\geq V_u/\phi - V_c \\ &\geq 367\text{-kip}/1.0 - 128\text{-kip} \\ &\geq 239\text{-kip} \end{aligned}$$

Using  $s = 4$  in,  $d = 21.5$ -in,  $f_y = 75$  ksi, the required area for shear steel is

$$V_s = A_v f_y d/s$$

$$A_v \geq 0.59\text{-in}^2$$

Three #4 ties will work  $(0.2 \text{ in}^2/\text{tie}) (3 \text{ ties}) = 0.6 \text{ in}^2$

Note that Section 11.4.7.9 of ACI 318-11 states that “ $V_s$  shall not be taken greater than  $8 (f'_c)^{1/2} b_w d$ ”.

$$\begin{aligned} V_s &= 8 (f'_c)^{1/2} b_w d \\ &= 8 (5000)^{1/2} 21.5\text{-in } 24\text{-in} \\ &= 291.8\text{-kip} > 239\text{-kip.} \end{aligned}$$

OK

/2/

#### **D-3.4 Tie Force Evaluation Complete.**

After provision of ties forces and additional stirrups as specified in D-3.2 and D-3.3, the tie force procedure is complete and the structure meets minimum requirements for progressive collapse resistance.

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## APPENDIX E STRUCTURAL STEEL EXAMPLE

### E-1 INTRODUCTION.

A typical steel frame health care facility design and analysis example has been prepared to illustrate alternate path calculations. The structure is assumed to be occupied by 50 or more resident patients; placing the structure in Risk category III per UFC 3-301-01. This RC requires the Alternate Path Method to be applied to select elements to demonstrate capacity to resist progressive collapse as specified in Section 2-2. The structure does not include underground parking. Enhanced local resistance for all perimeter first story columns will also be required for this building per Section 2-2.

The example was prepared using tools and techniques commonly applied by structural engineering firms in the US. Computer software that is typical of that used for structural design was employed for preliminary design and for the alternate path analysis. To illustrate the various options given in the UFC, the example is prepared using the linear static and nonlinear dynamic analysis procedures.

### E-2 BASELINE PRELIMINARY DESIGN.

The structure is a four-story steel dual lateral system with a perimeter moment frame. The intended function of the building is health care, with occupancy of fifty or more resident patients. See Figures E-1 and E-2 for drawings of the building and the orientation of the members. The preliminary design, shown on the drawings below, has been sized to meet the requirements of IBC2006. In addition, the lateral drift of the frame has been evaluated for a performance limit of L/400 under a 10-year wind. Limited contribution of gravity framing due to partial restraint provided by simple connections was ignored for lateral load resistance (and stiffness).

#### E-2.1 Modeling Assumptions.

- 1) Members are represented by centerline elements (i.e. zero end offset to account for joint flexibility)
- 2) All moment connections are improved WUF.
- 3) Gravity framing connections are simple shear tabs, 3/8-in plate with 4 3/4-in A325N bolts and 1/4" weld with a depth of bolt group = 9-in, assumed to be pinned except for secondary member checks when they are considered partially restrained (PR) moment connections
- 4) Column to foundation connections are considered pinned
- 5) Each floor was taken as a rigid diaphragm
- 6) Gravity framing was designed as composite sections
- 7) All steel shapes ASTM A992
- 8) Concrete 4000 psi NWC
- 9) Floor system: 3" composite steel deck + 4 1/2" topping (total slab thickness = 7 1/2")
- 10) Roof system: metal deck only (no concrete fill)

## E-2.2 Loading Assumptions.

Dead loads (D):

Self weight of members

Floor: 3 + 4½" normal weight composite slab with a weight of 75 psf + 3 psf allowance for deck

Roof: metal deck 5 psf (including secondary members not modeled)

Super imposed load (SDL):

15 psf for ceiling weight, and mechanical loads (including membrane/insulation at roof)

Cladding (CL):

15 psf x 14'- 8" → 220 plf on perimeter of the building

Live load:

Floor (LL): 80 psf + 20 psf allowance for partitions

Roof (Lr): 20 psf

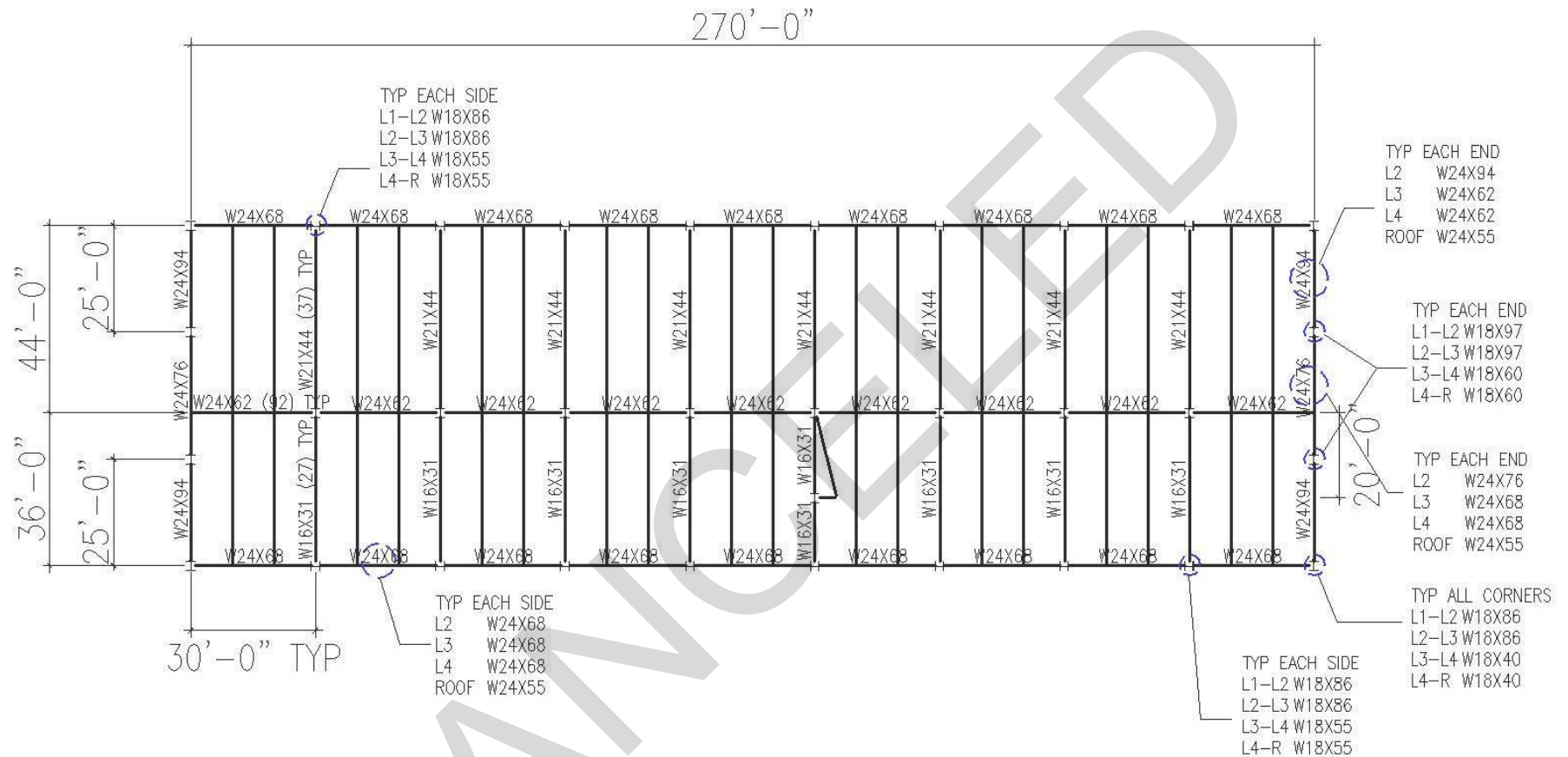
Wind Load (W) was determined per IBC 2006 using 110 mph with exposure = B and importance factor = 1.15

Earthquake Load (E) is assumed not to control the design because the building is in a non-seismic region.

Other Loads: Snow Loads (S), Rain Loads (R) are assumed to not control the design.

## E-2.3 Member Sizes.

Gravity floor design considers composite behavior and is identical for levels 2, 3 and 4. Roof gravity beams are non-composite with metal deck. Perimeter moment frames vary up the height of the building for drift control; see Figure E-1 for sizes.



**Figure E-1. Steel Building Plan**

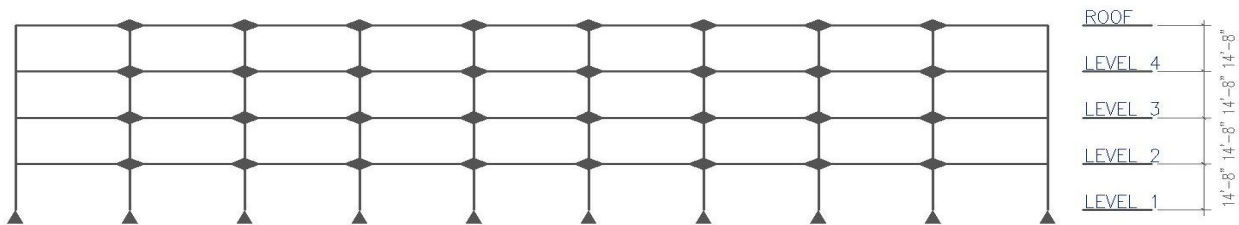


Figure E-2. Steel Building Elevation

### E-3 LINEAR STATIC PROCEDURE.

Locations of required columns removals are illustrated in Figure E-3. Each removal is considered separately. For the purpose of this example, the column below level 2 is removed. Section 3-2 requires additional analyses for removals at other levels.

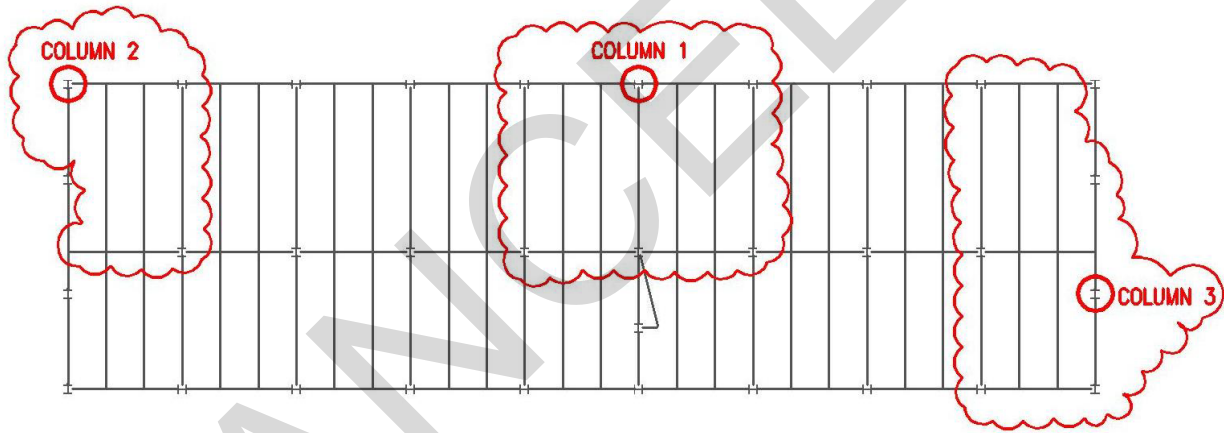


Figure E-3. Column Removal Locations

#### E-3.1 DCR and Irregularity Limitations.

The structure does not contain irregularities as defined by Section 3-2.11.1.1 and therefore DCR values are not limited for the use of the LSP.

#### E-3.2 Classification of Deformation Controlled and Force Controlled Actions.

Separate structural models are required to verify acceptability of components and actions which are deformation controlled and force controlled. Categorize these actions using the curves presented in Figure 3-7. A summary of classifications for this example is shown in Table E-1.



**Table E-1. Deformation Controlled Column Calculations**

Component	Deformation Controlled Action	Force Controlled Action
Moment Frames <ul style="list-style-type: none"> <li>• Beams</li> <li>• Columns</li> <li>• Joints</li> </ul>	Moment (M) M, Axial Load (P)	Shear (V) P, V V
Connections	M	V

### E-3.3 Determination of $m$ -Factors and Load Increase Factors.

Each component within the structure is assigned an  $m$  factor, or demand modifier which is determined from Table 5-1 and ASCE 41. Load increase factors (LIF) are applied to the area immediately affected by the removed column as required in Section 3-2.11. The LIF for the model to determine acceptability of force controlled actions is equal to 2. The LIF for the model to determine acceptability of deformation controlled actions is dependent on the lowest  $m$  factor for a component within the region of load increase. The  $m$  factors for each column removal location shown in Figure E-3 are summarized in Table E-2. The LIFs for deformation controlled actions based on these  $m$  factors are summarized in Table E-3.

**Table E-2. Component m Factors for Primary Deformation Controlled Actions**

Removed Column	Level	Beam/Girder	Beam/Girder m Factor	Simple Connection m Factor	Fixed Connection m Factor
1	2, 3, 4	W24x68	6.14	--	1.8
	2, 3, 4	W24x62	8	5.479	--
	2, 3	W24x146	8	--	1.79
	4	W24x117	6.52	--	1.79
	Roof	W24x55	8	--	1.8
	Roof	W24x62	8	5.479	--
	Roof	W24x76	8	5.479	1.8
2	2	W24x68	6.14	5.479	1.8
	2	W24x94	8	--	1.79
	2	W24x76	8	--	1.8
	2	W24x62	8	4.516	--
	2	W24x146	8	5.479	1.79
	3, 4	W24x68	6.14	5.479	1.8
	3, 4	W24x62	8	4.516	1.8
	3	W24x146	8	5.479	1.79
	4	W24x117	6.52	5.479	1.79
	Roof	W24x55	8	5.479	1.8
	Roof	W24x62	8	4.516	--
	Roof	W24x76	8	5.479	1.8
3	2	W24x68	6.14	5.479	1.8
	2	W24x94	8	--	1.79
	2	W24x76	8	--	1.8
	2	W24x62	8	4.516	--
	2	W24x146	8	5.479	1.79
	3, 4	W24x62	8	4.516	1.8
	3, 4	W24x68	6.14	5.479	1.8
	3	W24x146	8	5.479	1.79
	4	W24x117	6.52	5.479	1.79
	Roof	W24x55	8	5.479	1.8
	Roof	W24x62	8	5.479	--
	Roof	W24x76	8	5.479	1.8

**Table E-3. Load Increase Factors**

Removed Column	$m_{LIF}$ (Smallest <i>m-factor</i> )	$\Omega_{LD}$ , LIF for Deformation Controlled Actions	$\Omega_{LF}$ , LIF for Force Controlled Actions
1	1.8	2.72	2
2	1.79	2.71	2
3	1.79	2.71	2

### E-3.4 Alternate Path Analysis.

The software used for this example was SAP 2000NL. The details of this example can be generally applied in any structural software capable of nonlinear static analysis. The “Staged Construction” option in SAP was used to ensure proper redistribution of loads upon member removal. Comparable software should also have the capability of load redistribution, or loads must be redistributed manually.

#### E-3.4.1 Develop Preliminary Model.

See Figure E-4 for a model developed in SAP2000. Gravity beams not on column lines are not modeled.

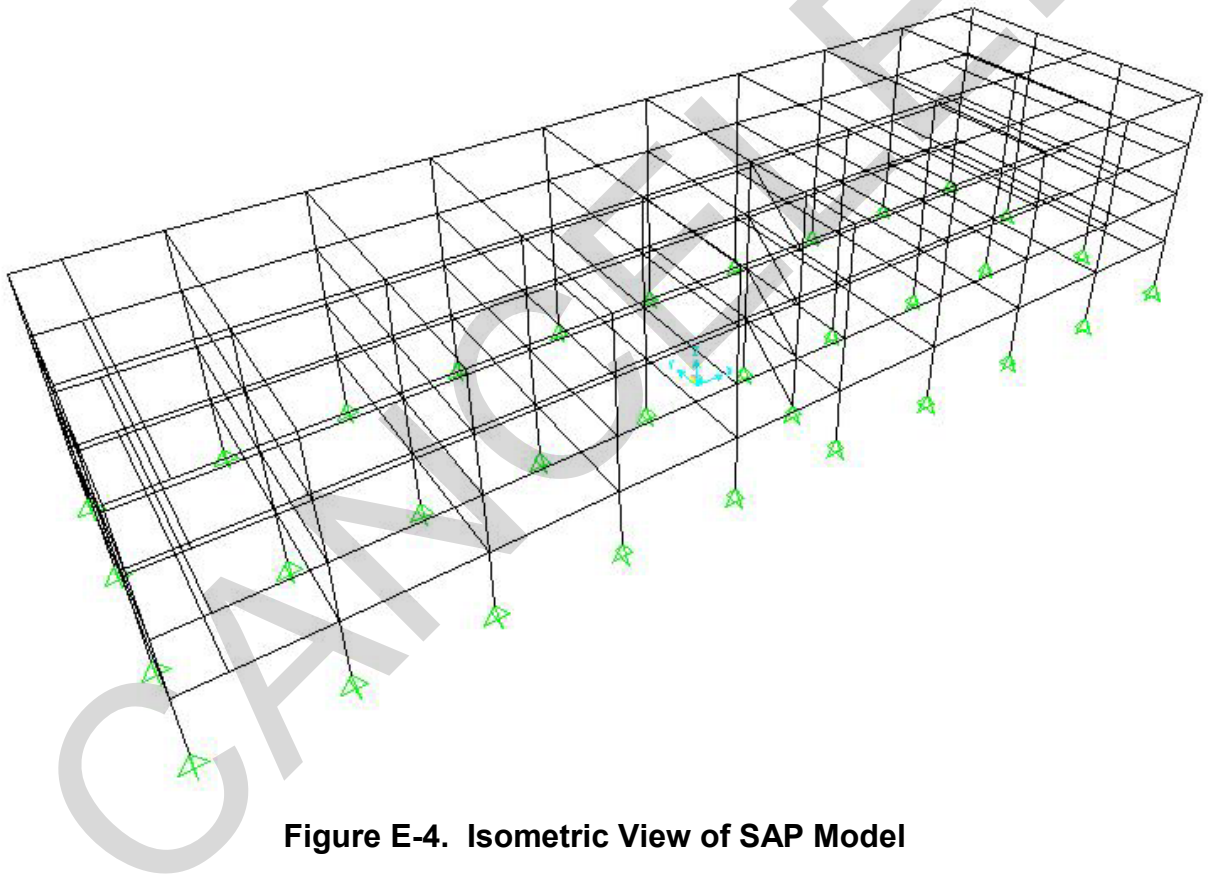


Figure E-4. Isometric View of SAP Model

#### E-3.4.2 Assign Groups.

Assign each column that is to be removed to a separate group. In this example, removal of three columns is demonstrated, each supporting the first elevated level. Columns are removed at three plan locations, one at a time.

### E-3.4.3 Define Load Cases and Assign Loads.

Since increased loads are only applied over the location of the removed column, separate load cases are needed to assign the increased loading over each element to be removed. Separate models are needed to check force controlled actions and deformation controlled actions because of different LIFs.

### E-3.4.4 Define Analysis Cases.

The “Staged Construction” option in SAP allows for the creation of separate analysis cases to automate the removal of columns. Create analysis cases which capture the stiffness for column removal. To do this, click Staged Construction button. In stage 1 add ALL, in stage 2 remove the column under investigation. Using these staged construction analysis cases as the initial stiffness, add a new analysis case for each column being removed. Within these analysis cases, assign all loads to be used in this analysis case per the load combinations in Equation 3-10 and 3-11. Click Nonlinear parameters button and choose P-delta option. It is possible to use P-delta + large displacements, but it is not necessarily needed for this analysis. Figure E-5 shows a screenshot of the interface for definition of analysis cases and their assigned loads.

**Analysis Case Data - Nonlinear Static**

Analysis Case Name:  Set Def Name Modify/Show...

Analysis Case Type:

Initial Conditions:  
☐ Zero Initial Conditions - Start from Unstressed State  
☒ Continue from State at End of Nonlinear Case   
Important Note: Loads from this previous case are included in the current case

Analysis Type:  
☐ Linear  
☒ Nonlinear  
☐ Nonlinear Staged Construction

Modal Analysis Case:  
 All Modal Loads Applied Use Modes from Case

Loads Applied

Load Type	Load Name	Scale Factor
Load	P1-di-DL-2	2.06
Load	Live	0.5
Load	slab	1.2
Load	roof	1.2
Load	LatXp	1.
Load	P1-di-DL-2	2.06
Load	P1-di-DL-3	2.06
Load	P1-di-DL-4	2.06

Add Modify Delete

Geometric Nonlinearity Parameters:  
☐ None  
☒ P-Delta  
☐ P-Delta plus Large Displacements

Other Parameters:  
 Load Application:  Modify/Show...  
 Results Saved:  Modify/Show...  
 Nonlinear Parameters:  Modify/Show...

OK Cancel

**Figure E-5. Analysis Case Definition**

### E-3.4.5 Define Design Combinations.

In order to use the SAP design procedures to evaluate acceptance criteria design combinations must be defined. Create a design combination for each analysis case (12 total design cases created in this example, additional cases would be required for column removal at other elevations).

### E-3.4.6 Run Analysis.

It is important to check that both stages of every analysis case converge. If the analysis does not converge, there is a problem with the model and it must be fixed.

### E-3.4.7 Run Design and Compare to Acceptance Criteria.

After each analysis case converges, perform the SAP design. The design details allow for the comparison of each components  $m$  factor to the ratio of  $Q_{UD}/\Phi Q_{CE}$  and comparison of  $Q_{UF}/\Phi Q_{CL}$  to unity. In this example, the deformation controlled moment acceptance at beam ends is governed by the  $m$  factor for the improved WUF moment connections. By reviewing the moment diagrams of the design combinations for the progressive collapse cases it was determined that the moment at beam ends is greater than at other locations along its length. Based on this information the moment ratio provided by SAP within the deformation controlled model for the defined design load combinations can be compared directly to the connection  $m$  factor of 1.80. The design details can also be used to review the beam shear demand ratio within the force controlled model. Figure E-6 shows SAP screen captures of the moment ratios from the deformation controlled model for the removal of column 1 prior to any member upgrades. Figure E-7 and Figure E-8 show similar screen captures for removal of columns 2 and 3 respectively. Also pictured are the interaction ratios of columns in the vicinity of the removed members. Column upgrades are discussed below.

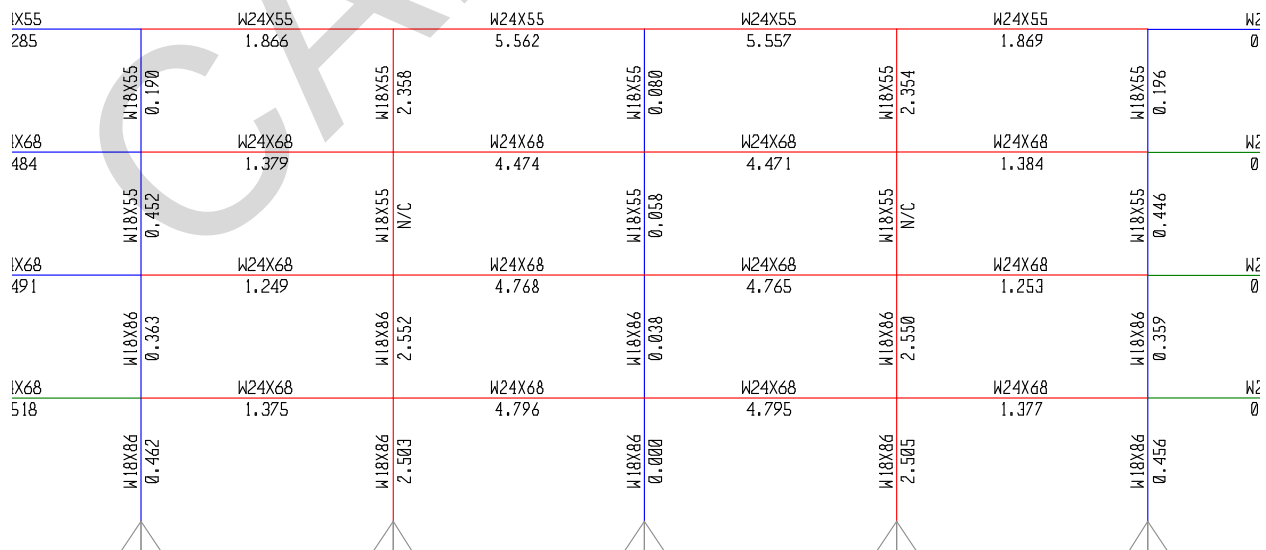


Figure E-6. Moment Ratios Due to Column 1 Removal with Original Design

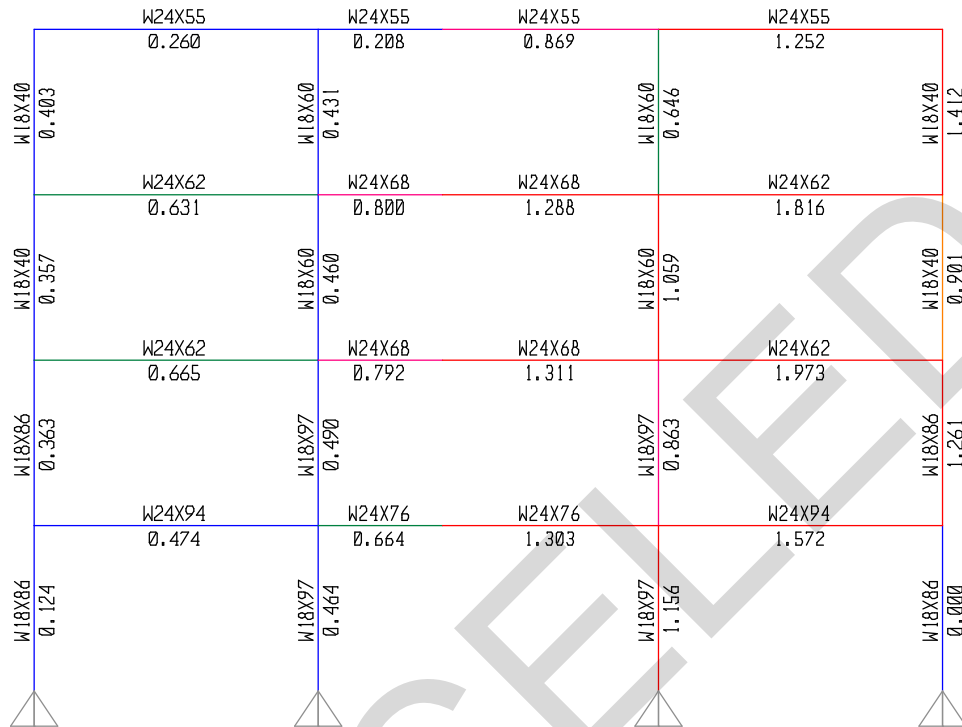


Figure E-7. Moment Ratios Due to Column 2 Removal with Original Design



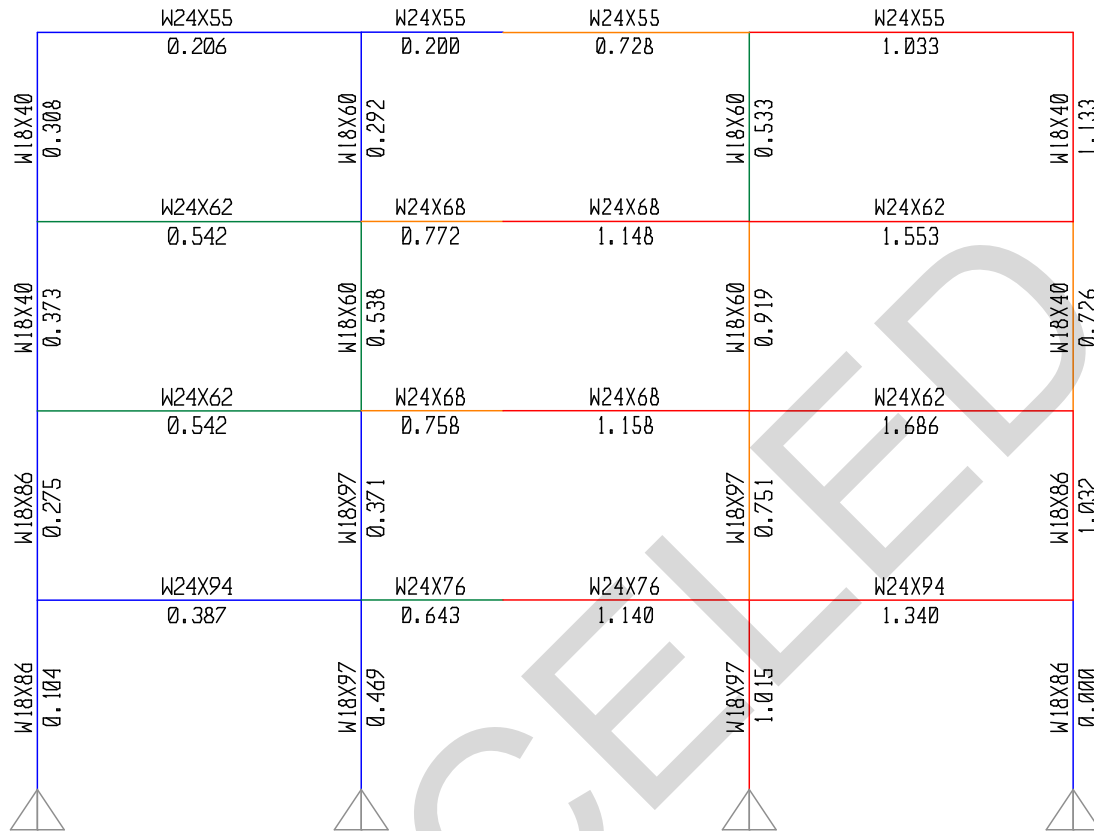
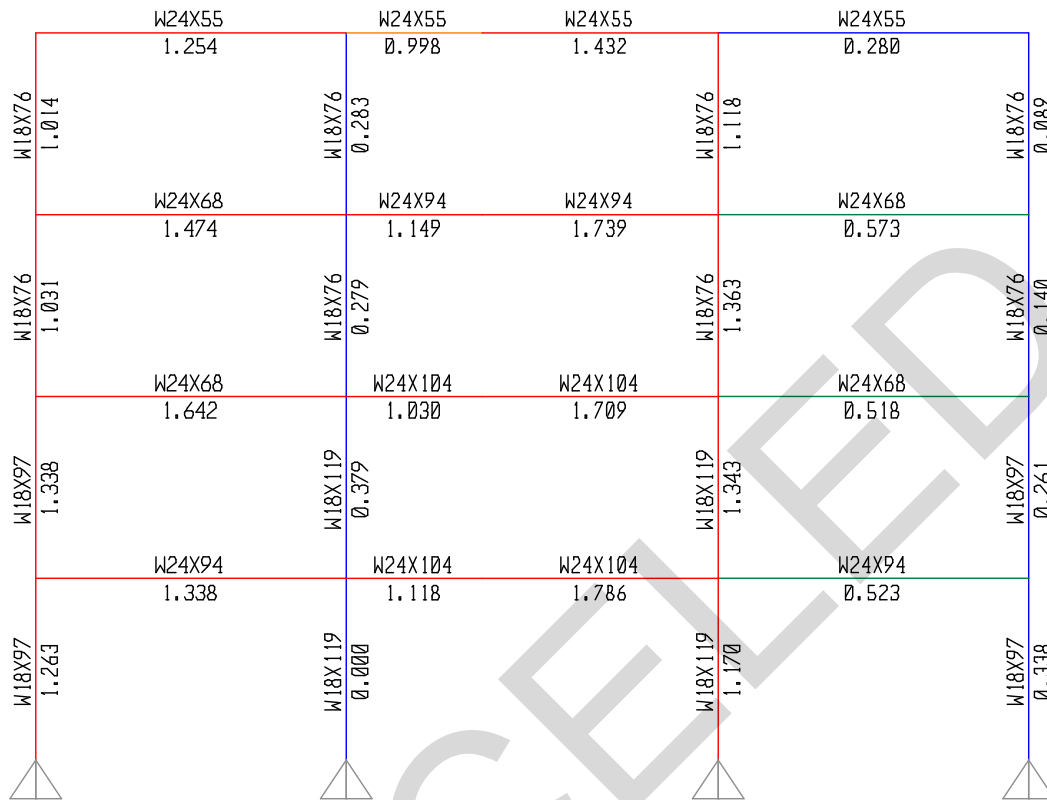


Figure E-10. Moment Ratios Due to Column 2 Removal with Redesign





**Figure E-11. Moment Ratios Due to Column 3 Removal with Redesign**

To determine the acceptability of columns, the deformation-controlled model is reviewed to determine the level of axial load. In accordance with ASCE 41, any column with an axial load ratio of greater than or equal to 0.5 must be checked using the model for force controlled actions. These force controlled columns must have interaction values that do not exceed 1. Using the force controlled model, column sizes are increased to reduce interaction values less than unity or to reduce the axial demand in order to evaluate the column as deformation controlled.

Deformation controlled columns (axial load ratio less than 0.5) are checked using the acceptance criteria from ASCE 41. An example calculation for this verification of acceptance is shown in Table E-4.

**Table E-4. Deformation Controlled Column Calculations**

Removed Column	1
Column Size	W18x106
$P/\phi P_n$	0.49
$8/9 M/\phi M_n$	0.59
$m = 9 (1.5/3 P/P_{cl})$	ASCE 41 Table 9-4
$m = 9 (1.5/3 \times 0.49 \times 0.9)$	2.385
Interaction = $P/\phi P_n + (8/9 M/\phi M_n)/m$	$0.49 + 0.59/2.385 = 0.73$ <b>OK</b>

### **E-3.4.8 Secondary Component Checks.**

After verifying that all primary members satisfy the force- and deformation-controlled acceptance criteria, the secondary members must also be checked. The following calculations present the checks for the gravity beam and the interior connection of the gravity beam at removal location 1, where the closest secondary beam to the loss location is a W21x44 that spans 44-ft.

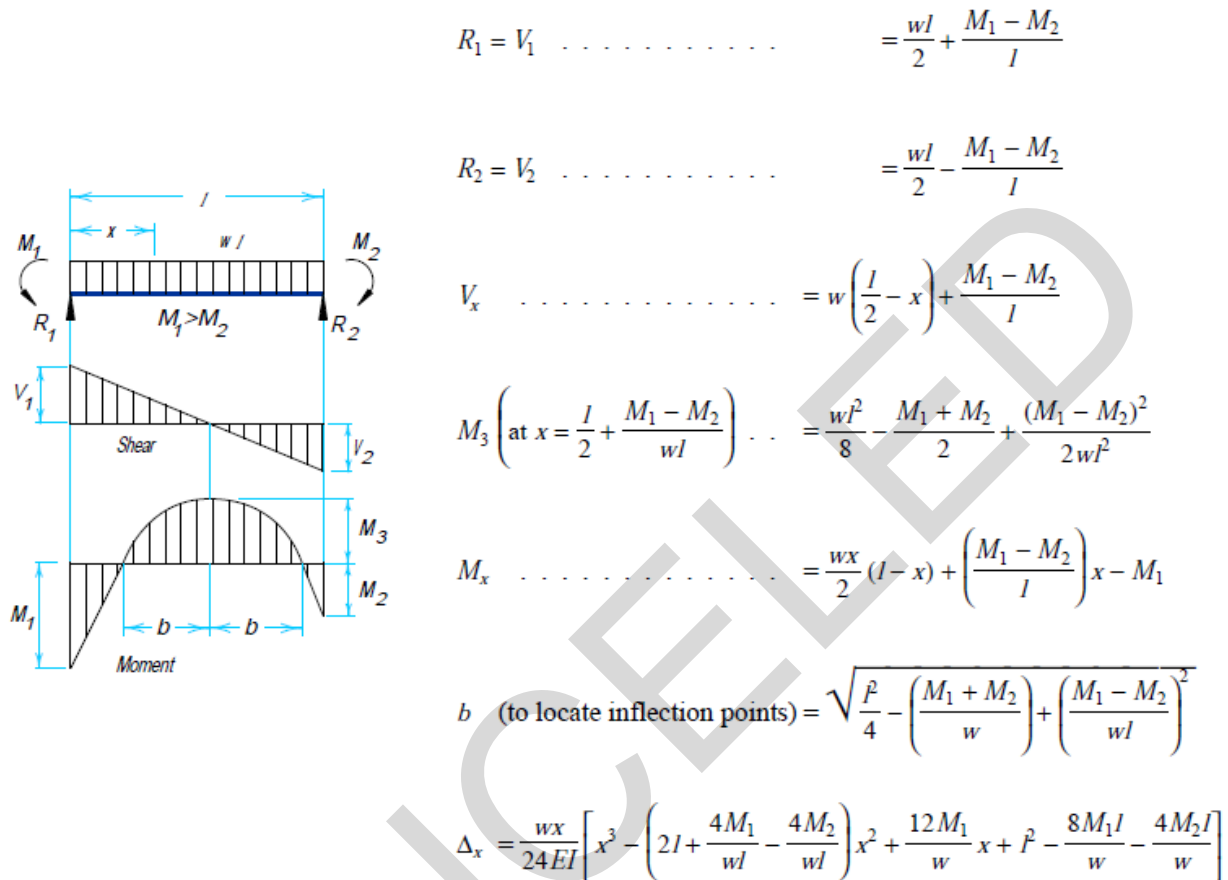
Acceptance checks of gravity beams in steel frame structures present a unique challenge within the framework of linear static analysis. Typically, gravity beams and simple shear tab connections are considered secondary members and not included in the linear static model. However, these members may fail if subjected to large dynamic loads and/or significant deflections due to column removal and may pose a hazard to occupants or lead to additional structural failure if they detach and impact the floor below. While force- and deformation-controlled actions can be checked in a straight-forward manner with nonlinear procedures, the linear static procedure and criteria are based on m-factors applied to the moments and other deformation-controlled actions and thus moments must be determined to perform the checks, even at the ends of gravity beams which are often considered to be pinned. While allowable plastic rotations are available for nonlinear static and nonlinear dynamic procedures, they cannot be used for checks in the linear static method, as the analysis procedures are different, in terms of load adjustment factors, explicit consideration of hinges, fidelity of the nonlinear models, etc. Thus, the linear static acceptance criteria must be based on moments, shears, and forces. As simple shear tab connections can be considered partially restrained (PR) connections, their flexural strength can be calculated with an approximate rotational stiffness and the overall rotations, for comparison to the flexural demand.

#### **E-3.4-8.1 Deformation-Controlled Actions**

For the gravity beam and the simple shear tab connection, the deformation controlled actions are the moments.

#### **Gravity Beam**

There are two contributions to the peak moment demand in the gravity beam. The first is due to the factored linear static load; while the gravity beam is not included in the linear static model, it will experience dynamic and nonlinear effects and the load must be increased accordingly. The second is the end moment created by the rotational stiffness of the simple shear tab connection and the displacements at the end of the beam, as determined from the linear static analysis for column removal at location 1, using the deformation-controlled load case. The combination of end moments and uniform load corresponds to the loading case shown in Figure E-12, which is taken from the AISC LRFD design manual.



**Figure E-12. Shear and Moment in Simply Supported Beam with End Moments and Uniform Loading**

The load  $w$  in Figure E-12 is the factored linear static load as calculated with Equation 3-10:

$$w = G_{LD} = \Omega_{LD} [1.2 D + (0.5 L \text{ or } 0.2 S)]$$

$$w = G_{LD} = 2.7 [1.2 (SW + DL_{\text{floor}} + SDL) + 0.5 (LL_{\text{floor}})] = 4.5\text{-kip/ft}$$

where

$\Omega_{LD}$  = Load increase factor = 2.7 as discussed earlier for column removal location 1

SW = Self Weight = 44-lb/ft for W21/44

$DL_{\text{floor}}$  = Dead Load of floor over tributary area of gravity beam = 78-psf \* 10-ft

SDL = Superimposed Dead Load over tributary area = 15-psf \* 10-ft

$LL_{\text{floor}}$  = Live Load of floor over tributary area of gravity beam = 100-psf \* 10-ft

The end moments in Figure E-12 are calculated from the rotations created by the relative displacement at the end of the beams, as calculated for the column removal at location 1 using the deformation-controlled load case. The rotation is calculated as:

$$\theta = \Delta/L = 4.09\text{-in}/528\text{-in} = 0.0077\text{-rad}$$

where

$$\begin{aligned}\theta &= \text{Chord rotation} \\ \Delta &= \text{Relative displacement} = 4.09\text{-in} \\ L &= \text{Beam length} = 44\text{-ft} = 528\text{-in}\end{aligned}$$

To determine the resulting end moments, the approximate stiffness for a partially restrained connection is calculated using Eq. 9-15 from ASCE 41:

$$K_o = M_{CE}/0.005$$

where

$M_{CE}$  = Expected moment strength of the simple shear tab connection

The expected moment strength for the simple shear connection is based on the shear strength of the connection multiplied by the eccentricity of the bolt group, which is 3.5 inches in accordance with Figure 10-11 of the AISC Manual, 13<sup>th</sup> Edition. The shear strength from Table 10-9a of the AISC Manual is 63.6-k for 4 ¾-in A325N bolts. The expected moment strength and partially restrained connection stiffness are:

$$\begin{aligned}M_{CE} &= 63.6\text{-kip} \times 3.5\text{-in} = 22.6\text{-in-kip} \\ K_o &= 22.6\text{-in-kip}/0.005 = 44.5 \times 10^3\text{-in-kip/rad}\end{aligned}$$

Thus, the end moment demands ( $M_1$  and  $M_2$  in Figure E-12) are

$$M_1 = M_2 = K_o \theta = 44.5 \times 10^3\text{-in-kip/rad} \times 0.0077\text{-rad} = 342.7.0\text{-in-kip} = 28.55\text{-ft-kip}$$

The maximum moment demand is found using the equation for  $M_3$  in Figure E-12.

$$\begin{aligned}M_3 &= w L^2/8 - (M_1 + M_2)/2 + (M_1 - M_2)^2/2wL^2 \\ &= 4.5\text{-kip/ft} [(44\text{-ft})^2]/8 - (28.55\text{-ft-kip} - 28.55\text{-ft-kip})/2 + \\ &\quad (28.55\text{-ft-kip} + 28.55\text{-ft-kip})^2/[2 (4.5\text{-kip/ft}) (44\text{-ft})^2] \\ &= 1089\text{-ft-kip}\end{aligned}$$

This is the demand or  $Q_{UD}$  for the gravity beam. For a W21x44, the expected moment strength is

$$Q_{CE} = \Omega F_y Z_x = 1.1 \times 50\text{-ksi} \times 95.4\text{-in}^3 = 5247\text{-in-kip} = 437.3\text{-ft-kip}.$$

where

$$\begin{aligned}\Omega &= \text{Overstrength factor} = 1.1 \text{ from ASCE 41 Table 9-3} \\ F_y &= \text{Yield strength} = 50\text{-ksi} \\ Z_x &= \text{Plastic modulus, from AISC LRFD design manual} = 95.4\text{-in}^3\end{aligned}$$

Note that the unbraced length must be checked as part of the design.

Checking Equation 3-13 and with  $m = 12$  for a secondary compact beam, per ASCE 41 Table 9.5,

$$\begin{aligned}\Phi m Q_{CE} &\geq Q_{UD} \\ 0.9 (12) (437.3\text{-ft-kip}) &= 4723\text{-ft-kip} \geq 1089\text{-ft-kip} \quad \text{OK}\end{aligned}$$

### Simple Shear Tab Connection

A somewhat similar procedure is performed for the simple shear tab connection. In this case, there are two contributions to the moment demand at the connection. The first is the moment demand created by the shear reaction from the factored linear static load, multiplied by the eccentricity of 3.5-in. In this case, the factored linear static load is used to calculate the shear reaction and moment demand:

$$\begin{aligned}w &= G_{LD} = 2.7 [1.2 (SW + DL_{\text{floor}} + SDL) + 0.5 (LL_{\text{floor}})] = 4.5\text{-kip/ft} \\ V &= 0.5 w L + (M_1 - M_2)/L = 0.5 (4.5\text{-kip/ft}) 44\text{-ft} + (28.55\text{-kip} + 28.55\text{-kip})/44\text{-ft} = \\ &\quad 100.3\text{-kip} \\ M_{\text{Dload}} &= V * 3.5\text{-in} = 351\text{-in-kip}\end{aligned}$$

The second moment demand is generated by the relative displacements at the end of the gravity beam, as calculated from the linear static model for removal location 1 with the factored linear static load. The chord rotation is as before,  $\theta = 0.0077\text{-rad}$ . The approximate stiffness for a partially restrained connection is calculated with Equation 9-15 of ASCE 41, or  $K_o = 44.5 \times 10^3\text{-in-kip/rad}$ . This moment demand is

$$M_{\text{Ddispl}} = K_o \theta = 44.5 \times 10^3\text{-in-kip/rad} \cdot 0.0077\text{-rad} = 342.7\text{-in-kip}$$

The total demand is

$$M_{UD} = M_{\text{Dload}} + M_{\text{Ddispl}} = 351\text{-in-kip} + 342.7\text{-in-kip} = 693.7\text{-in-kip}$$

The strength of the simple shear tab connection was calculated earlier and is based on the design shear load for the connection times the eccentricity of the bolt group or

$$M_{CE} = 63.6\text{-kip} * 3.5\text{-in} = 222.6\text{-in-kip}$$

For the simple shear tab connection, Table 5-1 in Chapter 5 shows that the  $m$ -factor for secondary members is.

$$m = 8.7 - 0.161d_{bg} = 7.25$$

where

$d_{bg}$  = Depth of bolt group or 9-in with 4  $\frac{3}{4}$ -in A325N bolts.

Checking Equation 3-13 and with  $m = 7.25$  for a simple shear tab connection, per Table 5.1,

$$\Phi m Q_{CE} \geq Q_{UD}$$

$$0.9 (7.725) (222.6\text{-in-kip}) = 1613.9\text{-in-kip} \geq 693.7\text{-in-kip} \quad \text{OK}$$

### E-3.4-8.2 Force-Controlled Actions

For the gravity beam and the simple shear tab connection, the force-controlled action is the shear.

#### Gravity Beam

There are two contributions to the peak shear demand in the gravity beam. The first is due to the factored force-controlled linear static load; while the gravity beam is not included in the linear static model, it will experience dynamic and nonlinear effects and the load must be increased accordingly. The second is the end shear due to the end moments created by the displacements at the end of the beam (as determined from the linear static model with the force-controlled load case) and the rotational stiffness of the simple shear tab connection. The combination of end moments and uniform load corresponds to the loading case shown in Figure E-12.

The load  $w$  in Figure E-12 is the factored linear static load as calculated with Equation 3-12.

$$w = G_{LF} = \Omega_{LF} [1.2 D + (0.5 L \text{ or } 0.2 S)]$$

$$w = G_{LF} = 2.0 [1.2 (SW + DL_{\text{floor}} + SDL) + 0.5 (LL_{\text{floor}})] = 3.34\text{-kip/ft}$$

The end moments in Figure E-12 are calculated from the rotations created by the relative displacement at the end of the beams, as calculated for the column removal at location 1 using the force-controlled load case; this displacement is 3.03-in. The rotation is calculated as:

$$\theta = \Delta/L = 3.03\text{-in}/528\text{-in} = 0.0057\text{-rad}$$

To determine the resulting end moments, the approximate stiffness for a partially restrained connection is used, as calculated earlier.

$$K_o = M_{CE}/0.005 = 44.5 \times 10^3\text{-in-kip/rad}$$

Thus, the end moment demands ( $M_1$  and  $M_2$  in Figure E-12) are

$$M_1 = M_2 = K_o \theta = 44.5 \times 10^3 \text{-in-kip/rad} \times 0.0057 \text{-rad} = 253.7 \text{-in-kip} = 21.1 \text{-ft-kip}$$

The maximum shear demand is found using the equation for  $V_1$  in Figure E-12.

$$\begin{aligned} V_1 &= w L/2 + (M_1 - M_2)/L \\ &= 3.34 \text{-kip/ft} [(44\text{-ft})]/2 + (21.1 \text{-ft-kip} + 21.1 \text{-ft-kip})/44\text{-ft} \\ &= 74.4 \text{-kip} \end{aligned}$$

This is the demand or  $V_{UF}$  for the gravity beam. For a W21x44, the lower bound shear strength is

$$V_{CL} = 0.6 t_w d F_y = 0.6 (0.35\text{-in}) (20.66\text{-in}) (50\text{-ksi}) = 216.9 \text{-kip}$$

where

$$\begin{aligned} t_w &= \text{Web thickness for W21x44} = 0.35\text{-in} \\ d &= \text{Depth of W21x44} = 20.66\text{-in} \\ F_y &= \text{Yield stress} = 50\text{-ksi} \end{aligned}$$

Checking Equation 3-13,

$$\begin{aligned} \Phi Q_{CL} &\geq Q_{UF} \\ 0.9 (216.9 \text{-kip}) &= 195.2 \text{-kip} \geq 74.4 \text{-kip} \quad \text{OK} \end{aligned}$$

### Simple Shear Tab Connection

A similar procedure is performed for the simple shear tab connection. In this case, there are two contributions to the shear demand at the connection. The first is the shear demand created by the shear reaction from the factored linear static load. In this case, the factored linear static load is used to calculate the shear demand:

$$\begin{aligned} w &= G_{LD} = 2.0 [1.2 (SW + DL_{\text{floor}} + SDL) + 0.5 (LL_{\text{floor}})] = 3.338 \text{-kip/ft} \\ V_{\text{Dload}} &= 0.5 w L + (M_1 - M_2)/L = 0.5 (3.338 \text{-kip/ft}) 44\text{-ft} + (21.1 \text{-ft-kip} + 21.1 \text{-ft-kip})/44\text{-ft} \\ &= 74.4 \text{-kip} \end{aligned}$$

The second shear demand is generated by the moment created by relative displacements at the end of the gravity beam, as calculated from the linear static model for removal location 1 with the factored force-controlled linear static load. The chord rotation and stiffness are as before,  $\theta = 0.0057 \text{-rad}$  and  $K_o = 44.5 \times 10^3 \text{-in-kip/rad}$ . This moment demand is

$$M_{\text{Ddispl}} = K_o \theta = 44.5 \times 10^3 \text{-in-kip/rad} \times 0.0057 \text{-rad} = 253.7 \text{-in-kip}$$

From statics for a beam subjected to two end moments, the shear demand due to displacement is

$$V_{Ddispl} = 2 M_{Ddispl}/L = 2 (253.7\text{-in-kip})/528\text{-in} = 0.96\text{-kip}$$

The total demand is

$$V_{UF} = V_{DLoad} + V_{Ddispl} = 74.4\text{-kip} + 0.96\text{-kip} = 75.4\text{-kip}$$

The shear strength of the simple shear tab connection is taken from Table 10-9a of the AISC Manual.

$$V_{CL} = 63.6\text{-kip}$$

Checking Equation 3-13,

$$\Phi Q_{CL} \geq Q_{UF}$$
$$0.9 (63.6\text{-kip}) = 57.2\text{-kip} \geq 75.4\text{-kip} \quad \textbf{NG}$$

Thus, the shear strength of the simple shear tab connection must be increased. Since the original design strength of the connection is based on the typical design load combination (1.2D + 1.6L) and the demand is based on the smaller load combination (1.2D + 0.5L) multiplied by the LIF of 2.0, this is not unexpected. Therefore, the strength of the simple shear tab must be increased to 75.4-kips and the deformation-controlled action (the moment) for the connection must be re-checked. For this example, the deformation controlled action is acceptable, by inspection.

Note that the axial force in these beams and connections are not checked, due to the small displacements at the ends of the beams. Similarly, the concrete slab is not checked, based on the small rotations and engineering judgment.

### E-3.5 Enhanced Local Resistance.

ELR provisions for \3\ Risk Category /3/ III require that all perimeter columns at the first floor above grade achieve design shear strengths that exceed the shear demand associated with the flexural demand of the columns. For \3\ Risk Category /3/ III, the flexural demand is determined from the building design after it meets the AP requirements. For the purposes of ELR evaluation, the columns are considered fixed at the first level above grade due to continuity of the column above the first level and the rigid support provided by the floor diaphragm and pinned at the base. The flexural demand is based on the development of a 3-hinge mechanism in the column, with one hinge present as the pinned end of the column, a plastic hinge that develops at the fixed end at the first level above grade and a plastic hinge that develops within the column.

The required shear strength (i.e., the shear demand) can be found in plastic structural design references or in Table 4-4 of PDC TR-06-01 *Methodology Manual for the Single-Degree-of-Freedom Blast Effects Design Spreadsheets (SBEDS)*. From PDC



TR-06-01, the largest reaction/shear for a pinned-fixed, uniformly loaded beam is  $5 r_u L/8$ , where  $r_u$  is the ultimate resistance and equal to  $12 M_p/L^2$ .  $M_p$  is the plastic moment of the column cross-section and thus is the nominal flexural strength of the perimeter columns, which have been sized to meet the AP requirements as well as standard design requirements. Thus,

$$\begin{aligned} V_u &= 5 r_u L/8 \\ r_u &= 12 M_p/L^2 \end{aligned}$$

and

$$V_u = 7.5 M_p / L \quad \text{Equation (E-1)}$$

where

$V_u$	=	Shear demand (required shear strength)
$M_p$	=	Nominal flexural strength, accounting for axial load; also, the plastic moment
$r_u$	=	Ultimate resistance,
$L$	=	Column height

In this example, the design shear strength for all columns other than corner columns exceeds the shear demand so no member size increases were required beyond those determined using the alternate path procedure. For corner columns, there is no W18 section that can provide adequate design shear strength to meet the shear demand. To satisfy these provisions, web doubler plates were added. At these corner locations the axial load is less than 10% of the column capacity and was ignored.

The corner column is a W18x97 with a strong axis plastic modulus of  $211 \text{ in}^3$  per Figure E-11. The shear demand is calculated with Equation E-1 and using an over-strength factor of 1.1 (per Section 3-3.1) is

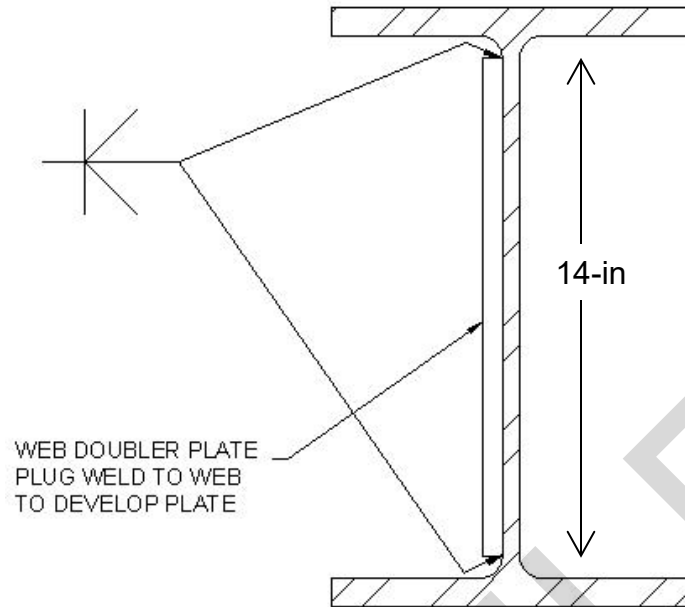
$$V_u = 7.5 M_p / L = 7.5 * 1.1 * 50\text{-ksi} * 211\text{-in}^3 / 174\text{-in} = 495\text{-kips}$$

The design shear strength (per AISC Chapter G) is:

$$\Phi V_n = 1.0 * 0.6 * t_w * h * F_y = 0.6 * 0.535\text{-in} * 18.6\text{-in} * 50\text{-ksi} = 298\text{-kips}$$

The required additional design shear strength is achieved by adding a doubler plate to the web of the W18x97. The thickness is found by subtracting the design shear strength (298-kips) from the shear demand (495-kips) and using this difference to size a 14-in high plate. A 1/2" thick doubler plate is required for all corner columns. See Figure E-13 for a diagram of the doubler plate used to enhance strong axis shear strength.

/2/



**Figure E-13. Web Doubler Plate for ELR**

### **E-3.6 Alternate Path Design Method Complete.**

Once the model converges, all acceptance criteria have been met, and enhanced local resistance has been provided, the building has satisfied progressive collapse resistance requirements of this UFC.

#### E-4 NON LINEAR DYNAMIC PROCEDURE (NDP).

Locations of required columns removals are illustrated in Figure E-14. Each removal is considered separately. For the purpose of this example, the column below level 2 is removed, Section 3-2 requires additional analyses for removals at other levels.

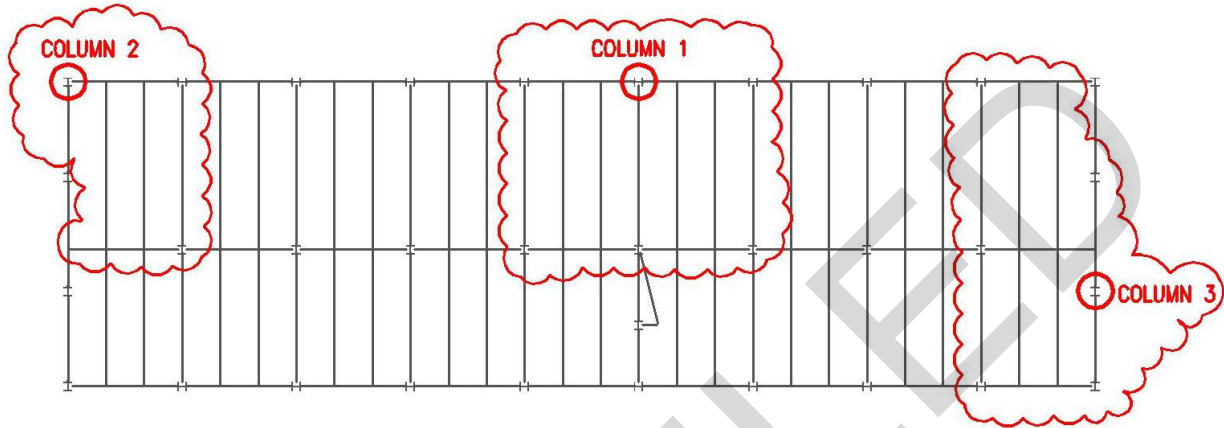


Figure E-14. Column Removal Locations

##### E-4.1 DCR and Irregularity Limitations.

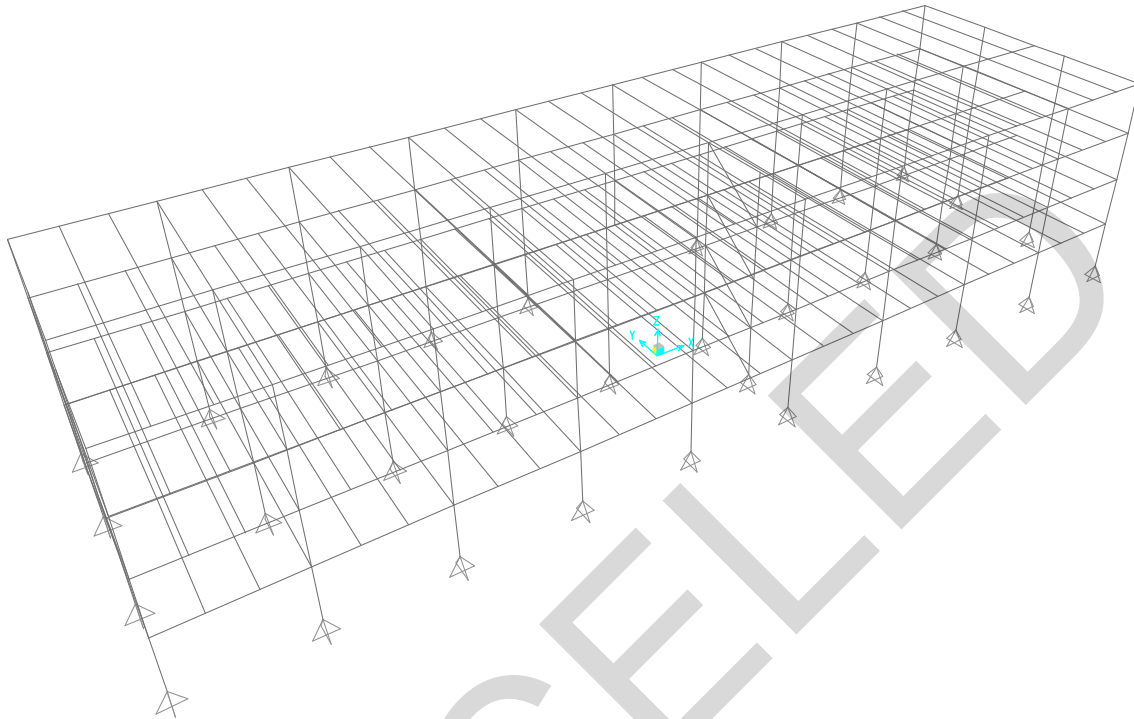
There are no DCR or geometric irregularity limitations on the use of the NDP.

##### E-4.2 Alternate Path Analysis.

The software used and screen shots depicted for this example was SAP 2000NL. The details of this example can be generally applied in any structural software capable of nonlinear static analysis. The “Staged Construction” option in SAP was used to ensure proper redistribution of loads upon member removal. Comparable software should also have the capability of load redistribution, or loads must be redistributed manually.

##### E-4.2.1 Develop Preliminary Model.

See Figure E-15 for a model developed in SAP2000. All beams and columns are modeled including gravity beams.



**Figure E-15. Isometric View of SAP Model**

#### **E-4.2.2 Define Load Cases and Assign Loads.**

The dynamic nature of this procedure does not require an increase factor to account for dynamic effects. No additional load cases are needed to account for dynamic load increase. For this reason, a single model may be used to verify acceptance of force controlled actions and deformation controlled actions.

#### **E-4.2.3 Define Analysis Cases.**

The nonlinear dynamic procedure requires one analysis case for each column removal. Analysis cases are created to determine the forces present at equilibrium in each column to be removed. For each column removal, the column member is deleted in the structural model and the internal forces determined from the equilibrium model are applied to the structure as a load case to the joint or joints at each column end. These static nonlinear analysis cases are used as the starting conditions for the column removals.

Within these analysis cases, assign all loads to be used in this analysis case per the load combination in Equation 3-18. Click Nonlinear parameters button and choose P-delta option. It is possible to use P-delta + large displacements, but it is not necessarily

needed for this analysis. Figure E-16 shows a screenshot of the interface for definition of analysis cases and their assigned loads.

**Analysis Case Data - Nonlinear Static**

Analysis Case Name: PreXp    Set Def Name    Notes: Modify/Show...

Analysis Case Type: Static

Initial Conditions:
 

- ☒ Zero Initial Conditions - Start from Unstressed State
- ☐ Continue from State at End of Nonlinear Case

 Important Note: Loads from this previous case are included in the current case

Analysis Type:
 

- ☐ Linear
- ☒ Nonlinear
- ☐ Nonlinear Staged Construction

Modal Analysis Case:
 All Modal Loads Applied Use Modes from Case: MODAL

Geometric Nonlinearity Parameters:
 

- ☐ None
- ☒ P-Delta
- ☐ P-Delta plus Large Displacements

Loads Applied:

Load Type	Load Name	Scale Factor
Load	DEAD	1.2
Load	DEAD	1.2
Load	Clad	1.2
Load	MEP	1.2
Load	Live	0.5
Load	Part	0.5
Load	LatXp	1.
Load	C2-L2Xp	1.

Buttons: Add, Modify, Delete

Other Parameters:
 

- Load Application: Full Load    Modify/Show...
- Results Saved: Final State Only    Modify/Show...
- Nonlinear Parameters: Default    Modify/Show...

Buttons: OK, Cancel

**Figure E-16. Analysis Case Definition**

After equilibrium is reached for the structure, remove the column by ramping down the column forces under a duration for removal of less than one tenth of the period associated with the structural response mode for the element removal. The analysis shall continue until the maximum displacement is reached or one cycle of vertical motion occurs at the column or wall section removal location. Figure E-17 shows a screen shot of the column removal analysis case definition.

**Analysis Case Data - Nonlinear Direct Integration History**

Analysis Case Name: DNL-Xp-PC [Set Def Name] [Modify/Show...]

Notes: [Modify/Show...]

Analysis Case Type: Time History

Initial Conditions:
 

- ☐ Zero Initial Conditions - Start from Unstressed State
- ☒ Continue from State at End of Nonlinear Case: PreXp

 Important Note: Loads from this previous case are included in the current case

Analysis Type:
 

- ☐ Linear
- ☒ Nonlinear

Time History Type:
 

- ☐ Modal
- ☒ Direct Integration

Geometric Nonlinearity Parameters:
 

- ☐ None
- ☒ P-Delta
- ☐ P-Delta plus Large Displacements

Modal Analysis Case:
 

- Use Modes from Case: MODAL

Loads Applied:

Load Type	Load Name	Function	Scale Factor
Load	C2-L2-Xp	RDown	1.
Load	C2-L2-Xp	RDown	1.

[Add] [Modify] [Delete]

☐ Show Advanced Load Parameters

Time Step Data:
 

- Number of Output Time Steps: 200
- Output Time Step Size: 5.000E-03

Time History Motion Type:
 

- ☒ Transient
- ☐ Periodic

Other Parameters:
 

- Damping: Proportional Damping [Modify/Show...]
- Time Integration: Hilber-Hughes-Taylor [Modify/Show...]
- Nonlinear Parameters: User Defined [Modify/Show...]

[OK] [Cancel]

Figure E-17. Analysis Case Definition

#### E-4.2.4 Define Design Combinations.

SAP design procedures may be used to evaluate whether columns are deformation or force controlled. Design checks also aid in the definitions of column hinges by determining axial load demand and capacity. Create a design combination for each analysis case (12 total design cases created in this example, additional cases would be required for column removal at other elevations).

#### **E-4.2.5 Run Dynamic Analysis.**

It is important to check that both stages of every analysis case converge. If the analysis does not converge, there is a problem with the model and it must be fixed. The problem could be numerical with assumptions made in SAP, but the most likely reason is that the model has a plastic hinge that failed or a mechanism has formed. At this point, the model cannot support the load.

#### **E-4.2.6 Plastic Hinges.**

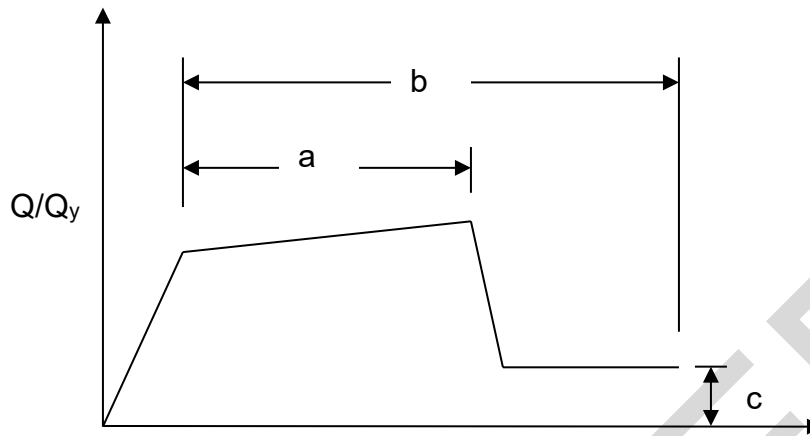
For the nonlinear alternate load path method, plastic hinges are allowed to form along the members. These hinges are based on maximum moment values calculated using phi factors and over-strength factors per the UFC. However, only flexural moments can cause a plastic hinge to form in beam members, and only the axial-moment interaction (PMM) can cause a plastic hinge to form in a column. Any shear or torsion values that would cause a hinge to form would result in an immediate failure.

#### **E-4.2.7 Hinge Locations.**

Theoretically hinges can occur anywhere along the beam. However, hinges are allowed to occur at the ends of each member and at the midspan of the flexural members. This simplifies the model by placing hinges in the most probable locations.

#### **E-4.2.8 Hinge Properties.**

Nonlinear acceptance criteria and component definitions are from Chapter 9 of ASCE 41 for the Life Safety condition for primary and secondary components. Use the modeling parameters and guidance, including definitions of stiffness, to create the analytical model. For beams subjected to flexure or flexure plus axial tension, use the Collapse Prevention values for primary and secondary elements. For the Fully Restrained (FR) and Partially Restrained (PR) connections listed in Tables 5-1 and 5-2 in this UFC, use the specified plastic rotations and modeling parameters as given. Figure E-18 shows the form of the plastic hinges for this model.



**Figure E-18. Generalized Force-Deformation Hinge Definition**

Beam and connection hinge properties are determined using ASCE 41 Table 9-6 and UFC Table 5-2. A summary of hinge properties and acceptance criteria for beams and connections corresponding to Figure E-18 are shown in Tables E-5, E-6 and E-7. When defining connection and beam hinges, be certain to include applicable strength reduction factors per AISC as required by this UFC.

**Table E-5. Beam Hinge Properties**

Beam	Plastic Rotation Angle		Residual Strength Ratio	Acceptance Criteria	
	a	b		Primary	Secondary
W24x55	$9\theta_y$	$11\theta_y$	0.6	$8\theta_y$	$11\theta_y$
W24x62	$9\theta_y$	$11\theta_y$	0.6	$8\theta_y$	$11\theta_y$
W24x76	$9\theta_y$	$11\theta_y$	0.6	$8\theta_y$	$11\theta_y$
W24x84	$9\theta_y$	$11\theta_y$	0.6	$8\theta_y$	$11\theta_y$
W24x94	$9\theta_y$	$11\theta_y$	0.6	$8\theta_y$	$11\theta_y$
W24x131	$9\theta_y$	$11\theta_y$	0.6	$8\theta_y$	$11\theta_y$
W24x68	$7.15\theta_y$	$9.15\theta_y$	0.452	$6.15\theta_y$	$8.41\theta_y$
W24x104	$4.76\theta_y$	$6.76\theta_y$	0.260	$3.76\theta_y$	$5.06\theta_y$
W24x117	$7.52\theta_y$	$9.52\theta_y$	0.482	$6.52\theta_y$	$8.93\theta_y$



**Table E-6. Fully Restrained Connection Hinge Properties**

Beam	Depth	Plastic Rotation Angle		Residual Strength Ratio	Acceptance Criteria
		a	b		
	(in)	(rad)	(rad)		(rad)
		(0.021-0.0003D)	(0.05-0.0006D)		(0.021-0.0003D)
W24x55	23.6	0.01392	0.03584	0.2	0.01392
W24x62	23.7	0.01389	0.03578	0.2	0.01389
W24x68	23.7	0.01389	0.03578	0.2	0.01389
W24x76	23.9	0.01383	0.03566	0.2	0.01383
W24x84	24.1	0.01377	0.03554	0.2	0.01377
W24x94	24.3	0.01371	0.03542	0.2	0.01371
W24x104	24.1	0.01377	0.03554	0.2	0.01377
W24x117	24.3	0.01371	0.03542	0.2	0.01371
W24x131	24.5	0.01365	0.03530	0.2	0.01365

**Table E-7. Shear Tab Partially Restrained Connection Hinge Properties**

Beam	D <sub>bg</sub>	V <sub>max</sub>	M <sub>capacity</sub>	Plastic Rotation Angle		Residual Strength Ratio	Acceptance Criteria	
				a	b		Primary	Secondary
	(in)	(kips)	(kip-in)	(rad)	(rad)		(rad)	(rad)
				(0.0502-0.0015D)	(0.072-0.0022D)		(0.0502-0.0015D)	(0.0503-0.0011D)
W16x31	6	56	168	0.0412	0.0588	0.2	0.0412	0.0437
W16x31 (R)	3	32	96	0.0457	0.0654	0.2	0.0457	0.0470
W21x44	6	67	201	0.0412	0.0588	0.2	0.0412	0.0437
W21x44 (R)	3	32	96	0.0457	0.0654	0.2	0.0457	0.0470
W24x55 (R)	3	32	96	0.0457	0.0654	0.2	0.0457	0.0470
W24x62	12	126	378	0.0322	0.0456	0.2	0.0322	0.0371
W24x62 (R)	3	32	96	0.0457	0.0654	0.2	0.0457	0.0470
W24x68	3	55	165	0.0457	0.0654	0.2	0.0457	0.0470
W24x84	3	32	96	0.0457	0.0654	0.2	0.0457	0.0470
W24x94	3	55	165	0.0457	0.0654	0.2	0.0457	0.0470
W24x104	3	55	165	0.0457	0.0654	0.2	0.0457	0.0470

Column hinge properties are determined using ASCE 41 Table 9-6. These hinge definitions are dependent on the level of axial load present in the member. Because the hinge properties are based on the level of force present, they must be updated when the force level changes significantly. The initial model run was used for preliminary definitions of column hinges. A summary of hinge properties and acceptance criteria for columns used in this example corresponding to Figure E-18 are shown in Tables E-8, E-9 and E-10.

**Table E-8. Column Hinge Properties for Removal of Column 1**

Column	P/P <sub>cl</sub>	Plastic Rotation Angle		Residual Strength Ratio	Acceptance Criteria
		A	b		
		(rad)	(rad)	c	Primary (rad)
W18x40	0.12	0.0504	0.0623	0.557	0.0332
W18x40	0.42	0.0118	0.0189	0.2	0.0159
W18x76	0.09	0.0425	0.0554	0.406	0.0262
W18x76	0.12	0.0352	0.0471	0.349	0.0209
W18x76	0.19	0.0323	0.0433	0.349	0.0192
W18x76	0.21	0.0278	0.0423	0.2	0.0121
W18x76	0.38	0.0169	0.0257	0.2	0.0059
W18x86	0.17	0.0474	0.0586	0.557	0.0312
W18x106	0.24	0.0331	0.0511	0.2	0.0241
W18x106	0.27	0.0296	0.0458	0.2	0.0216
W18x106	0.35	0.0197	0.0305	0.2	0.0144
W18x106	0.39	0.0158	0.0244	0.2	0.0115

**Table E-9. Column Hinge Properties for Removal of Column 2**

Column	P/P <sub>cl</sub>	Plastic Rotation Angle		Residual Strength Ratio	Acceptance Criteria
		A	b	c	Primary
		(rad)	(rad)		(rad)
W18x50	0.05	0.0530	0.0664	0.512	0.0470
W18x50	0.15	0.0471	0.0591	0.512	0.0418
W18x50	0.19	0.0451	0.0565	0.512	0.0400
W18x60	0.06	0.0590	0.0721	0.6	0.0393
W18x60	0.13	0.0545	0.0667	0.6	0.0364
W18x60	0.27	0.0252	0.0384	0.2	0.0117
W18x60	0.40	0.0160	0.0244	0.2	0.0063
W18x76	0.04	0.0384	0.0515	0.349	0.0229
W18x76	0.06	0.0376	0.0504	0.349	0.0224
W18x76	0.19	0.0323	0.0433	0.349	0.0192
W18x76	0.27	0.0236	0.0359	0.2	0.0097
W18x86	0.09	0.0521	0.0644	0.557	0.0343
W18x86	0.11	0.0509	0.0629	0.557	0.0335
W18x86	0.13	0.0500	0.0618	0.557	0.0329
W18x97	0.17	0.0499	0.0610	0.6	0.0333
W18x97	0.23	0.0345	0.0533	0.2	0.0251
W18x97	0.25	0.0322	0.0498	0.2	0.0234
W18x97	0.37	0.0182	0.0281	0.2	0.0132
W18x106	0.24	0.0333	0.0514	0.2	0.0242
W18x106	0.34	0.0208	0.0321	0.2	0.0151
W18x106	0.35	0.0198	0.0305	0.2	0.0144
W18x106	0.49	0.0067	0.0104	0.2	0.0049

**Table E-10. Column Hinge Properties for Removal of Column 3**

Column	P/Pcl	Plastic Rotation Angle		Residual Strength Ratio	Acceptance Criteria
		a	b	c	Primary
		(rad)	(rad)		(rad)
W18x50	0.05	0.0530	0.0664	0.512	0.0470
W18x50	0.14	0.0477	0.0598	0.512	0.0423
W18x50	0.19	0.0451	0.0566	0.512	0.0400
W18x50	0.41	0.0157	0.0239	0.2	0.0060
W18x60	0.06	0.0590	0.0722	0.6	0.0394
W18x60	0.14	0.0537	0.0657	0.6	0.0358
W18x60	0.27	0.0252	0.0384	0.2	0.0117
W18x60	0.40	0.0161	0.0245	0.2	0.0063
W18x86	0.11	0.0508	0.0628	0.557	0.0334
W18x86	0.16	0.0480	0.0593	0.557	0.0316
W18x86	0.23	0.0334	0.0515	0.2	0.0229
W18x86	0.35	0.0202	0.0311	0.2	0.0135
W18x97	0.18	0.0498	0.0608	0.6	0.0332
W18x97	0.24	0.0340	0.0526	0.2	0.0248
W18x97	0.35	0.0202	0.0312	0.2	0.0147

#### E-4.2.9 Iterate Dynamic Analysis.

It is important to check that both stages of every analysis case converge. If the analysis does not converge, there is a problem with the model and it must be fixed. The problem could be numerical with assumptions made in SAP, but the most likely reason is that the model has a plastic hinge that failed or a mechanism has formed. At this point, the model cannot support the load. If the analysis fails to converge,

- 1) Since the analysis did not converge, members must be redesigned. To determine which members must be redesigned, step through the incomplete progression of plastic hinge formations. The final steps saved by SAP will often give the best results on which beams or columns to redesign. See Figure E-18 for the final step in the hinge formations of an analysis that did not converge.
- 2) To view the plastic hinges, click Display – Deformed Shape. Choose an analysis case and click to the last step of that case. Any hinge that forms will “light up,” and its color denotes the region the hinge has progressed (see Figure E-17). A hinge deformed in excess of the limit defined by its acceptance criteria, or is orange or red has failed.

3) Once members have been selected to be redesigned, re-run the analysis. Repeat this process until the structure converges. The engineer must check each analysis case to make sure that no hinge has failed. Once the analysis converges and no hinges fail, perform a shear check on each member. See Figures E-19 through E-21 for final deformed shapes and hinge formations and Figures E-22 through E-24 for final member sizes for each column removal location.

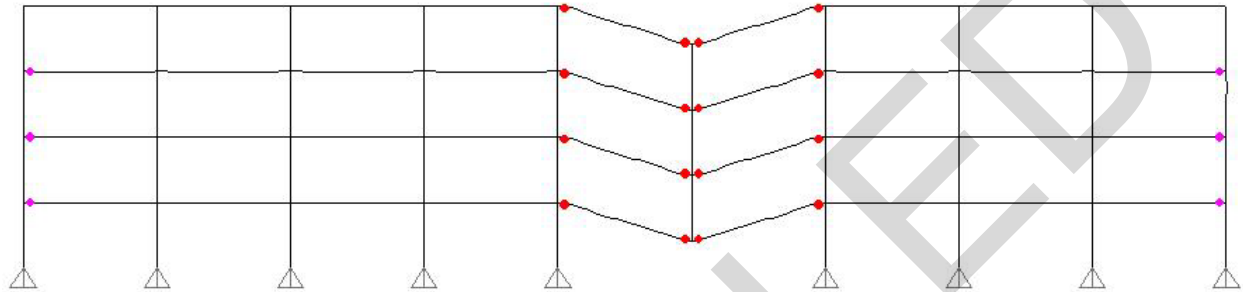


Figure E-19. Column 1 Removal Failed Convergence

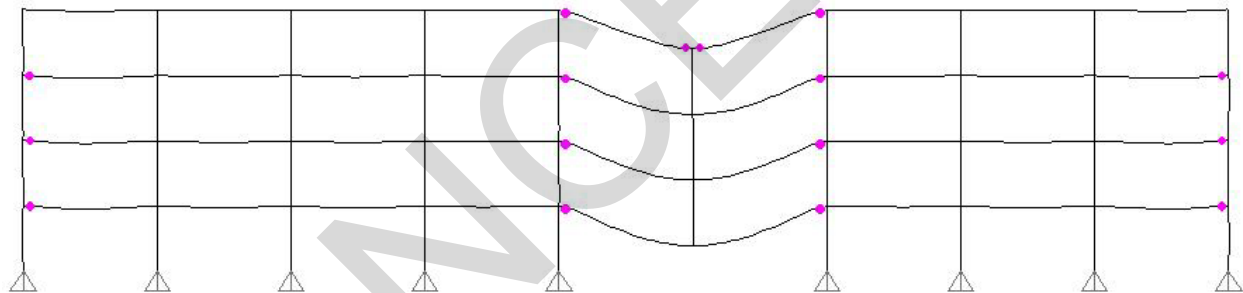


Figure E-20. Column 1 Removal Convergence After Redesign

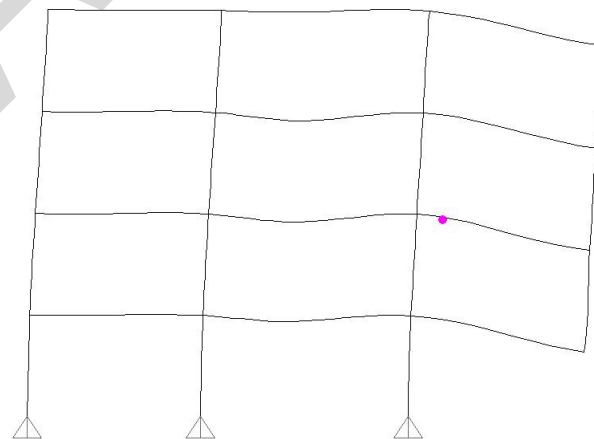


Figure E-21. Column 2 Removal Convergence After Redesign

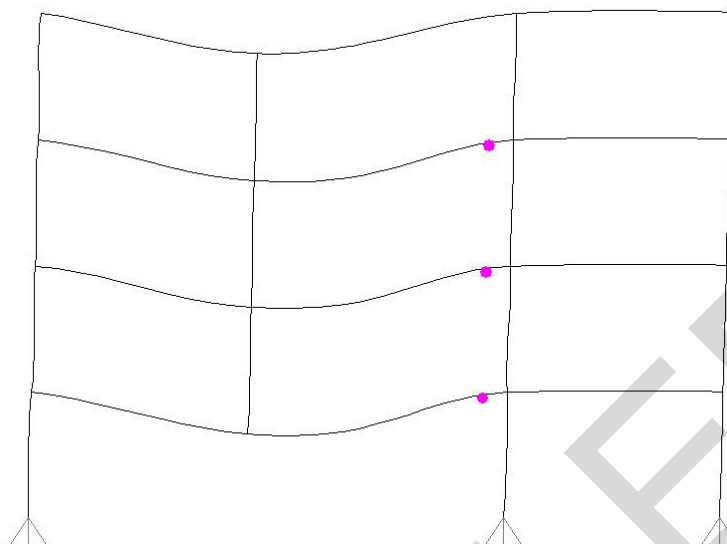


Figure E-22. Column 3 Removal Convergence After Redesign

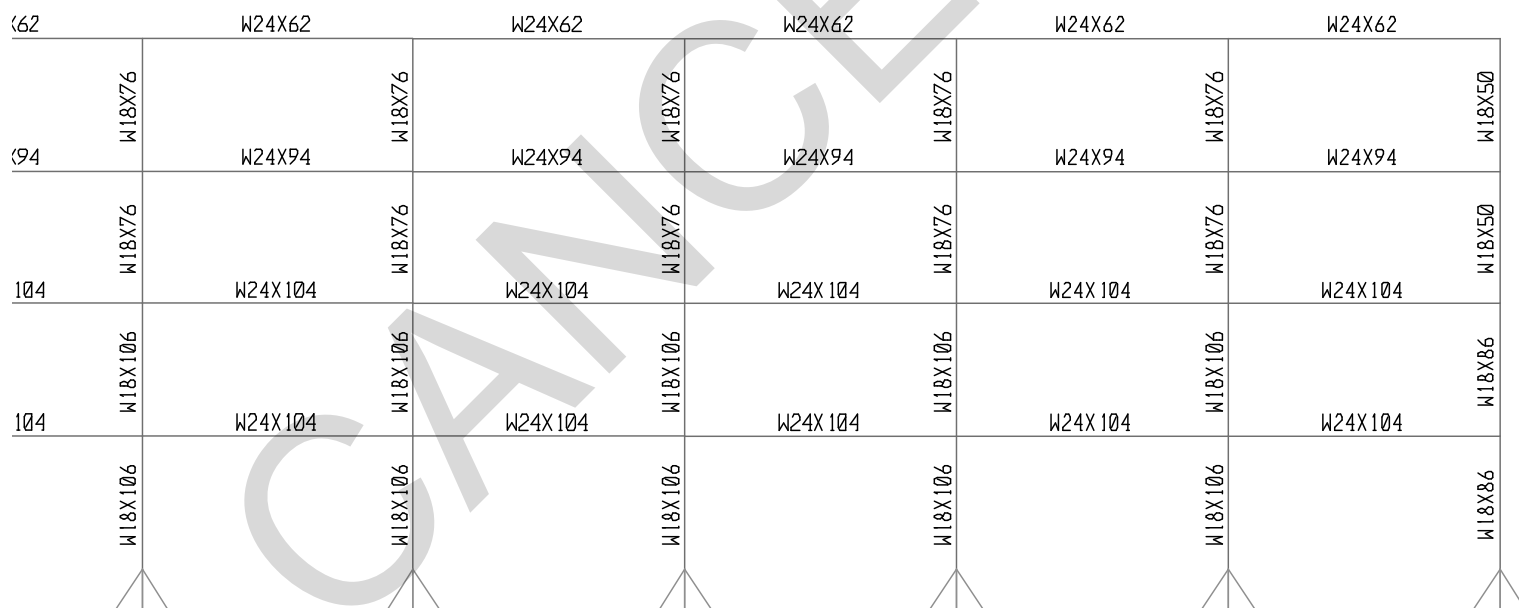
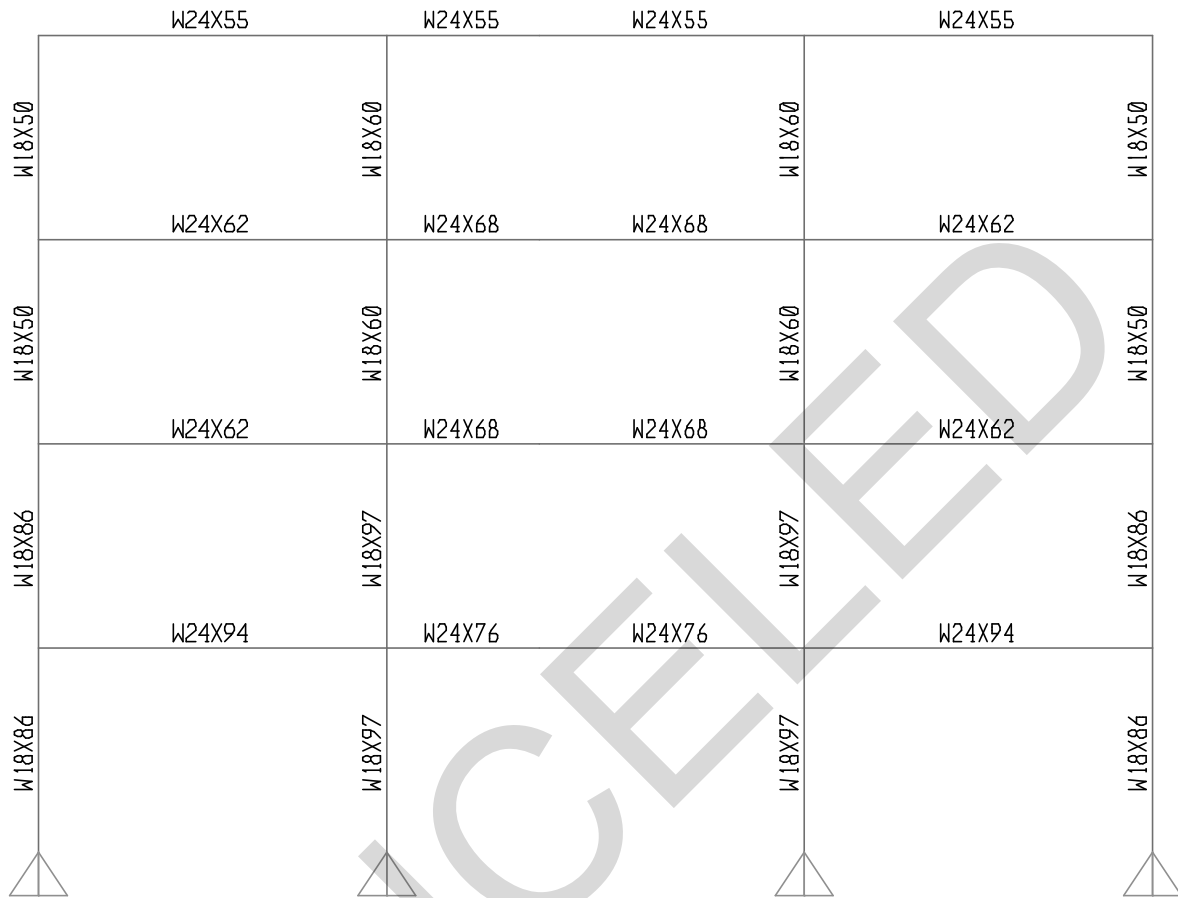


Figure E-23. Typical Member Sizes After Redesign for Column 1 Removal



**Figure E-24. Typical Member Sizes After Redesign for Column 2 or 3 Removal**

#### **E-4.2.10 Secondary Component Checks.**

Because the gravity beams were explicitly included in the model, there are no secondary components to check. As with the Linear Static solution, the slab is adequate due to the small deformations and by engineering judgment.

#### **E-4.3 Enhanced Local Resistance.**

The provisions for Risk Category III require that all perimeter columns at the first floor above grade achieve design shear strengths that exceed the shear demand associated with the flexural demand of the columns. For Risk Category III, the nominal flexural strength is determined from the building design after it meets the AP requirements. The ELR design process for the Nonlinear Dynamic solution is similar to that used for the Linear Static solution in Section E-3.5.



In this example, the design shear strength for all columns other than corner columns exceeds the shear demand so no member size increases were required

beyond those determined using the alternate path procedure. For corner columns, there is no W18 section that can provide adequate design shear strength to meet the shear demand. In order to satisfy these provisions, web doubler plates were added. At these corner locations the axial load is less than 10% of the column capacity and was ignored.

The corner column is a W18x86 with a strong axis plastic modulus of 186 in<sup>3</sup>. The shear demand is calculated with Equation E-1 and using an over-strength factor of 1.1 (per Section 3-3.1) is

$$V_u = 7.5 M_p / L = 7.5 * 1.1 * 50 \text{ ksi} * 186 \text{ in}^3 / 174 \text{ in} = 441 \text{ kips}$$

The design shear strength (per AISC Chapter G) is:

$$\Phi V_n = 1.0 * 0.6 * t_w * h * F_y = 0.6 * 0.480 \text{ in} * 18.4 \text{ in} * 50 \text{ ksi} = 265 \text{ kips}$$

The required additional design shear strength is achieved by adding a doubler plate to the web of the W18x86. The thickness is found by subtracting the design shear strength (265-kips) from the shear demand (441-kips) and using this difference to size a 14-in high plate. A 7/16" thick doubler plate is required for all corner columns. See Figure E-13 for a diagram of the doubler plate used to enhance strong axis shear strength.

#### **E-4.4 Alternate Path Design Method Complete.**

Once the model converges, all acceptance criteria have been met, and enhanced local resistance has been provided, the building has satisfied progressive collapse resistance requirements of this UFC.

#### **E-5 RESULTS COMPARISON.**

Member size requirements determined from the linear static and nonlinear dynamic procedures are presented and compared to original size requirements in Tables E-11 and E-12.

**Table E-11. Moment Frame Size Comparison**

Location	Level	Original Size	LSP AP Size	NDP AP Size
Long Side	2	W24x68	W24x146	W24x104
Long Side	3	W24x68	W24x146	W24x104
Long Side	4	W24x68	W24x117	W24x94
Long Side	Roof	W24x55	W24x76	W24x62
Short Side – Exterior	2	W24x94	W24x94	W24x94
Short Side – Exterior	3	W24x62	W24x68	W24x62
Short Side – Exterior	4	W24x62	W24x68	W24x62
Short Side – Exterior	Roof	W24x55	W24x55	W24x55
Short Side – Central	2	W24x76	W24x104	W24x76
Short Side – Central	3	W24x68	W24x104	W24x68
Short Side – Central	4	W24x68	W24x94	W24x68
Short Side – Central	Roof	W24x55	W24x55	W24x55

**Table E-12. Perimeter Column Size Comparison**

Location	Level	Original Size	LSP AP Size	NDP AP Size
Long Side – Interior	1-3	W18x86	W18x175	W18x106
Long Side – Interior	3-Roof	W18x55	W18x106	W18x76
Short Side – Interior	1-3	W18x97	W18x119	W18x97
Short Side – Interior	3-Roof	W18x60	W18x76	W18x60
Corner	1-3	W18x86	W18x97	W18x86
Corner	3-Roof	W18x40	W18x76	W18x50

The frame size increases required for progressive collapse resistance are summarized by total weight in Table E-13. The weight of the exterior moment frames (girders and columns) are reported for the long side and short side for the baseline design, and after providing progressive collapse resistance using each of the linear static and nonlinear dynamic procedures.

**Table E-13. Frame Weight Comparison**

Frame	Original Weight (tons)	LSP AP Weight (tons)	NDP AP Weight (tons)
Long Side	58.2	108.7	78.2
Short Side	20.1	24.5	20.4

## APPENDIX F WOOD EXAMPLE

### F-1 INTRODUCTION

A wood load-bearing wall structure example has been prepared to illustrate Alternate Path analysis of a bearing wall type structure. The structure is assumed to have an occupancy of less than 100 people and is classified as \3\ Risk Category /3/ II per UFC 3-301-01.

The example has been prepared using tools and techniques commonly applied by structural engineering firms in the US. For wood design, this example relies primarily upon hand analysis supplemented with standard design tables published by various wood organizations. \3\ Risk Category /3/ II Option 2: Alternate Path has been selected from UFC Table 2-2 to evaluate resistance to progressive collapse.

### F-2 BASELINE DESIGN

The example building is a 3-story wood structure, six bays long and two bays deep. The relevant design information is shown in Figures F-1 through F-4.

Platform construction is used. The internal shear walls are load bearing, with engineered I-joists (EIJ) running in the direction of the long length of the structure. One 4.5-foot x 6-foot window opening is present in each room. The footings are reinforced concrete with masonry. There are no internal partition walls.

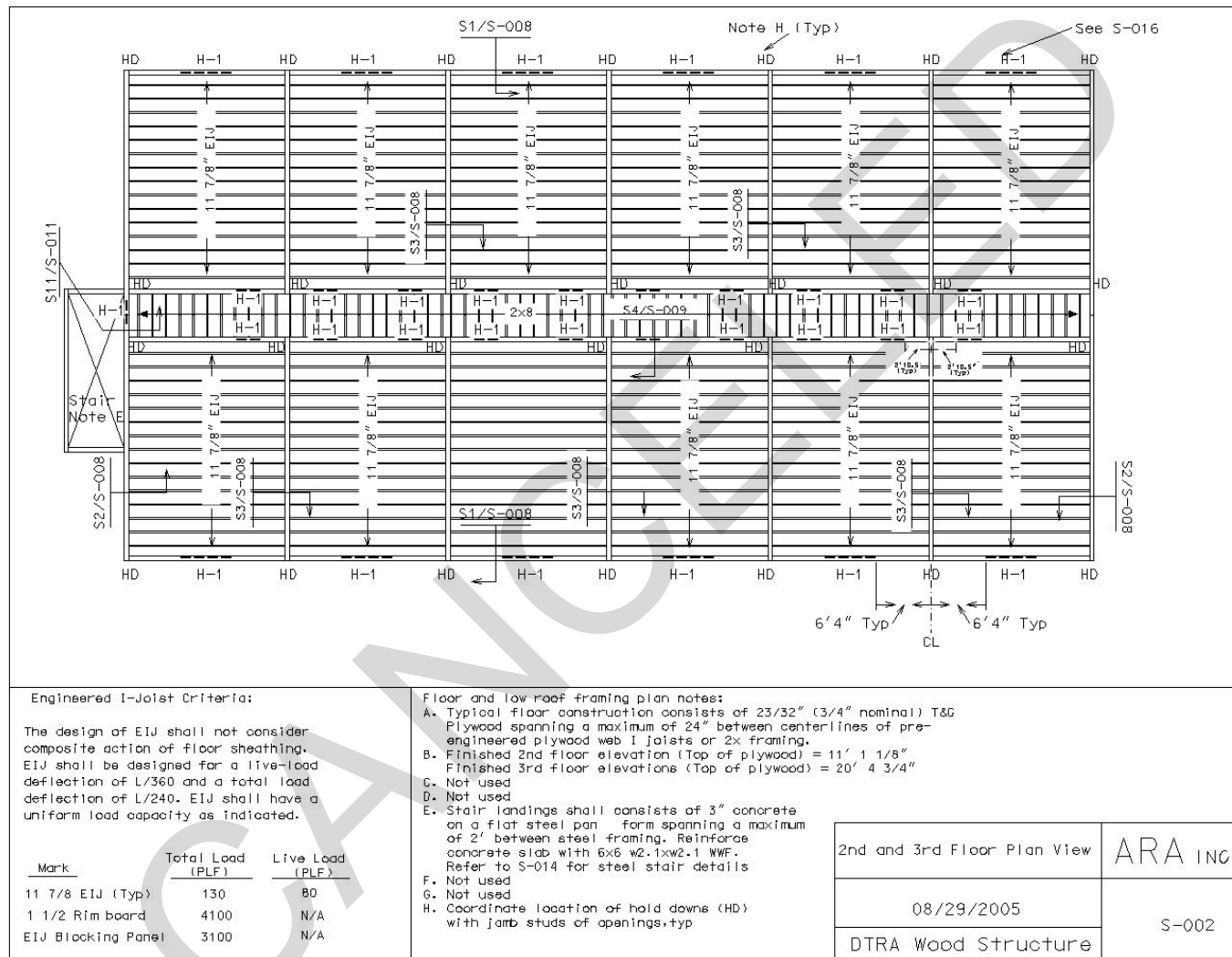
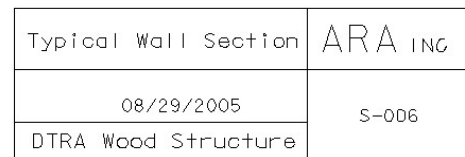


Figure F-1. Wood Frame Example Plan



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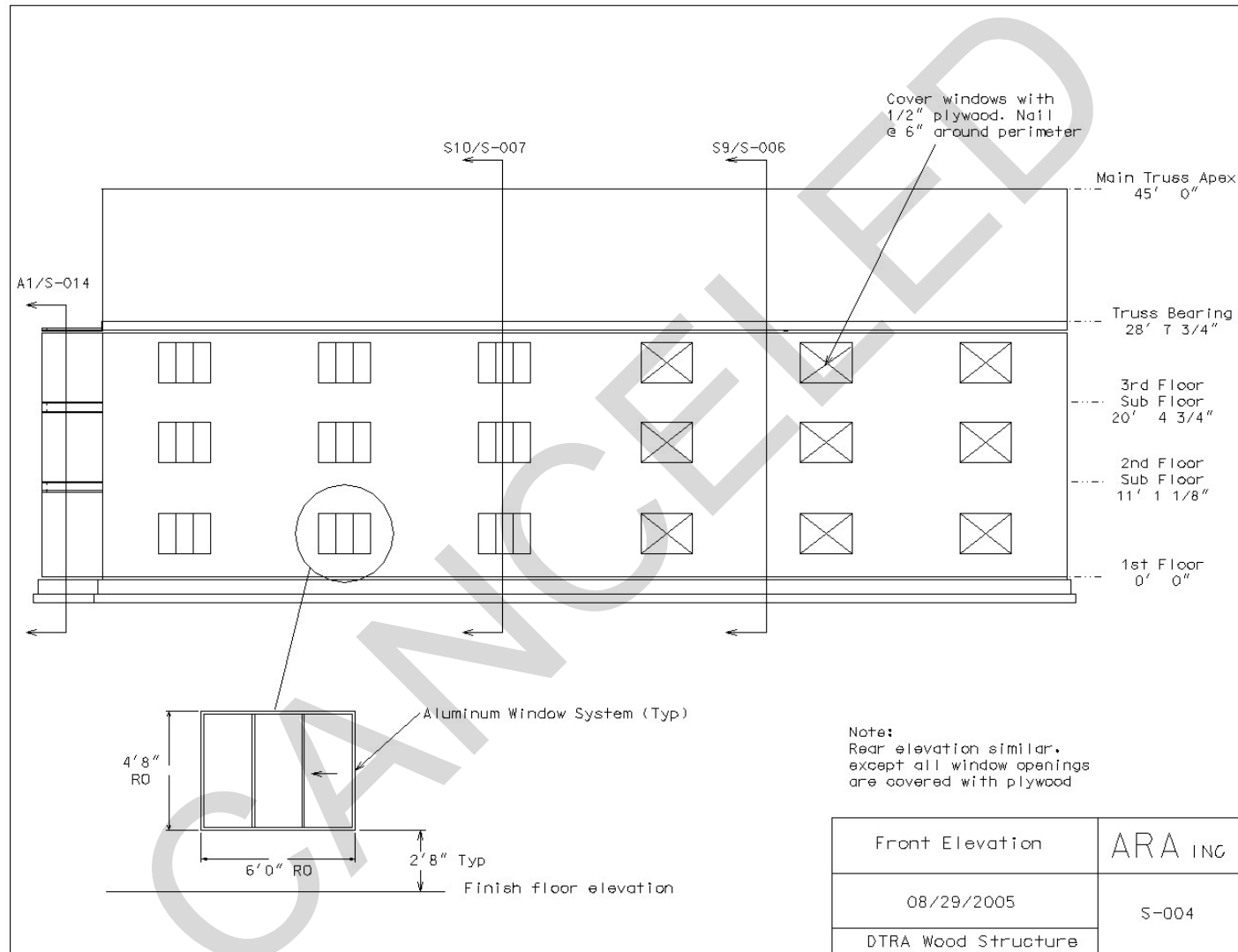
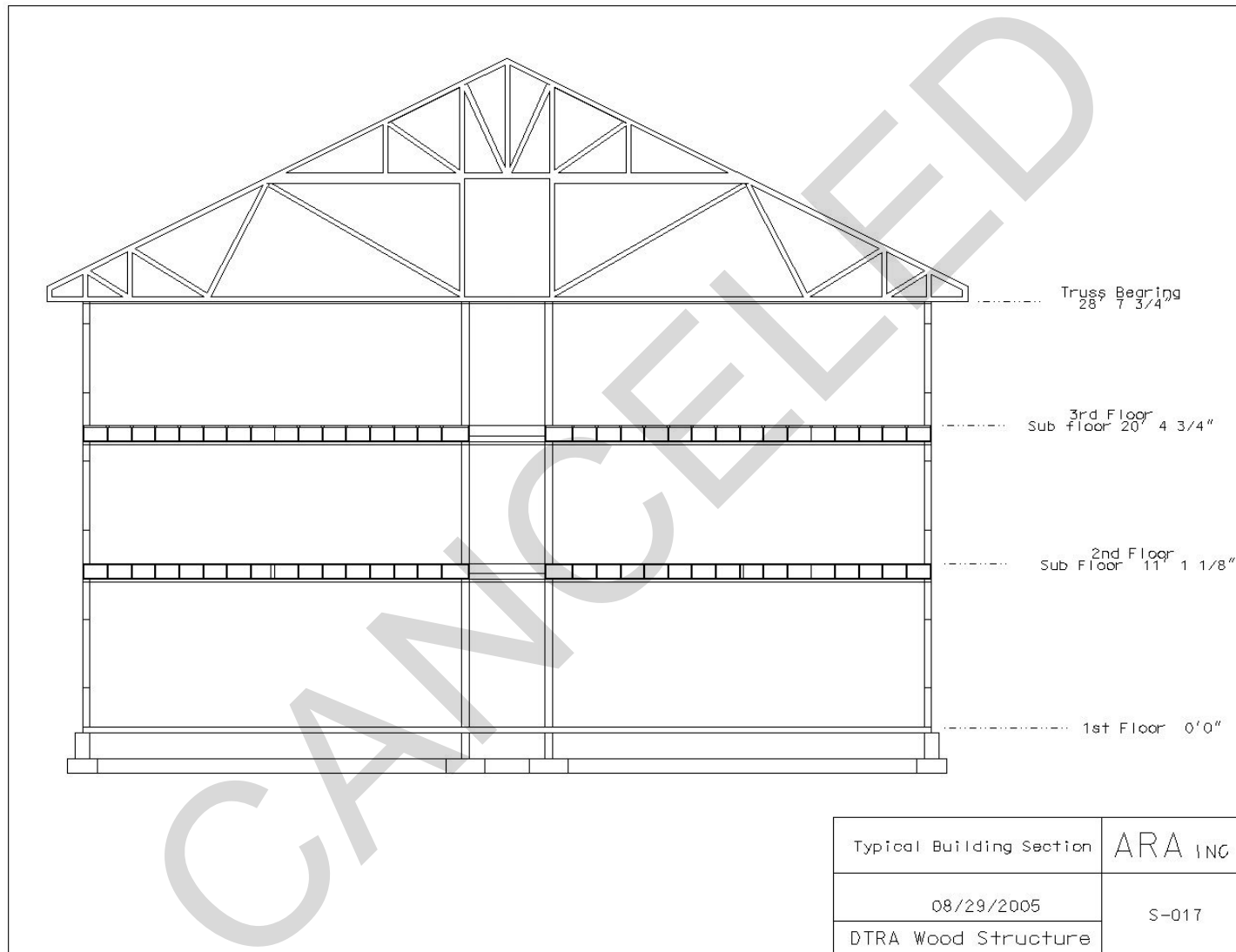


Figure F-3. Wood Frame Example Exterior Wall Elevation



**Figure F-4. Wood Frame Example Building Section**

## **F-2.1 Construction and Materials**

### **Wall Construction:**

2x6 at 24 inches on center

Exterior Walls: 19/32" Plywood Sheathing Exterior Face, blocked; 5/8" Gypsum Wallboard, unblocked Interior Face

Interior Walls: 19/32" Plywood Sheathing both Faces, blocked

### **Floor Construction:**

Engineered I-joists (EIJ) at 24 inches on center with 0.75-inch tongue and groove plywood sub-floor. Bottom surface 5/8" gypsum wallboard with a 1/2" channel separation for noise and fire suppression.

### **Roof Construction:**

Engineered Trusses with 1/2" Plywood Sheathing

### **Material Grades:**

Plywood: Voluntary Product Standard PS 1-07 Exposure 1; APA Rated

Wood Framing: Produced to American Softwood Lumber Standard Voluntary Product Standard PS 20.

- 19% maximum moisture content
- Grade 2 or better
- Southern Pine (SPIB)

## **F-2.2 Loading Assumptions**

Typical loading relevant to AP Analysis:

Floors:

Dead Load: 25 psf including ceiling

Live Load: 40 psf

Roof:

Dead Load: 25 psf including roofing and bottom surface ceiling

Live Load: 20 psf

Snow Load: 7 psf (design, including all applicable ASCE 7-05 factors)

Walls:

Dead Load: 7 psf including sills and headers

Live loads reducible depending on tributary area considered.



### **F-2.3 Relevant Standards and Reference Documents**

ANSI/AF&PA NDS-2005 National Design Specification for Wood Construction  
ASD/LRFD

ANSI/AF&PA SDPWS-2005 ASD/LRFD Special Design Provisions for Wind and  
Seismic With Commentary

International Building Code 2006

For design compatible with the UFC document LRFD is followed.

### **F-3 ALTERNATE PATH ANALYSIS**

An alternate path (AP) analysis is conducted according to the requirements of Section 3-2. Removal scenarios are defined by 3-2.9.1 and 3-2.9.2. To bridge the wall removals, the remaining load bearing walls will be utilized as shear wall elements. The linear static AP analysis method will be followed.

#### **F-3.1 Scope and Analysis Assumptions**

Four scenarios of wall removal will be evaluated:

1. Removal of interior load-bearing wall (see Figure F-5) at first story
2. Removal of exterior long walls (see Figure F-6) at second story
3. Removal of exterior long walls (see Figure F-8) at second story (alternate location)
4. Removal of exterior long walls (see Figure F-9) at third story

General assumptions of the analysis are as follows:

- No wind or internal pressure acts on interior or exterior walls during wall removal scenario
- Contributions of EIJ and engineered roof trusses to the alternate path capacity are neglected due to uncertainty in specific properties during the design phase. At the designer's option, these elements could be included, provided that appropriate performance specifications including AP requirements are incorporated into the contract documents.

Per ASCE 41-13  
C12.2.2.5:

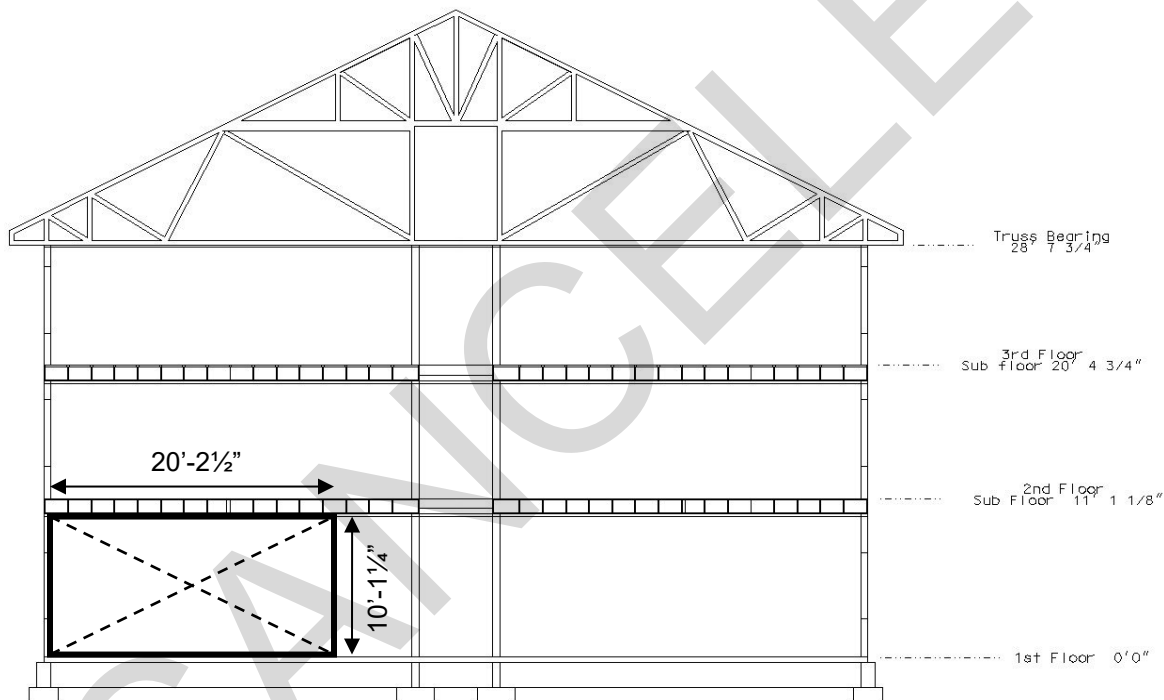
"Actions associated with wood and light metal framing components generally are deformation controlled, and expected strength material properties will be used most often."

8.3.3:

“Demands on connectors, including nails, screws, lags, bolts, split rings, and shear plates used to link wood components to other wood or metal components shall be considered deformation controlled actions. Demands on bodies of connections, and bodies of connection hardware, shall be considered force-controlled actions.”

For the AP analysis of this structure, wood shear wall assemblies and primary connections will be taken to be deformation controlled elements. Bodies of connectors will be checked as force controlled elements as required.

**F-3.2 AP Analysis of Interior Load Bearing Wall Removal**



**Figure F-5. Interior Load Bearing Wall Removal**

As shown in Figure F-5, a segment of wall of length  $2H$  is removed from the lower floor interior load bearing wall.  $H$  is taken as the distance between the 1<sup>st</sup> floor sub-floor elevation and the bottom of the 11'-7/8" EIJ floor joists at the 2<sup>nd</sup> floor ( $=10'-1\frac{1}{4}"$ ).

The remaining structure creates a shear wall element that must span the floor and wall loads from the 2<sup>nd</sup> and 3<sup>rd</sup> stories. The shear wall is taken to have an overall depth of 17'-6 5/8" (2<sup>nd</sup> floor to truss bearing). The boundary (chord) elements consist of (2) 2x6 plates provided at the top and bottom of each wall.

To bridge the removed section of wall, the shear wall panel must span from the exterior wall to a wall pier segment consisting of the remaining portion of the removed lower story interior wall. Note that no continuity action that might engage the right-hand wall panels in Figure F-5 is possible due to the corridor opening.

The structure contains no irregularities and therefore the linear static AP analysis is permitted per Section 3-2.11.1.1.

Loading:

Dead Load

Floors: 25 psf \* 18'-8" \* 2 floors = 934 plf

Wall: 7 psf \* 18'-6 1/2" = 130 plf

Live Load

40 psf \* 18.67 \* 2 floors = 1494 plf

Reduce live load in accordance with IBC 2006 1607.9.2:

A = 18.67' \* 20.21' \* 2 = 755 SF

R = 0.08 (A - 150) = 0.08 (755 - 150) = 48.4%

R<sub>max</sub> = 60% or 23.1(1 + 25/40) = 37.5%

Live load used in AP check = 1494 plf (1 - 0.375) = 934 plf

Linear Static AP Load Case for Deformation Controlled Elements:

$$G_{LD} = \Omega_{LD} [1.2D + 0.5L]$$

From ASCE 41-13 Table 12-3:

for "Wood Structural Panel Sheathing or Siding"

h/b = 17.55/20.21 = 0.87 < 2.0

m = 3.8 for Life Safety Primary Element

Assume all connections to be nailed:

m = 6.0 for "Nails – Wood to Wood"

m = 4.0 for "Nails – Wood to Metal"

m = 3.8 controls

$\Omega_{LD}$  = 2.0m = 7.6 (UFC Table 3-4)

$G_{LD}$  = 7.6\*(1.2\*(934+130) + 0.5\*(934)) = 7.6\*(1744 plf) = 13,254 plf

Deformation controlled actions:

Shear:  $Q_{UD} = 13,254 \text{ plf} * 20.21' / 2 / 17.55' = 7,631 \text{ plf}$  in wall

Capacity checks of deformation controlled actions:

Shear in Wall:

Wall shear strength  $\phi Q_{CE} = 1.5 (\phi V_s)$

where  $\phi V_s$  = LRFD shear strength of wall taken from ANSI/AFPA SDPWS-2005 and factor 1.5 for expected strength is taken from ASCE 41-13 12.4.9.2, but note that Section 3-2.11.7.1 requires inclusion of the material specific  $\phi$  factor whereas ASCE 41-13 uses a  $\phi$  of 1.0.

From ANSI/AFPA SDPWS-2005 Table 4.3A:

For "Wood Structural Panels – Sheathing", 19/32 with 10d nailing at 6" at edges:

$V_{wc} = 950 \text{ plf}$  (for one side of wall)

Note that value for wind is chosen rather than seismic per UFC Ch. 7.

Since the interior walls have plywood sheathing on both sides, strength of each individual side is additive per ANSI/AFPA SDPWS-2005 4.3.3.2.

$\phi = 0.8$  for LRFD per ANSI/AFPA SDPWS-2005 4.3.3

$\phi V_s = 0.8 (950) (2) = 1520 \text{ plf}$

$\phi Q_{CE} = 1.5 (\phi V_s) = 1.5 (1520) = 2280 \text{ plf}$

Check UFC Equation 3-13:

$\phi m Q_{CE} \geq Q_{UD}$

$2280 \text{ plf} (3.8) = 8,664 \text{ plf} > 7,631 \text{ plf}$  **OK**

At the edge of the removed wall segment adjacent to the building exterior, the interior load bearing wall must transfer its reaction in shear to the exterior wall for distribution to an adequate length of exterior load bearing stud wall.

Exterior Wall Check:

$Q_{UD} = 7,631 \text{ plf} / 2 = 3,816 \text{ plf}$  (half of reaction distributed each direction)

From ANSI/AFPA SDPWS-2005 Table 4.3A:

Exterior sheathing plywood 19/32 with 10d nailing at 6" at edges:

$$\begin{aligned}V_{wc} &= 950 \text{ plf} \\ \phi V_s &= 0.8 (950) = 760 \text{ plf} \\ \phi Q_{CE} &= 1.5 (760) = 1140 \text{ plf}\end{aligned}$$

From ASCE 41-13 Table 12-3  $m = 4.7$  for Gypsum Wallboard with  $h/b \leq 1.0$ . Assuming the shear reaction is spread over a length  $b$  less than 17.55', the  $m$  factor for wood structural panels controls.

Check UFC Equation 3-13:

$$\begin{aligned}\phi m Q_{CE} &\geq Q_{UD} \\ 1140 (3.8) &= 4,322 \text{ plf} > 3,816 \text{ plf} \quad \mathbf{OK}\end{aligned}$$

For force controlled actions:

$$\Omega_{LD} = 2.0 \text{ (UFC Table 3-4)}$$

$$G_{LF} = 2.0 * (1.2 * (934 + 130) + 0.5 * (934)) = 2.0 * (1744 \text{ plf}) = 3,488 \text{ plf}$$

Force controlled actions:

$$\begin{aligned}\text{Shear: } Q_{UF} &= 3,488 \text{ plf} * 20.21' / 2 / 17.55' = 2,008 \text{ plf in wall} \\ \text{Chord Force: } Q_{UF} &= 3,488 \text{ plf} * 20.21^2 / (8 * 17.55) = 10,147 \text{ pounds}\end{aligned}$$

Though ASCE 41 is somewhat ambiguous on the subject, consider the chord forces to be deformation controlled actions because ASCE 41 Table 12-3 designates "Frame Components Subject to Axial Compression" as force controlled. Treating both tension and compression chords as force controlled will be conservative.

From the NDS supplement for 2x6 #2 Southern Pine:

$$\begin{aligned}F_t &= 825 \text{ psi} \\ F_c &= 1650 \text{ psi} \\ F_{c\perp} &= 565 \text{ psi} \\ E_{min} &= 580,000 \text{ psi}\end{aligned}$$

Chord force limit will be governed by tension capacity since compression chord is fully braced in both directions:

$$\text{Adjusted reference design value } F'_t = C_M C_t C_F C_i K_F \phi_t \lambda F_t$$

All adjustment values are 1.0 except:

$$\begin{aligned}K_F &= 2.16 / \phi_t \\ \phi_t &= 0.80\end{aligned}$$

Note  $\lambda = 1.0$  for short duration loading event

$$F'_t = 2.16 \times 825 = 1782 \text{ psi}$$

$$\phi Q_{CL} = 1782 \text{ psi} (8.25 \text{ in}^2) = 14,702 \text{ pounds for (1) } 2 \times 6 > Q_{UF} = 10,147 \text{ pounds}$$

Note that by providing adequate strength for (1) 2x6 plate to act as a chord, the pair of 2x6's may be stagger spliced via nailing without the need for a direct metal plate splice of both plates. Nailing for splices should be sized for 10,147 pounds.

Check bearing transfer of bridged load to load bearing stud walls as force controlled action.

$$Q_{UF} = 3,488 \text{ plf} \times 20.21' / 2 = 35,246 \text{ pounds}$$

2\ 2x6 studs at 24" on center with in-plane bracing provided by sheathing. Check as columns for out-of-plane strength at lower story./2/

Adjusted reference design value  $F'_c = C_M C_t C_F C_i C_P K_F \phi_c \lambda F_c$

All adjustment values are 1.0 except:

$$K_F = 2.16 / \phi_c$$

$$\phi_c = 0.90$$

Note  $\lambda = 1.0$  for short duration loading event

$C_P$  calculation:

$$F_{c*} = K_F \phi_c \lambda F_c = 2.16 \times 1650 = 3564 \text{ psi}$$

$$E'_{min} = C_M C_t C_i C_T K_F \phi_s E_{min}$$

All adjustment values are 1.0 except:

$$K_F = 1.5 / \phi_s$$

$$\phi_c = 0.85$$

$$E'_{min} = 580,000 \times 1.5 = 870,000 \text{ psi}$$

$$l_e = 11' - 1 \frac{1}{8}" - 4 \frac{1}{2}" = 10' - 8 \frac{5}{8}"$$

$$F_{cE} = 0.822 \times (870,000) / (128.625 / 5.5)^2 = 1308 \text{ psi} \quad (\text{NDS 3.7.1})$$

$$F_{cE} / F_{c*} = 1308 / 3564 = 0.367$$

$$C_p = 0.334 \text{ per NDS Equation 3.7-1}$$

$$\phi Q_{CL} = 0.334 (3564 \text{ psi}) (8.25 \text{ in}^2) = 9820 \text{ pounds per stud}$$

$$\phi Q_{CL} = 9820 / 2' = 4910 \text{ plf}$$

Check bearing on plates:

Adjusted reference design value  $F'_{c\perp} = C_M C_t C_i C_b K_F \phi_c \lambda F_{c\perp}$

All adjustment values are 1.0 except:

$$K_F = 1.875 / \phi_s$$

$$\begin{aligned}\phi_c &= 0.9 \\ C_b &= 1.25 \text{ for } 1.5'' \text{ wide member (NDS Table 3.10.4)} \\ F'_{c\perp} &= 1.25 * 1.875 * 565 \text{ psi} = 1324 \text{ psi}\end{aligned}$$

$$\phi Q_{CL} = (1324 \text{ psi})(8.25 \text{ in}^2) = 10,923 \text{ pounds per stud}$$

$$\phi Q_{CL} = 10,923/2' = 5461 \text{ plf}$$

$$\phi Q_{CL} = 4910 \text{ plf controls}$$

$$\text{Required width of wall} = 35,246 \text{ pounds} / 4910 \text{ plf} = 7'-3''$$

Adequate width available at external wall

$$\text{Available wall pier at interior wall} = 25'-11'' - 20'-2 \frac{1}{2}'' = 5'-8 \frac{1}{2}'' \text{ NG}$$

Note that location of the removed section of wall could shift such that the situation on the interior side is similar to that considered in Figure F-5 for the exterior side. Therefore, the reaction must be transferred by shear into the interior corridor transverse load bearing walls. The wall construction is identical so shear capacity will be adequate.

Removal of a wall segment above the 2<sup>nd</sup> floor would produce a very similar analysis and results to those described above as the load resisted is roughly proportional to the available depth of remaining wall.

Similarly, removal of an exterior load bearing wall along the short side would produce results consistent with the interior removal as the tributary area of floor carried is roughly half that of an interior wall, and the provided capacity consists of one face plywood and one face drywall, giving larger than half the capacity of the interior wall.

#### /2/Secondary Component Check

EIJ joists span 18'-8" and are supported by the short direction load bearing walls. Removal of a load bearing wall will result in loss of support for EIJs.

Accordingly, all EIJs must be attached to the wall above to prevent collapse in the event of a bearing wall removal. The floor joists shall be hung from the wall above using a tension tie nailed to the 2x6 wall stud above which will include a threaded tension rod bolted through the top plate of the removed wall (See Figure F-6). The top plate of the removed wall shall be assumed to be held in place due by the tension rod.

Connection of each EIJ will consist of multiple connectors so it shall be considered force controlled:

Floor loadings:

$$DL = 25 \text{ psf} * 18.67' * 2' = 934 \text{ lbs}$$

$$LL = 40 \text{ psf} * 18.67' * 2' = 1494 \text{ lbs} \quad (\text{no live load reduction due to limited tributary area for each EIJ tension tie})$$

$$\Omega_{LD} = 2.0 \text{ (UFC Table 3-4)}$$

$$G_{LF} = 2.0 * (1.2 * DL + 0.5 * LL)$$

$$G_{LF} = 2.0 * (1.2 * (934 \text{ lbs}) + 0.5 * (1494)) = 3735 \text{ lbs}$$

The tension tie capacity must be greater than 3735 lbs. Accordingly, provide one (1) Simpson HTT5 with 26-10dx1 1/2" nails into the wall stud above with a capacity of 4350 lbs.

The head plate of the wall below consists of (2) 2x6 southern pine plates.

The shear in the 2x6 head plates will be a force controlled action.

For each 2x6 head plate:

$$V' = 2/3 F_v' * A$$

$$F_v' = 175 \text{ psi}$$

$$A = 8.25 \text{ in}^2$$

$$F_v' = C_D C_M C_t C_i K_F \phi_v \lambda F_v$$

All adjustment factors 1.0 except:

$$K_F = 2.16 / \phi_b$$

$$\phi_b = 0.75$$

Note  $\lambda = 1.0$  for short duration loading event

$$F_v' = 2.16 / 0.75 * 0.75 * 175 \text{ psi} = 378 \text{ psi}$$

$$V' = 2/3 * 378 \text{ psi} * 8.25 \text{ in}^2 = 2079 \text{ lbs per head plate}$$

$$\phi Q_{CL} = 2079 \text{ lbs} * 2 = 4158 \text{ lbs} > 3735 \text{ lbs} \quad \text{OK}$$

The flexure in the 2x6 head plates will be a deformation controlled action.

From ASCE 41-13 Table 12-3:

For "frame components subject to axial tension and/or bending":

$m = 2.5$  for Life Safety Secondary Element



$$\Omega_{LD} = 2.0\text{m} = 5.0 \text{ (UFC Table 3-4)}$$

$$G_{LD} = 5.0 * (1.2 * (934 \text{ lbs}) + 0.5 * (1494 \text{ lbs})) = 5.0 * (1868 \text{ lbs}) = 9340 \text{ lbs}$$

Since the head plate will be part of a continuous span the maximum moment will be:

$$Q_{UD} = G_{LD} * L / 8 = 9340 \text{ lb} * 2 \text{ ft} / 8 = 2335 \text{ lb-ft}$$

Flexural capacity of each 2x6 head plate (weak axis bending):

$$M' = F_b' * S$$

$$S_y = 2.063 \text{ in}^3$$

$$F_b = 1250 \text{ psi}$$

$$F_b' = C_D C_M C_t C_L C_F C_{fu} C_i C_r K_F \phi_b \lambda F_b$$

All adjustment factors 1.0 except:

$$C_{fu} = 1.15$$

$$K_F = 2.16 / \phi_b$$

$$\phi_b = 0.85$$

Note  $\lambda = 1.0$  for short duration loading event

$$F_b' = 1.15 * 2.16 / 0.85 * 0.85 * 1250 \text{ psi} = 3105 \text{ psi}$$

$$M' = 3105 \text{ psi} * 2.063 \text{ in}^3 = 6406 \text{ lb-in} = 534 \text{ lb-ft}$$

$$\phi_m Q_{CE} \geq Q_{UD}$$

$$\phi_m Q_{CE} = 2.5 * 534 \text{ lb-ft} * 2 = 2670 \text{ lb-ft} > 2335 \text{ lb-ft} \quad \text{OK}$$

Since the shear and flexural capacity of both 2x6 head plates is required to support secondary members, a 3<sup>rd</sup> 2x6 head plate must be provided in order to accommodate splicing. All splices shall be staggered. See Figure F-6 for a floor joist support detail.

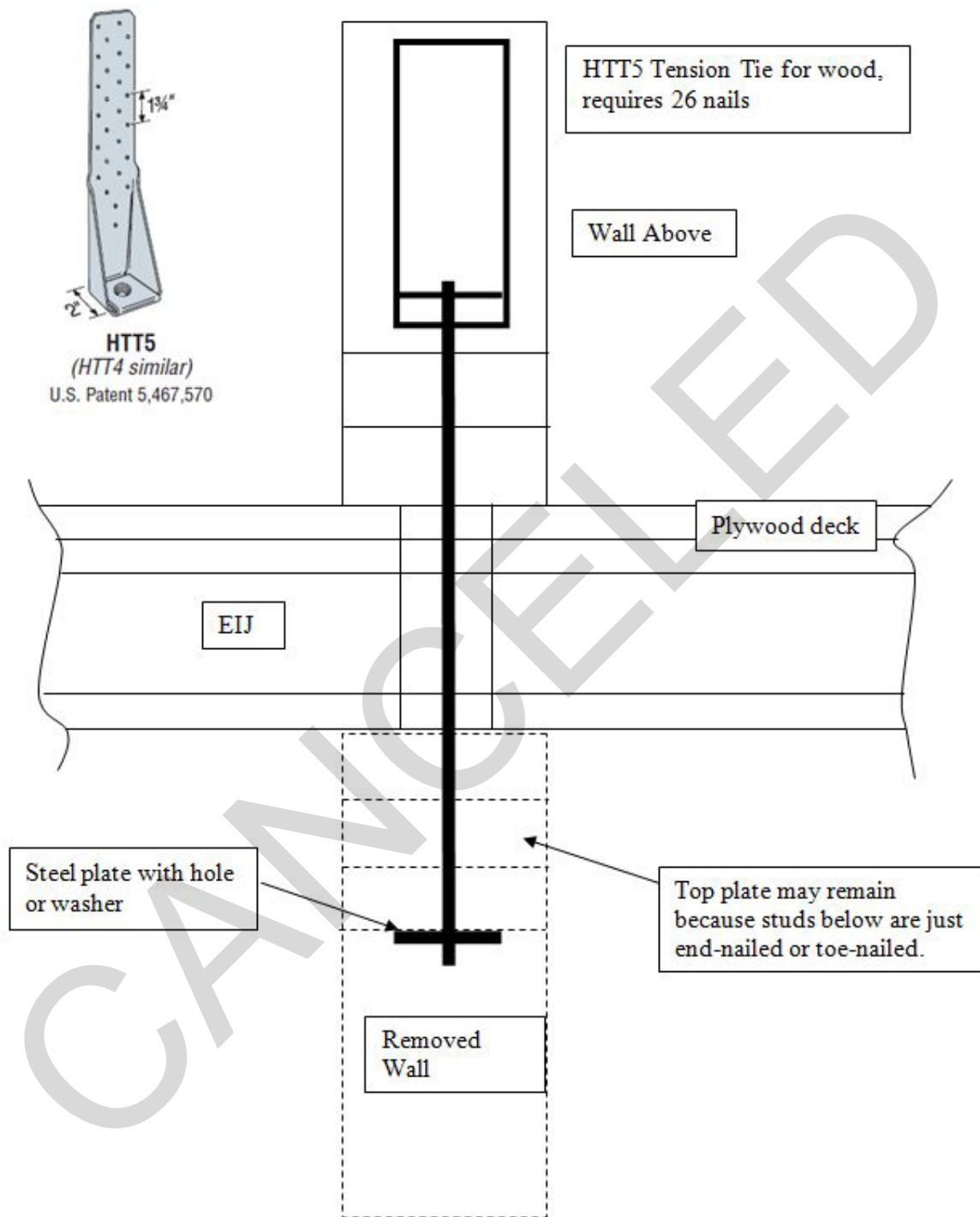
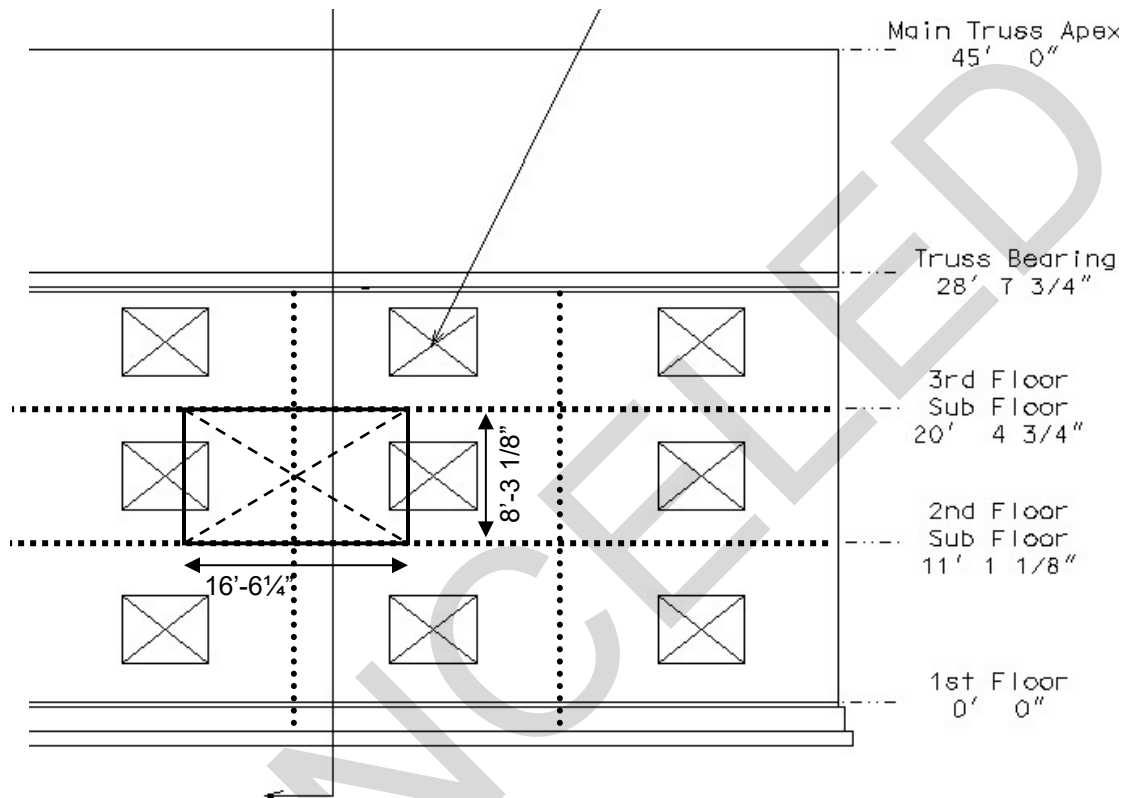


Figure F-6. Floor Joist Support Detail

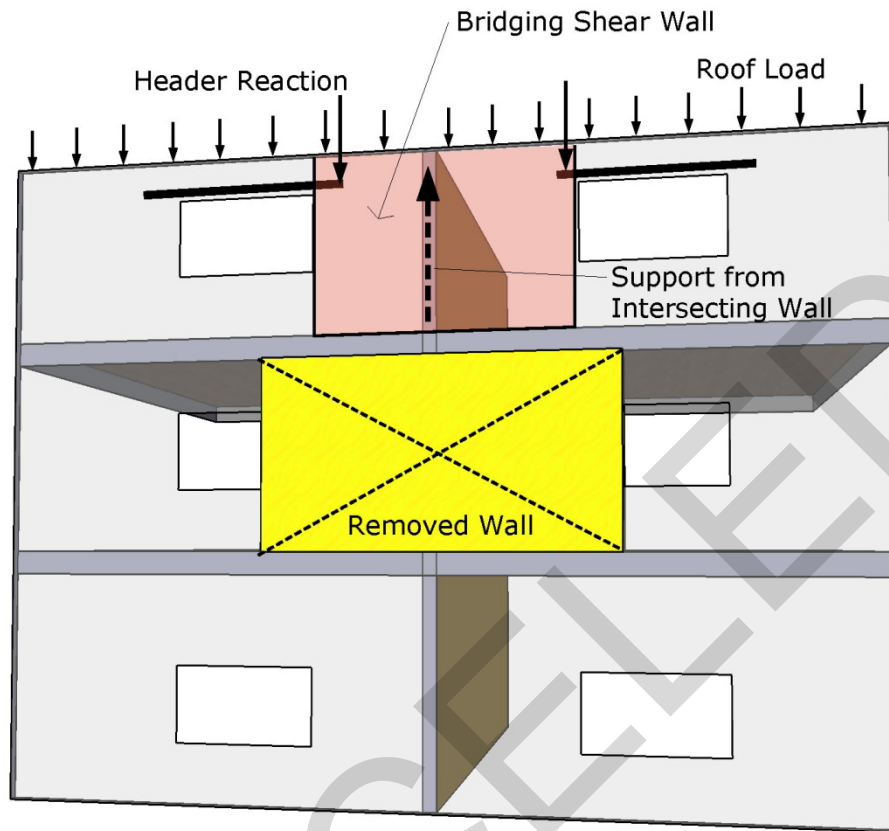
### F-3.3 AP Analysis of Removal of Exterior Load Bearing Walls (Long Direction)

As shown in Figure F-7, a segment of wall of length  $2H$  is removed between the second and third floors.  $H$  is taken as the distance between the 2<sup>nd</sup> floor sub-floor elevation and the bottom of the 11-7/8" EIJ floor joists at the 3<sup>rd</sup> floor ( $=8'-3\frac{1}{8}"$ ).



**Figure F-7. Exterior Wall Removal**

The chosen area of removal takes away the jamb and jack studs supporting the header of the window opening above the third floor. This header must now transfer its reaction via the remaining jack studs into the remaining wall above the 3<sup>rd</sup> floor. This remaining wall segment must then act as a cantilever from a vertical support provided by the crossing interior wall. Any unbalanced loads in the cantilever system must be resolved via drag forces in the 3<sup>rd</sup> floor and top of wall sill elements to adjacent intact wall segments. See Figure F-8.



**Figure F-8. Load Path for Exterior Wall Removal**

The primary loading is that of the roof trusses:

Dead Load

Roof:  $25 \text{ psf} \times 25'-11" / 2 = 324 \text{ plf}$

Wall:  $7 \text{ psf} \times 8'-3" = 58 \text{ plf}$

Roof live load not applicable for UFC design combination

Snow Load

$7 \text{ psf} \times 25'-11" / 2 = 91 \text{ plf}$

Linear Static AP Load Case for Deformation Controlled Elements:

$$G_{LD} = \Omega_{LD} [1.2D + 0.2S]$$

From ASCE 41-13 Table 12-3:

for "Wood Structural Panel Sheathing or Siding"

$h/b = 6.33/9.25 = 0.68 < 2.0$

$m = 3.8$  for Life Safety Primary Element

$m = 4.7$  for Gypsum Wallboard with  $h/b \leq 1.0$  at interior face

Assume all connections to be nailed:

$m = 6.0$  for "Nails – Wood to Wood"

$m = 4.0$  for "Nails – Wood to Metal"

$m = 3.8$  controls

$\Omega_{LD} = 2.0m = 7.6$  (UFC Table 3-4)

$G_{LD} = 7.6*(1.2*(324+58) + 0.2*(91)) = 7.6*(477 \text{ plf}) = 3,622 \text{ plf}$

Header reaction =  $3622 \text{ plf} * 6' / 2 = 10,866$  pounds

Total shear in wall segment at intersection with interior wall  
=  $10,866 + 3622*(6'-4'') = 33,793$  pounds

Unit shear =  $33,793 / 8'-3'' = 3528 \text{ plf}$

Wall strength  $\phi Q_{CE} = 1140 \text{ plf}$  (established in wall analysis F-3.2)

Check UFC Equation 3-13:

$\phi m Q_{CE} \geq Q_{UD}$

$1140 (3.8) = 4332 \text{ plf} > 3,528 \text{ plf}$  **OK**

For force controlled actions:

$\Omega_{LD} = 2.0$  (UFC Table 3-4)

$G_{LF} = 2.0*(1.2*(324+58) + 0.2*(91)) = 2.0*(477 \text{ plf}) = 954 \text{ plf}$

Header reaction =  $954 \text{ plf} * 6' / 2 = 2,862$  pounds

Chord force =  $[2862*6 + 954*(6.33)^2 / 2] / 8.25' = 3,498$  pounds =  $Q_{UF}$

Chord tension  $\phi Q_{CL} = 14,702$  pounds for (1) 2x6 (established in analysis F-3.2)

Total available strength =  $14,702*2 = 29,404$  pounds  $> 3,498$  pounds **OK**

\2\The cantilever walls sections are to be supported by the interior load bearing wall that is perpendicular to cantilever wall sections.

Shear reaction from each side of the cantilever =  $33,793$  lbs

Total shear reaction from 2 cantilevers =  $2*33793 \text{ lbs} = 67,586 \text{ lbs}$

Interior load bearing shear walls with plywood at each side continue through the exterior wall to the exterior face of the building (see Figure F-1).

The exterior plywood sheathing of the long direction exterior walls must be nailed into the end 2x6's of the interior load bearing walls to allow for shear transfer. To carry the shear load from each side of the interior wall, two rows of 10d nails @6" will be required from the exterior sheathing into the end 2x6's.

Cantilever wall reaction then carried by shear in interior load bearing wall (between 3<sup>rd</sup> floor and truss bearing).

Interior load bearing wall has plywood sheathing on both sides.

$$\phi V_s = 0.8 * 2 * (950) = 1520 \text{ plf}$$
$$\phi Q_{CE} = 1.5 (1520) = 2280 \text{ plf}$$

8'-3" wall height for interior load bearing wall

$$Q_{UD} = 67,586 \text{ lbs} / 8.25 \text{ ft} = 8192 \text{ plf}$$
$$m\phi Q_{CE} = (3.8 * 2280 \text{ plf}) = 8664 \text{ plf} > 8192 \text{ plf} \quad \text{OK}$$

The interior wall below the 3<sup>rd</sup> floor supports the shear wall with compression in wall studs. Compression in studs is a force controlled action.

For force controlled actions:

$$\Omega_{LD} = 2.0 \text{ (UFC Table 3-4)}$$
$$G_{LF} = 2.0 * (1.2 * (324 + 58) + 0.2 * (91)) = 954 \text{ plf}$$

$$\text{Header reaction} = 6 \text{ ft} / 2 * 954 \text{ lb/ft} = 2862 \text{ lbs}$$

Reaction onto interior wall for each side:

$$954 \text{ plf} * (6.33 \text{ ft}) + 2862 \text{ lbs} = 8900 \text{ lbs}$$
$$\text{Total reaction (from both sides)} = 2 * 8900 \text{ lbs} = 17,800 \text{ lbs (from exterior walls)}$$

Bearing on interior wall studs (lowest level) will also include floor loading from 2 floors (18'-8" floor span)

$$DL = 25 \text{ psf each floor}$$
$$\text{Reduced LL} = 40 \text{ psf} (1 - 0.375) = 25 \text{ psf each floor}$$

$$DL = 2 \text{ floors} * 25 \text{ psf} * 18.67 \text{ ft} = 934 \text{ plf}$$
$$LL = 2 \text{ floors} * 25 \text{ psf} * 18.67 \text{ ft} = 934 \text{ plf}$$

$$G_{LF} = 2.0*(1.2*(934) + 0.5*(934)) = 1587 \text{ plf}$$

$$G_{LF} = 1587 \text{ plf} * 2 \text{ ft stud spacing} = 3174 \text{ lbs / stud}$$

Bearing on interior wall (lower story) carried by 2x6 studs at 2' on center with in-plane bracing provided by sheathing. Check as column for out-of-plane strength of lower story

Per F-3.2 Stud strength is 9820 lbs / stud

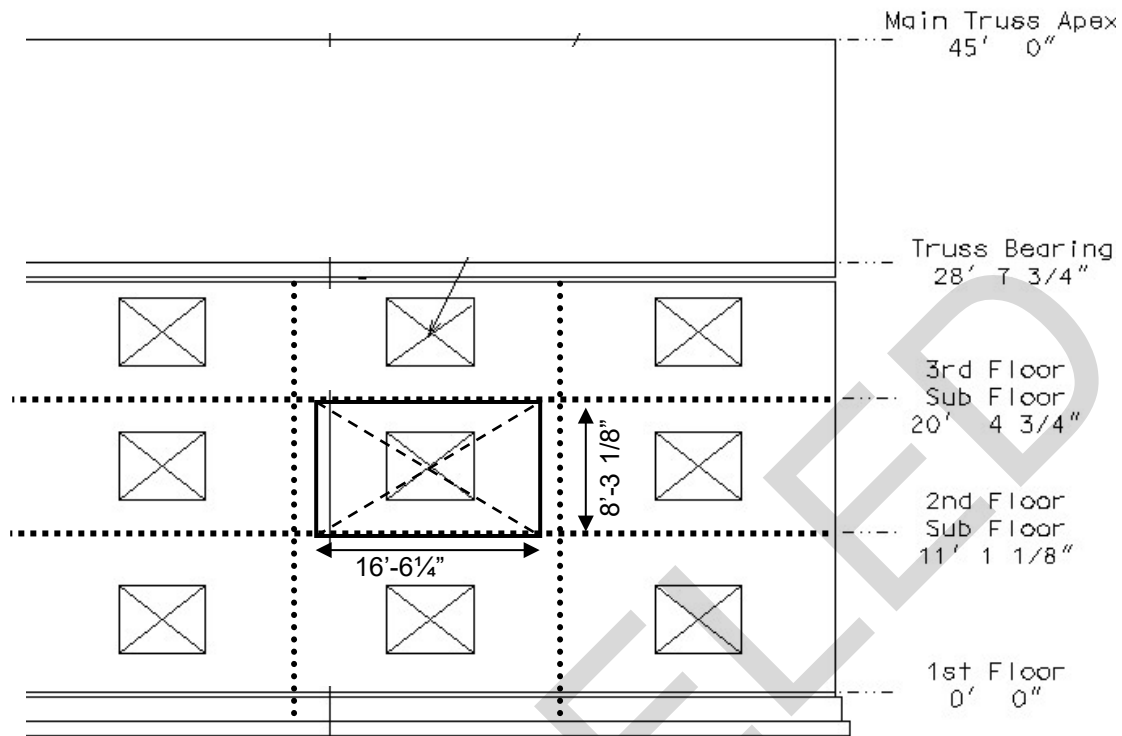
$$9820 \text{ lb / stud} - 3174 \text{ lb / stud} = 6646 \text{ lbs available per stud}$$

$$17,800 \text{ lbs} / 6646 \text{ lbs} = 2.7 \text{ studs}$$

3 studs available within 4 feet of interior wall      **OK**

#### **F-3.4      AP Analysis of Removal of Exterior Load Bearing Walls (Long Direction, Alternate Location)**

As shown in Figure F-9, a segment of wall of length 2H is removed between the second and third floors. H is taken as the distance between the 2<sup>nd</sup> floor sub-floor elevation and the bottom of the 11-7/8" EIJ floor joists at the 3<sup>rd</sup> floor (=8'-3 1/8").



**Figure F-9. Exterior Wall Removal**

The chosen area of removal between the second and the third floor will allow for the shear wall above to span over and be supported on each side of the removed wall section. (Note, shifted removal locations could be addressed similar to F-3.3, however they will be less critical because additional cantilever shear wall sections could be engaged).

The primary loading is that of the roof trusses:

Dead Load

Roof:  $25 \text{ psf} \times 25'-11" / 2 = 324 \text{ plf}$

Wall (including glazing):  $7 \text{ psf} \times 8'-3" = 58 \text{ plf}$

Roof live load not applicable for UFC design combination

Snow Load

$7 \text{ psf} \times 25'-11" / 2 = 91 \text{ plf}$

Linear Static AP Load Case for Deformation Controlled Elements:

$$G_{LD} = \Omega_{LD} [1.2D + 0.2S]$$



From ASCE 41-13 Table 12-3:

for "Wood Structural Panel Sheathing or Siding"

$$h/b = 8.25/9.25 = 0.89 < 2.0$$

$$m = 3.8 \text{ for Life Safety Primary Element}$$

$$m = 4.7 \text{ for Gypsum Wallboard with } h/b \leq 1.0 \text{ at interior face}$$

Assume all connections to be nailed:

$$m = 6.0 \text{ for "Nails – Wood to Wood"}$$

$$m = 4.0 \text{ for "Nails – Wood to Metal"}$$

$$m = 3.8 \text{ controls}$$

$$\Omega_{LD} = 2.0m = 7.6 \text{ (UFC Table 3-4)}$$

$$G_{LD} = 7.6*(1.2*(324+58) + 0.2*(91)) = 7.6*(477 \text{ plf}) = 3,622 \text{ plf}$$

Total force onto the removed section

$$F = 3622 \text{ lb/ft} * 16.5 \text{ ft} = 59,763 \text{ lbs}$$

$$\text{Shear each side } V = 59,763 \text{ lbs} / 2 = 29,882 \text{ lbs}$$

$$\text{Unit shear} = 29,882 \text{ lbs} / 9.25' = 3230 \text{ plf}$$

$$\text{Wall strength } \phi Q_{CE} = 1860 \text{ plf (established in wall analysis F-3.2)}$$

Check UFC Equation 3-13:

$$\phi m Q_{CE} \geq Q_{UD}$$

$$1860 (3.8) = 7,068 \text{ plf} > 3,230 \text{ plf} \quad \text{OK}$$

For force controlled actions:

$$\Omega_{LD} = 2.0 \text{ (UFC Table 3-4)}$$

$$G_{LF} = 2.0*(1.2*(324+58) + 0.2*(91)) = 2.0*(477 \text{ plf}) = 954 \text{ plf}$$

$$\text{Wall moment} = G_{LF} * L^2 / 8 = 954 \text{ plf} * (16.5')^2 / 8 = 32,466 \text{ lb-ft}$$

Chord force limit will be governed by tension strength since compression chord is fully braced in both directions

$$\text{Chord force} = M/d = [32,466 \text{ lb-ft}] / 9.25' = 3,510 \text{ pounds} = Q_{UF}$$

Chord tension  $\phi Q_{CL} = 14,702$  pounds for (1) 2x6 (established in analysis F-3.2)

Total available capacity =  $14,702 * 2 = 29,404$  pounds > 3,923 pounds OK

End reactions of spanning wall will be carried by studs in compression between the 1<sup>st</sup> and 2<sup>nd</sup> floor. Compression of studs will be force controlled.

$G_{LF} = 954$  plf + additional wall load carried by each stud to the foundation

$h = 28'-8" = 28.67'$  stud spacing = 2 ft wall DL = 7 psf

Additional wall load =  $954 \text{ plf} + 2.0 * (0.5 * 7 \text{ psf} * 28.67') = 201 \text{ lbs/ft}$

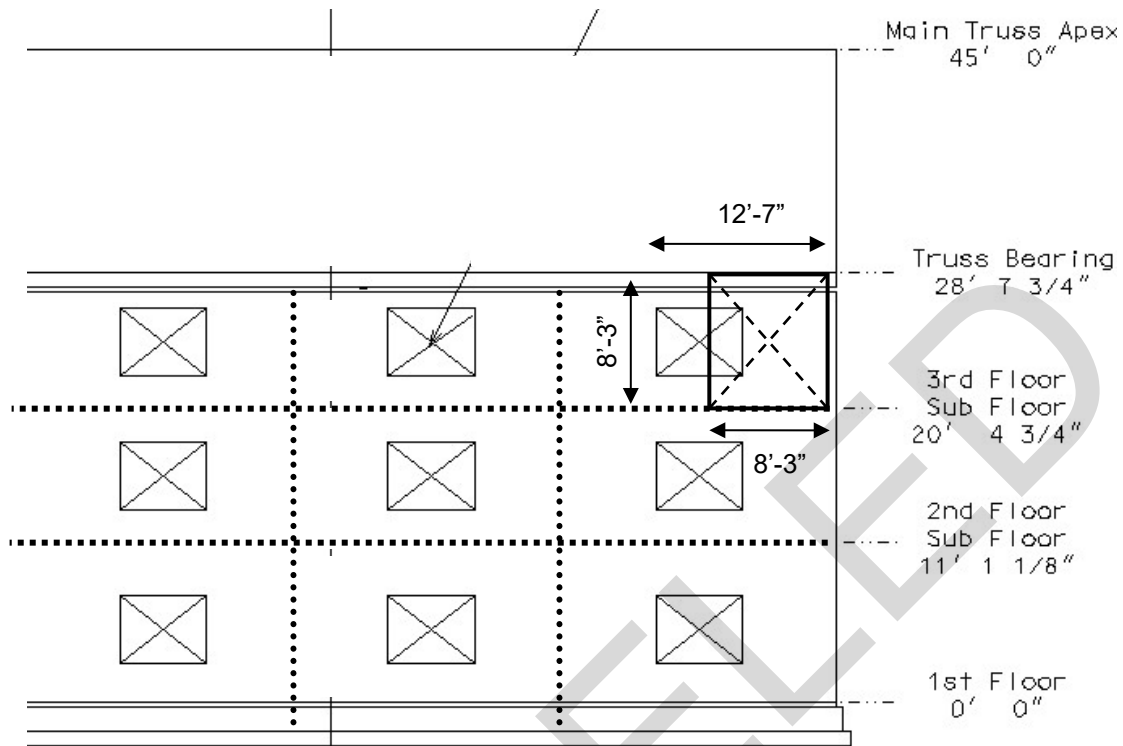
$Q_{UD} = 954 \text{ lbs/ft} * 0.5 * 16.5 \text{ feet} + 201 \text{ lbs/ft} * 2 \text{ ft} = 8273 \text{ lbs/stud}$

Compression carried by 2x6 studs at 2' on center with in-plane bracing provided by sheathing and blocking. Check as column for out-of-plane capacity of lower story

Per F-3.2 Stud capacity is 9820 lbs / stud > 8273 lbs /stud **OK**

### **F-3.5 AP Analysis of Removal of Exterior Load Bearing Walls Below Third Floor (Long Direction)**

As shown in Figure F-10, a segment of wall of length H is removed at exterior corners in each direction between the second and third floors. H is taken as the distance between the 3<sup>rd</sup> floor sub-floor elevation and the truss bearing location (=8'-3").



**Figure F-10. Exterior Corner Wall Removal**

The primary loading is that of the roof trusses:

Dead Load

$$\text{Roof: } 25 \text{ psf} * 25'-11" / 2 = 324 \text{ plf}$$

Roof live load not applicable for UFC design combination

Snow Load

$$7 \text{ psf} * 25'-11" / 2 = 91 \text{ plf}$$

Linear Static AP Load Case for Deformation Controlled Elements:

$$G_{LD} = \Omega_{LD} [1.2D + 0.2S]$$

From ASCE 41-13 Table 12-3:

for "Frame Components Subjected to Axial Tension and/or Bending"

$m = 2.5$  for Life Safety Primary Element

Connections to include:

$m = 6.0$  for "Nails – Wood to Wood"

$m = 2.8$  for "Machine Bolts – Metal to Wood"

$m = 2.5$  controls

$\Omega_{LD} = 2.0m = 5.0$  (UFC Table 3-4)

$G_{LD} = 5.0 * (1.2 * (324) + 0.2 * (91)) = 5.0 * (407 \text{ plf}) = 2,035 \text{ plf}$

Provide new glulam beams at truss level, as shown in Figure F-11, to provide capacity for corner cantilever

Cantilever span =  $L = 12'-7"$

$Q_{UD} = G_{LD} * L^2 / 2 = 2,035 \text{ plf} * (12.6 \text{ ft})^2 / 2 = 161,539 \text{ lb-ft}$

Try glulam 24F-V5 Southern Pine

ANSI/AFPA SDPWS-2005

$F_b = 2400 \text{ psi}$

Volume factor  $C_v = [(21 / L_o)^{(1/20)} * (12 / d)^{(1/20)} * (5.125 / b)^{(1/20)}] \leq 1$

$L_o = 2 * L$  for cantilever = 25.2 ft

Width =  $b = 3"$

Depth =  $d = 12.375"$

$C_v = [(21 / 12.6)^{(1/20)} * (12 / 12.375)^{(1/20)} * (5.125 / 3)^{(1/20)}] \leq 1$

$C_v = 1$

$S = b * d^2 / 6 = 76.6 \text{ in}^3$

$C_L = (1 + F_{be}/F_b) / 1.9 - \sqrt{((1 + F_{be}/F_b) / 1.9)^2 - (F_{be}/F_b) / 0.95}$

$R_b = \sqrt{L_e * d / b^2}$

Beam supported against lateral and rotational displacement by nailing at each truss bearing post at 48" on center.

$L_u = 48"$

$L_e = 1.33 * L_u$  (uniformly distributed load on cantilever) = 63.8"

$R_b = \sqrt{63.8 * 12.375 / 3^2}$

$F_{bE} = 1.2 E_{min} / R_b^2 = 1.2 * 0.78 * 10^6 * (1.5/0.85) / 9.37 = 18817$

$F_{be}/F_b = 18817 / 2400 = 7.84$

$$C_L = (1 + F_{be}/F_b) / 1.9 - \sqrt{((1 + F_{be}/F_b) / 1.9)^2 - (F_{be}/F_b) / 0.95}$$
$$C_L = (1 + 7.84) / 1.9 - \sqrt{((1 + 7.84) / 1.9)^2 - (7.84) / 0.95} = 0.99$$

Lesser of  $C_L$  or  $C_v$  shall apply,  $C_L = 0.99$

$$F'_b = C_L C_{fu} K_f \Phi_b \lambda F_b$$

All factors 1 except:

$$C_L = 0.99$$

$$\Phi = 0.85$$

$$K_f = 2.16 / \Phi$$

$$\lambda = 1 \text{ per UFC}$$

$$\phi F'_b = 0.99 * 2.16 * 2400 \text{ psi} = 5146.7 \text{ psi}$$

$$\phi Q_{CE} = M' = \phi F'_b * S = 5146.7 \text{ psi} * 76.6 \text{ in}^3 = 394,084 \text{ lb-in} = 32,840.3 \text{ lb-ft}$$

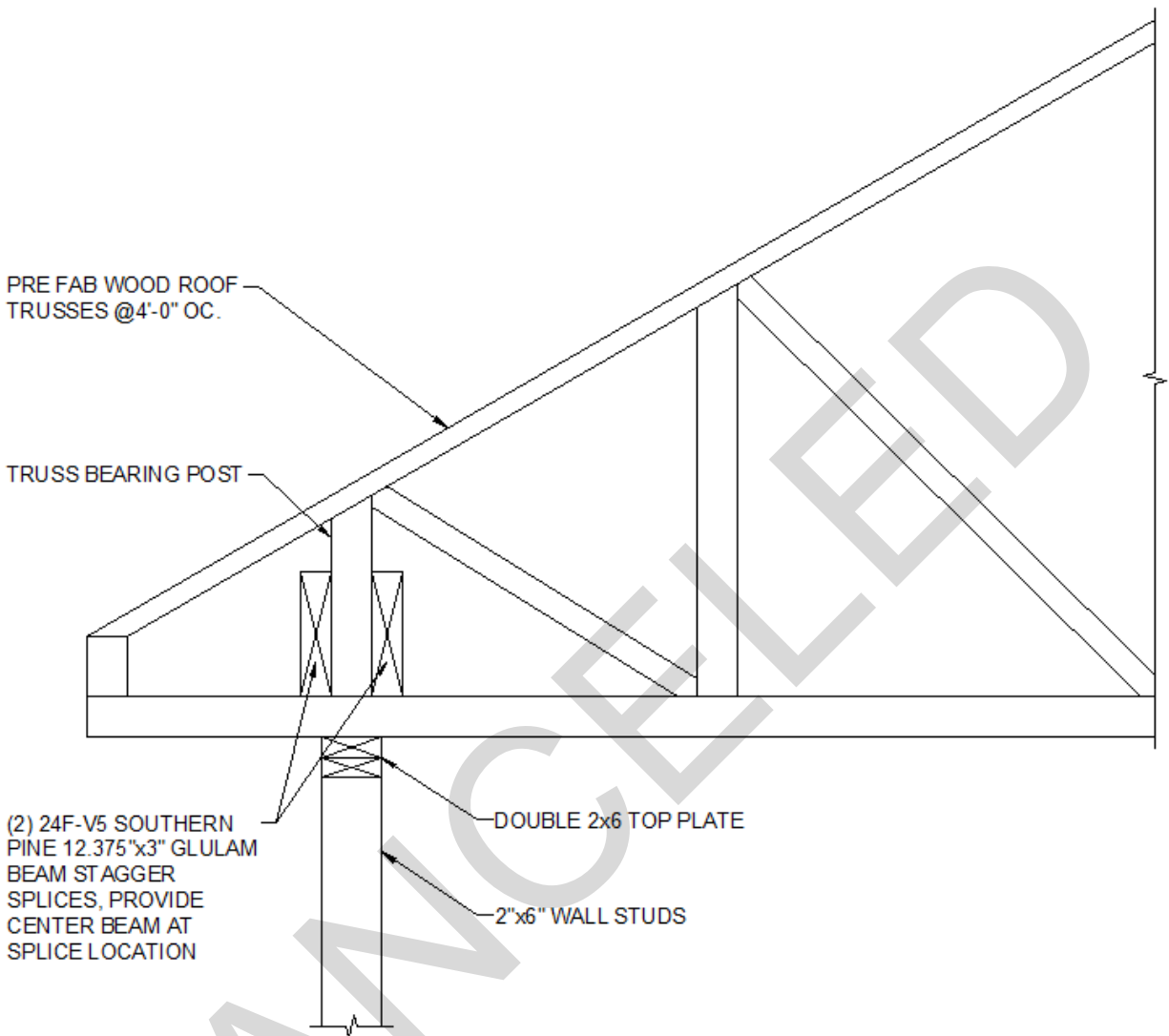
$$\phi m Q_{CE} \geq Q_{UD}$$

$$\phi m Q_{CE} = 32,840.3 \text{ lb-ft} * 2.5 = 82,100.8 \text{ lb-ft}$$

$$Q_{UD} = 161,539 \text{ lb-ft}$$

$Q_{UD} / \phi m Q_{CE} = 1.97$  therefore require two (2) 24F-V5 Southern Pine 12.375" x 3" beams

Provide one (1) 24F-V5 Southern Pine 12.375" x 3" beam at each side of roof truss vertical bearing post. Stagger the splice of these beams and provide a third identical beam between vertical truss posts at splice location.



**Figure F-11. Exterior Roof Beams**

Anchorage of beams required to prevent uplift of the cantilever backspan. Nail Glulam to truss posts, and utilize hold-down anchors, such as Simpson HD9B at each side of truss post to resist uplift.

Two tie downs shall be used so in accordance with ASCE 41-13 section 12.3.3:  
“demands on nails, screws, lags, bolts, split rings and shear plates used to link wood components to other wood or metal components shall be considered deformation controlled actions. Demands on bodies of connections and bodies of connection hardware shall be considered force controlled actions”

Accordingly, since 2 tie downs will be used, the action shall be considered force controlled.

$$\Omega_{LD} = 2.0 \text{ (UFC Table 3-4)}$$

$$G_{LF} = 2.0 * (1.2 * (324) + 0.2 * (91)) = 2.0 * (407 \text{ plf}) = 814 \text{ plf}$$

$$\text{Cantilever moment} = G_{LF} * L^2 / 2 = 814 \text{ plf} * (12.6 \text{ ft})^2 / 2 = 64615.3 \text{ lb-ft}$$

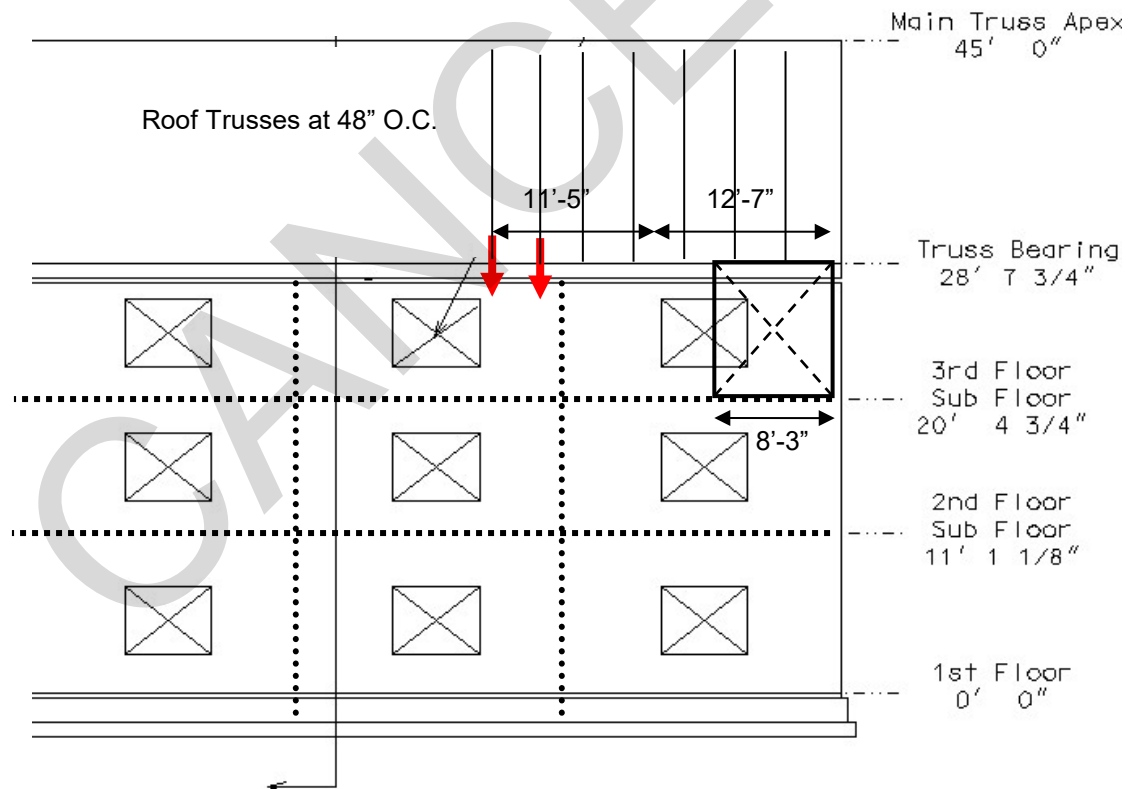
$$\text{Backspan distance} = 11'-5" = 11.4 \text{ ft}$$

$$\text{Uplift reaction} = 64615.3 \text{ lb-ft} / 11.4 \text{ ft} = 5668.8 \text{ lbs}$$

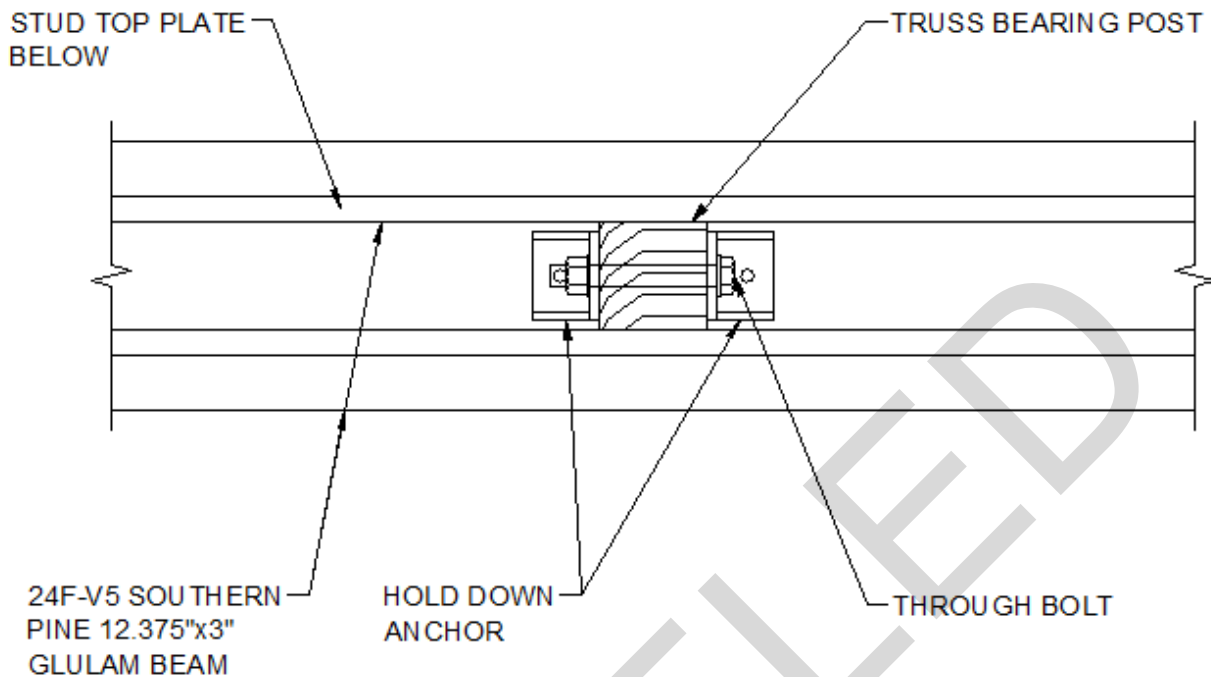
Based on bolting into (2) 2x6 wall headers:  
Capacity of Simpson HD9B = 7740 lbs

A single anchor would suffice; however provide 2 anchors to prevent connection eccentricity.

See Figure F-12 for tie down regions and Figure F-13 for anchorage configuration.



**Figure F-12. Exterior Corner Wall Removal Tie Down Locations**



**Figure F-13. Roof Beam Tie Down Anchorage, Plan View**

For a stud bearing to support bridging, beam shall be force-controlled:

Stud compression in lowest level:  $Q_{UF} = G_{LF} * L + \text{Uplift reaction} + \text{additional wall load}$

Additional wall load carried by each stud to the foundation

$h = 28'-8" = 28.67'$  stud spacing = 2 ft wall DL = 7 psf

Additional wall load =  $2.0 * (0.5 * 7 \text{ psf} * 28.67') = 201 \text{ lbs / ft} * 2 \text{ ft} = 402 \text{ lbs}$

$Q_{UF} = 814 \text{ lb/ft} * 12.6 \text{ ft} + 5669 \text{ lbs} + 402 \text{ lbs} = 16,328 \text{ lbs}$

Compression carried by 2x6 studs at 2' on center with in-plane bracing provided by sheathing and blocking. Check as column for out-of-plane strength of lower story

Per F-3.2 Stud strength is 9820 lbs / stud

$16,328 \text{ lbs} / 9,820 = 1.7 \text{ studs}$

2 studs available at each window jamb **OK**

/2/



Change 2  
1 June 2013

**APPENDIX G COLD FORMED STEEL EXAMPLE**

**G-1 INTRODUCTION**

A cold formed steel load-bearing wall structure example has been prepared to illustrate Alternate Path analysis of a cold formed steel structure. The structure is assumed to have an occupancy of less than 100 people and is classified as \3\ Risk Category /3/ II per UFC 3-301-01.

The example has been prepared using tools and techniques commonly applied by structural engineering firms in the US. For cold formed design, this example relies primarily upon hand analysis supplemented with publically available AISIWIN software and standard design tables published in the International Building Code. \3\ Risk Category /3/ II Option 2: Alternate Path has been selected from UFC Table 2-2 to evaluate resistance to progressive collapse.

**G-2 BASELINE DESIGN**

The example building is a 3-story cold formed steel structure, six bays long and two bays deep similar to the building considered in the Appendix F wood example. The relevant design information is shown in Figures G-1 through G-4.

Platform construction is used. The internal and end shear walls are load bearing, with a composite deck running in the direction of the long length of the structure. One 4.5-foot x 6-foot window opening is present in each room. The footings are reinforced concrete with masonry. There are no internal partition walls.

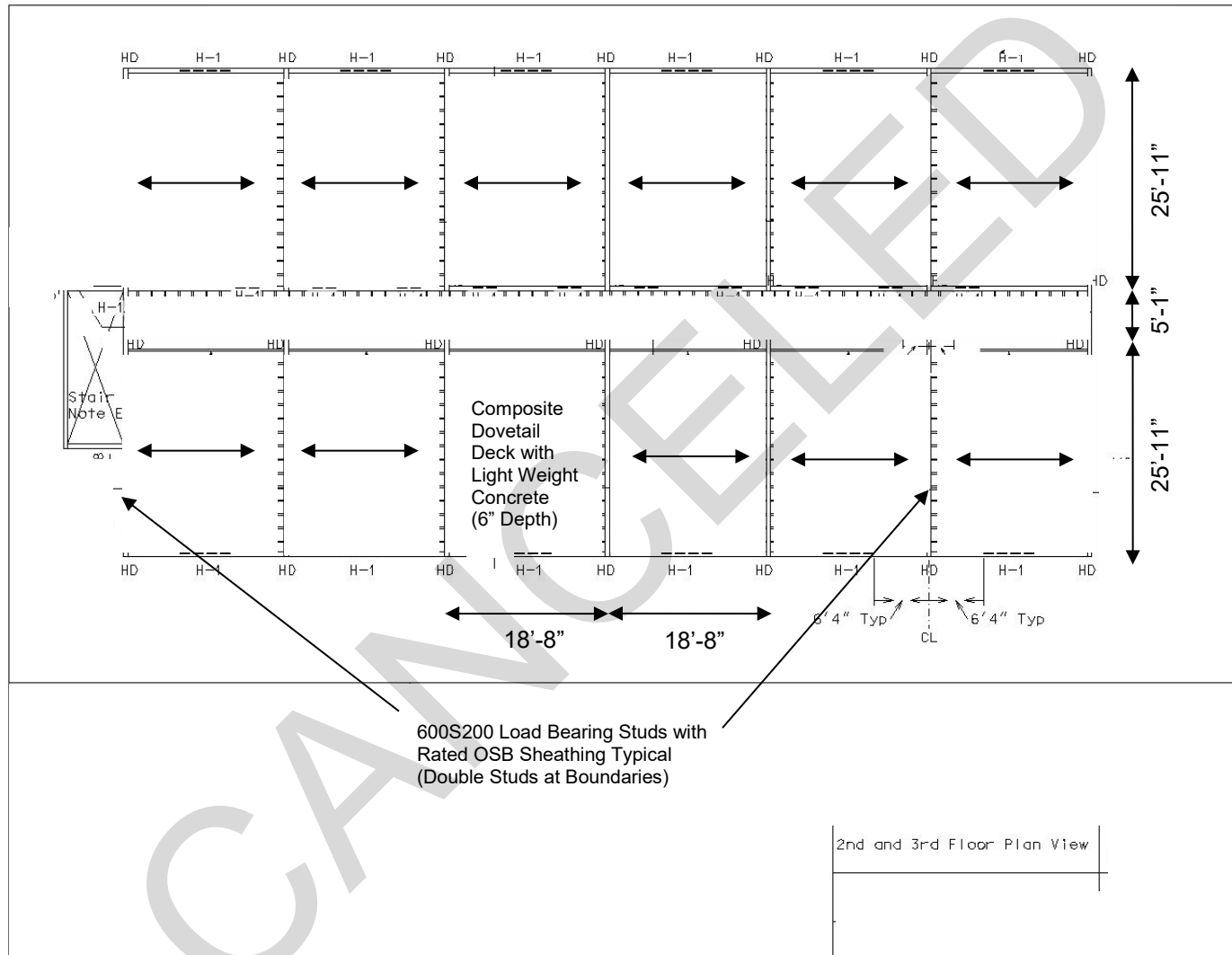
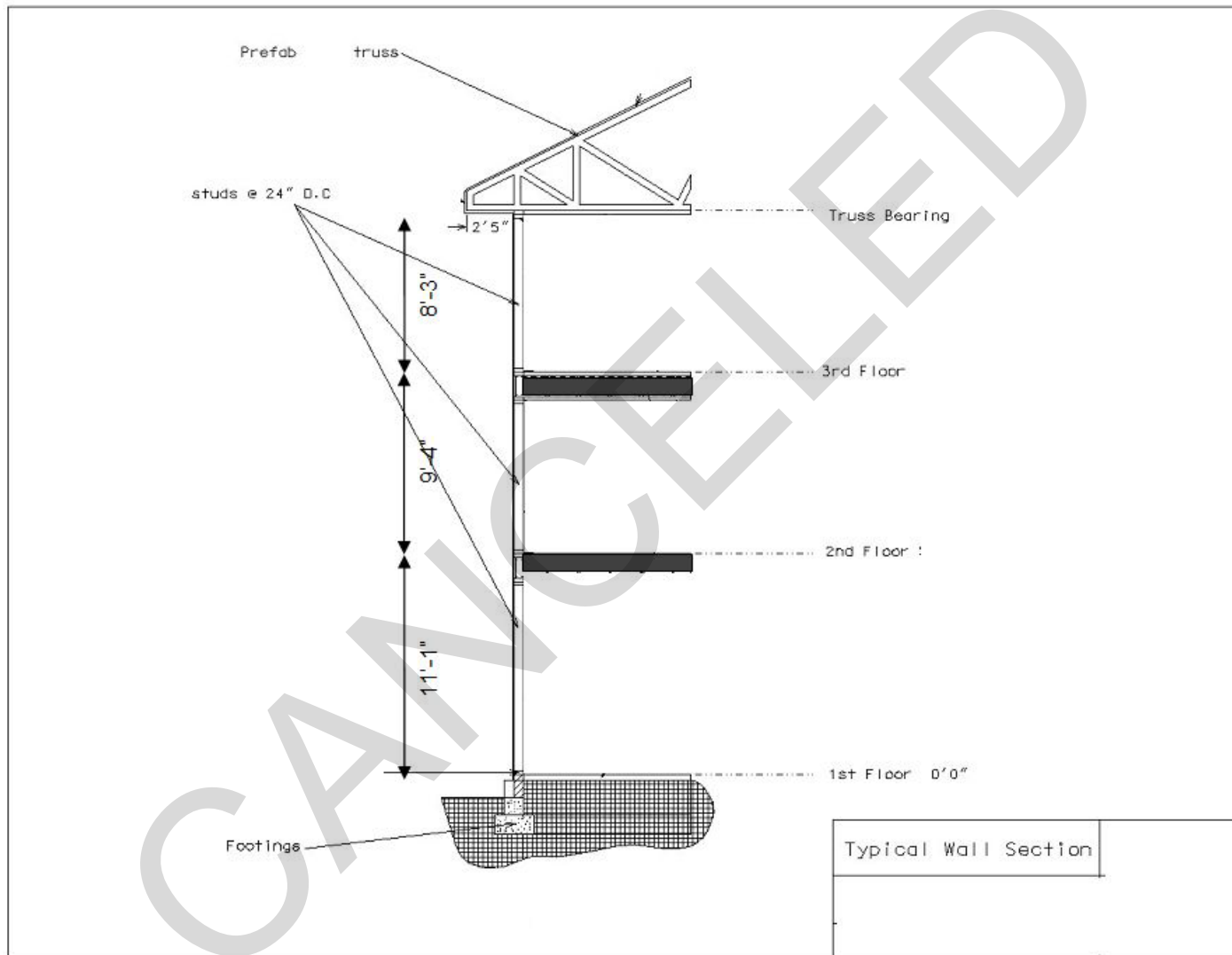


Figure G-1. Cold Formed Building Example Plan



**Figure G-2. Cold Formed Example Wall Section**

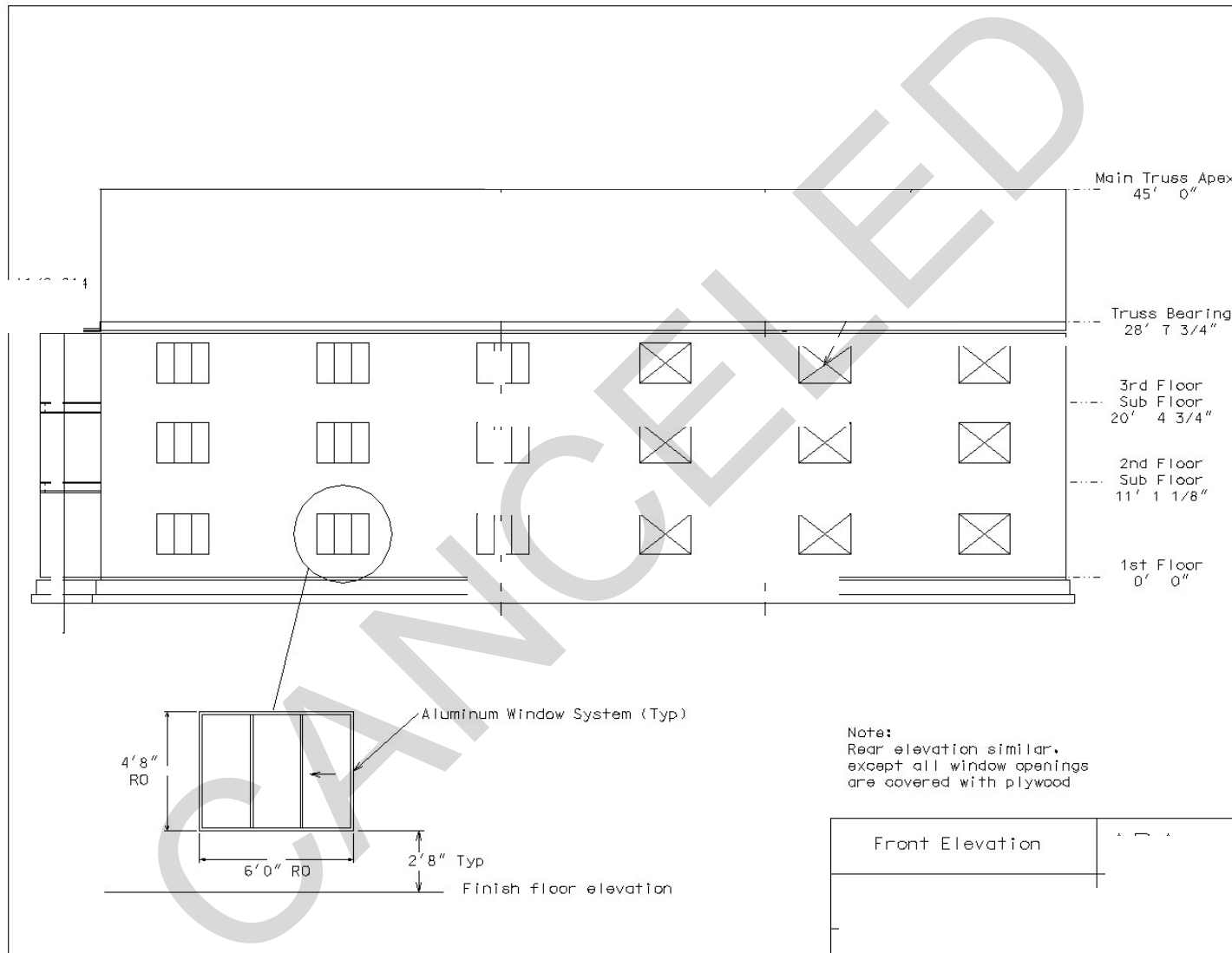


Figure G-3. Cold Formed Example Exterior Wall Elevation

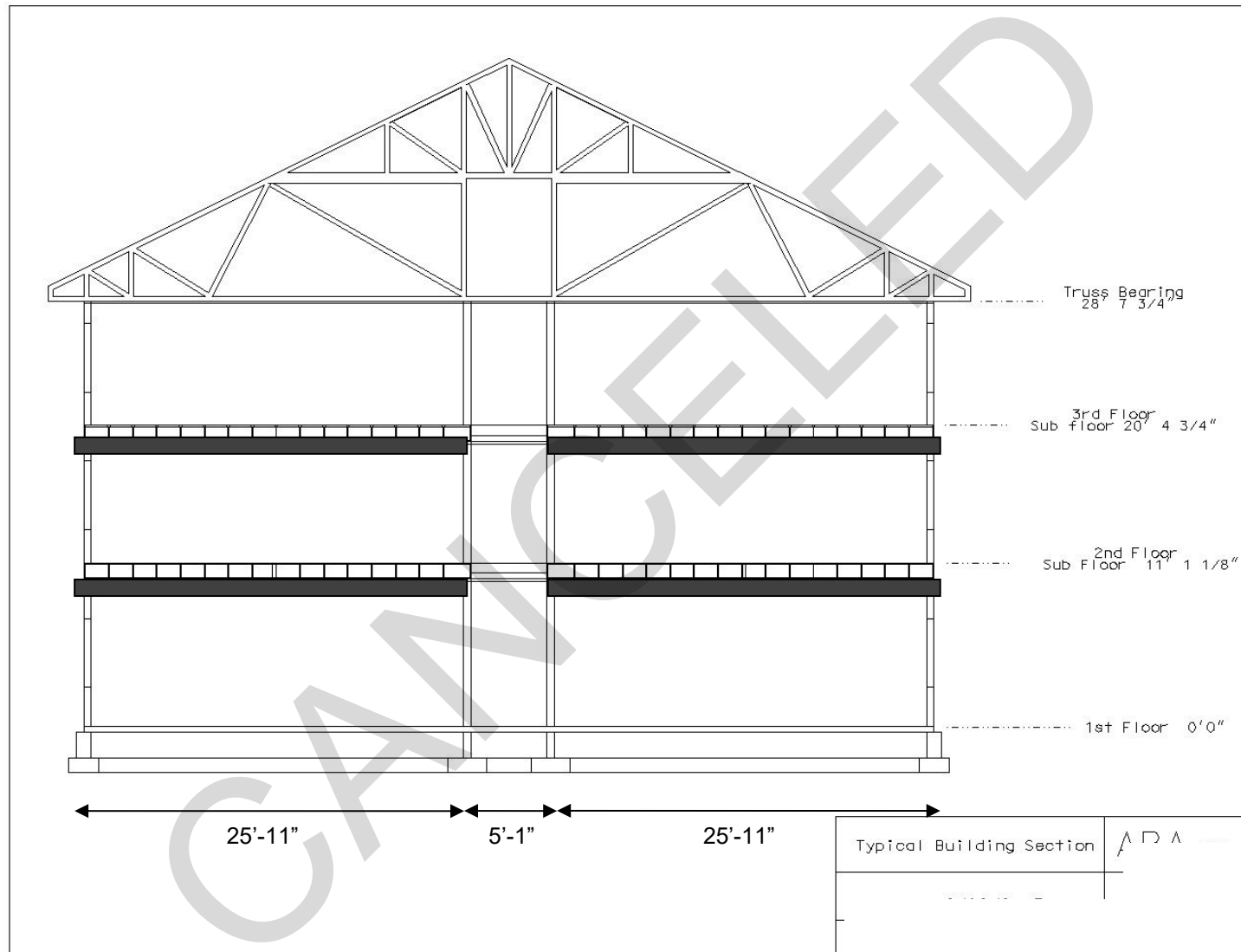


Figure G-4. Cold Formed Example Building Section

## **G-2.1 Construction and Materials**

### **Wall Construction:**

Interior Walls and Exterior End Walls: 7/16" rated OSB Sheathing both Faces, blocked 600S200-68 Gr50 studs at 24 inches on center  
(2) 600S200-68 Gr50 studs stitched together at wall ends

Exterior Long Direction Walls: 7/16" rated OSB Sheathing exterior face, blocked 600S200-54 Gr50 studs at 24 inches on center  
(2) 600S200-68 Gr50 studs stitched together at wall ends

### **Floor Construction:**

Composite dovetail deck with light weight concrete (6" total depth) such as CSI Versa Deck or Vulcraft VLI.

### **Roof Construction:**

Engineered Trusses with 1/2" Plywood Sheathing

### **Material Grades:**

A653 Grade 50 cold formed steel

## **G-2.2 Loading Assumptions**

Typical loading relevant to AP Analysis:

Floors:

Dead Load: 62 psf including concrete filled deck, MEP, ceiling

Live Load: 40 psf

Roof:

Dead Load: 25 psf including roofing and bottom surface ceiling

Live Load: 20 psf

Snow Load: 7 psf (design, including all applicable ASCE 7-05 factors)

Walls:

Dead Load: 7 psf including sills and headers

Live loads reducible depending on tributary area considered.

## **G-2.3 Relevant Standards and Reference Documents**

American Iron and Steel Institute (AISI) North American Specification for Design of Cold-Formed Steel Structural Members

International Building Code 2006

For design compatibility with UFC 4-023-03, the LRFD approach is followed.

### **G-3 ALTERNATE PATH ANALYSIS**

An alternate path (AP) analysis is conducted according to the requirements of Section 3-2. Removal scenarios are defined by 3-2.9. To bridge the wall removals, the remaining load bearing walls will be utilized as shear wall elements. The linear static AP analysis method will be followed.

#### **G-3.1 Scope and Analysis Assumptions**

Three scenarios of wall removal will be evaluated:

1. Removal of interior load-bearing wall (see Figure G-5) at first story
2. Removal of exterior long walls (see Figure G-6) at second story
3. Removal of exterior long walls (see Figure G-7) at third story

General assumptions of the analysis are as follows:

- No wind or internal pressure acts on interior or exterior walls during wall removal scenario
- Contributions of engineered roof trusses to the alternate path capacity are neglected due to uncertainty in specific properties during the design phase. At the designer's option, these elements could be included, provided that appropriate performance specifications including AP requirements are incorporated into the contract documents.

Per ASCE 41-13

C12.2.2.5:

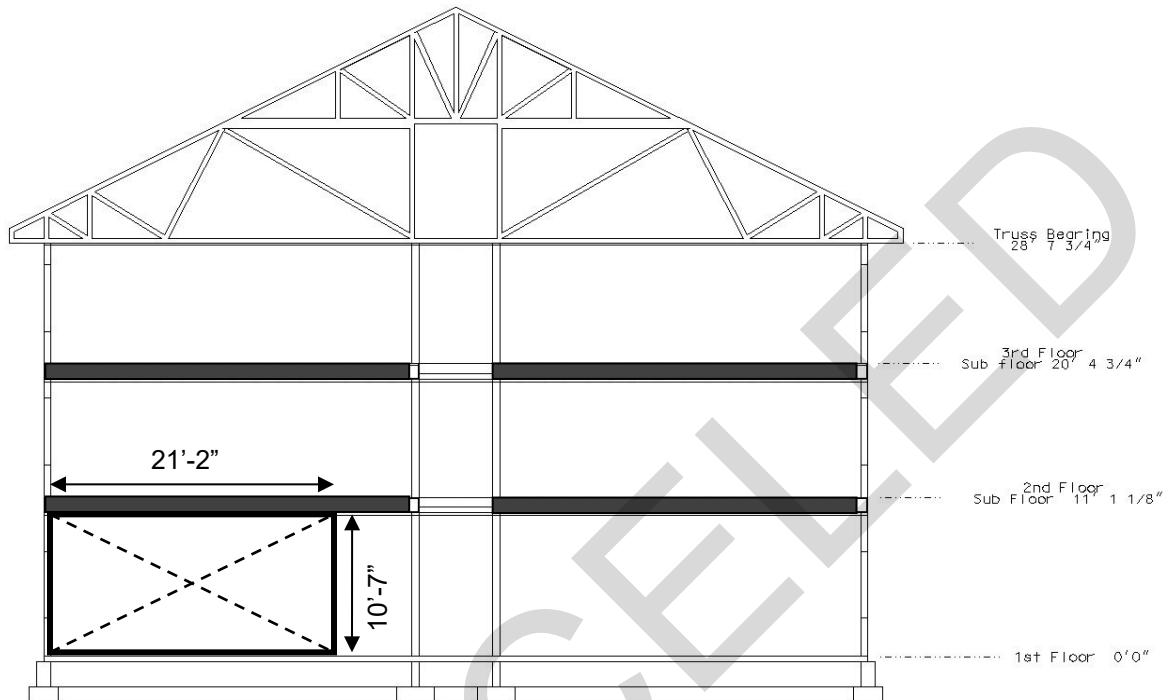
"Actions associated with wood and light metal framing components generally are deformation controlled, and expected strength material properties will be used most often."

12.3.3:

"Demands on connectors, including nails, screws, lags, bolts, split rings, and shear plates used to link wood components to other wood or metal components shall be considered deformation controlled actions. Demands on bodies of connections, and bodies of connection hardware, shall be considered force-controlled actions."

For the AP analysis of this structure, wood shear wall assemblies and primary connections will be taken to be deformation controlled elements. Bodies of connectors will be checked as force controlled elements as required.

### G-3.2 AP Analysis of Interior Load Bearing Wall Removal



**Figure G-5. Interior Load Bearing Wall Removal**

As shown in Figure G-5, a segment of wall of length  $2H$  is removed from the lower floor interior load bearing wall.  $H$  is taken as the distance between the 1<sup>st</sup> floor sub-floor elevation and the bottom of the 6-inch floor slab at the 2<sup>nd</sup> floor ( $=10'-7"$ ).

The remaining structure creates a shear wall element that must span the floor and wall loads from the 2<sup>nd</sup> and 3<sup>rd</sup> stories. The shear walls above will be divided by the floor slab at each level. The wall from level 2 to level 3 will have an overall depth of 8'-10" and the wall from level 3 to the roof level will have an overall depth of 8'-3". The boundary (chord) elements consist of 600T200 track provided at the top and bottom of each wall.

To bridge the removed section of wall, the shear wall panel must span from the exterior wall to a wall pier segment consisting of the remaining portion of the removed lower story interior wall. Note that no continuity action that might engage the right-hand wall panels in Figure G-5 is possible due to the corridor opening.

The structure contains no irregularities and therefore the linear static AP analysis is permitted per Section 3-2.11.1.1.



Loading:

Dead Load

Floors (each):  $62 \text{ psf} * 18'-8" = 1153 \text{ plf}$

Walls (each level):  $7 \text{ psf} * 9'-4" = 65 \text{ plf}$

Reduce live load in accordance with IBC 2006 1607.9.2:

$$A = 18.67' * 25.91' * 2 = 967.5 \text{ SF}$$

$$R = 0.08 (A - 150) = 0.08 (967.5 - 150) = 65.4\%$$

$$R_{\max} = 60\% \text{ or } 23.1(1 + 62/40) = 58.9\%$$

Live Load used in AP check (each level)

$$40 \text{ psf} * 18.67' * (1 - 0.589) = 307 \text{ plf}$$

Linear Static AP Load Case for Deformation Controlled Elements:

$$G_{LD} = \Omega_{LD} [1.2D + 0.5L]$$

From ASCE 41-13 Table 12-3:

for "Wood Structural Panel Sheathing or Siding"

$$h = 21.17' / 2 = 10.59'$$

$$b = 8.25'$$

$$h/b = 10.59' / 8.25' = 1.28 < 2.0$$

$$m = 3.8 \text{ for Life Safety Primary Element}$$

Connections to be screwed:

$$m = 1.8 \text{ for "Screws –Metal to Wood"}$$

$$m = 1.8 \text{ controls}$$

$$\Omega_{LD} = 2.0m = 3.6 \text{ (UFC Table 3-4)}$$

$$G_{LD} = 3.6 * (1.2 * (1153 + 62) + 0.5 * (307)) = 3.6 * (1640 \text{ plf}) = 5,802 \text{ plf (each level)}$$

Deformation controlled actions:

$$\text{Shear: } Q_{UD} = 5,802 \text{ plf} * 21.17' / 2 / 8.25' = 7,444 \text{ plf in wall}$$

Capacity checks of deformation controlled actions:

Shear in Wall:

$$\text{Wall shear strength } \phi Q_{CE} = 1.5 (\phi v_s)$$

where  $\phi v_s$  = LRFD shear strength of wall taken from IBC

Factor 1.5 for expected strength is taken from ASCE 41-13 12.4.9.2 but note that Section 3-2.11.7.1 requires inclusion of the material specific  $\phi$  factor whereas ASCE 41-13 uses a  $\phi$  of 1.0.

From IBC Table 2306.2:

For “Wood Structural Panels – Sheathing”, 7/16” rated OSB sheathing, fasteners at 4” O.C max at edges:

$$v_{wc} = 1410 \text{ plf (for one side of wall)}$$

Note that value for wind is chosen rather than seismic per UFC Ch. 7.

Since the interior walls have identical plywood sheathing on both sides, capacity of each individual side is additive per IBC 2211.2.2.

$$\phi = 0.55 \text{ for LRFD per IBC 2211.2.1}$$

$$\phi v_s = 0.55 (1410 \text{ plf}) (2) = 1551 \text{ plf}$$

$$\phi Q_{CE} = 1.5 (\phi v_s) = 1.5 (1551) = 2327 \text{ plf}$$

Check UFC Equation 3-13:

$$\phi m Q_{CE} \geq Q_{UD}$$

$$2327 \text{ plf} (3.8) = 8,842.6 \text{ plf} > 7,444 \text{ plf} \quad \text{OK}$$

For force controlled actions:

$$\Omega_{LD} = 2.0 \text{ (UFC Table 3-4)}$$

$$G_{LF} = 2.0 * (1.2 * (1153 + 62) + 0.5 * (363)) = 2.0 * (1640 \text{ plf}) = 3,280 \text{ plf (each level)}$$

Force controlled actions:

$$\text{Moment in each wall above: } Q_{UF} = 3,280 \text{ plf} * 21.17' * 21.17' / 8 = 183,749 \text{ lb-ft}$$

$$\text{Chord Force: } Q_{UF} = M/h = 183,749 \text{ lb-ft} / 8.25 \text{ ft} = 22,273 \text{ pounds}$$

Though ASCE 41 is somewhat ambiguous on the subject, consider the chord forces to be forced controlled actions because ASCE 41 Table 12-3 designates “Frame Components Subject to Axial Compression” as force controlled. Treating both tension and compression chords as force controlled will be conservative.

Each chord and track element will be fully braced in both directions:

600T200-54

ASTM A653 Gr 50

$F_y = 50 \text{ ksi}$

$F_u = 65 \text{ ksi}$

#### Tension yield

$$\begin{aligned}\phi_t &= 0.9 \\ T_n &= A_g F_y \\ A_g &= 0.5653 \text{ in}^2 \\ \phi_t T_n &= 0.9 * 0.5653 \text{ in}^2 * 50,000 \text{ psi} = 25,438 \text{ pounds}\end{aligned}$$

#### Tension fracture

$$\begin{aligned}\phi_t &= 0.75 \\ T_n &= A_n F_u \\ A_n &= A_g - 3 * 3/16" * 0.054" = 0.535 \text{ in}^2 \text{ (3 - \#8 screws)} \\ \phi_t T_n &= 0.75 * 0.535 \text{ in}^2 * 65,000 \text{ psi} = 26,078 \text{ pounds}\end{aligned}$$

#### Compression

$$\begin{aligned}\phi_t &= 0.85 \\ P_n &= A_g F_y \\ A_g &= 0.5653 \\ \phi_t T_n &= 0.85 * 0.5653 \text{ in}^2 * 50,000 \text{ psi} = 24,025 \text{ pounds}\end{aligned}$$

$$\phi Q_{CL} = 24,025 \text{ pounds for (1) 600T200-54} > Q_{UF} = 22,273 \text{ pounds}$$

Track must be spliced with a minimum gross area of 0.56 in<sup>2</sup> or doubled across splice locations

At each edge of the removed wall segment, the shear present in the wall above must be transferred into the load bearing walls studs below. Compression in wall studs shall be considered force controlled.

$$G_{LF} = 3,280 \text{ plf (each level)} \qquad 6,560 \text{ plf (2 levels) above}$$

$$\text{Wall shear: } Q_{UF} = G_{LF} * L / 2$$

$$Q_{UF} = 6,560 * (21.17 \text{ ft}) / 2 = 69,437 \text{ lbs each side of removal}$$

Typical studs are 600S200-68 at 24" on center and are braced in-plane by rated sheathing and mid-height blocking and will be controlled by out-of-plane buckling.

$$\text{Stud length at the lower level} = (11' - 1") - (0' - 6") = 10' - 7" = 10.6 \text{ ft}$$

Per American Iron and Steel Institute (AISI) North American Specification for Design of Cold-Formed Steel Structural Members

$$\begin{aligned}\phi_c &= 0.85 \\ \lambda_c &= \sqrt{(F_y/Fe)} \\ F_e &= \pi^2 E / (KL/r)^2 \\ F_y &= 50 \text{ ksi}\end{aligned}$$

$$\begin{aligned} E &= 29,000 \text{ ksi} \\ KL_x &= 127.2 \text{ in} \\ r_x &= 2.3162 \text{ in} \\ A_e &= 0.6574 \text{ in}^2 \end{aligned}$$

$$\begin{aligned} KL/r_x &= 127.2 \text{ in} / 2.3162 \text{ in} = 54.92 \\ F_e &= \pi^2 E / (KL/r)^2 = \pi^2 * 29000 / (54.92)^2 = 94.9 \text{ ksi} \\ \lambda_c &= \sqrt{50/94.9} = 0.726 \end{aligned}$$

$$\text{For } \lambda_c = 1.5 \quad F_n = (0.658^{\lambda_c^2}) F_y$$

$$F_n = 0.658^{0.726^2} * 50 \text{ ksi} = 27.23 \text{ ksi}$$

$$\phi_c P_n = \phi_c * F_n * A_e = 0.85 * 27.23 \text{ ksi} * 0.6574 \text{ in}^2 = 15.2 \text{ kips per stud}$$

$$Q_{UF} = 69.4 \text{ kips} / 15.2 \text{ kips per stud} = 4.6 \text{ studs}$$

Studs at 24 in on center, therefore 5 studs available per 8 feet of wall

Removed wall section could be located at edge of exterior wall, or shifted toward the interior bearing walls, therefore 5 studs must be available at either exterior or interior long direction walls. A minimum of 6 studs will be available at the interior long direction walls, therefore the wall stud compression strength is adequate.

Bearing load will be transferred through concrete floor system to allow distribution to multiple studs.

#### Secondary Component Check

The composite floor deck consists of 4" light weight concrete over 2" dovetail deck and must span 18'-8" to be supported by the short direction load bearing walls. Removal of a load bearing wall will result in loss of support for deck. Accordingly, the deck shall be connected to the wall above to prevent collapse in the event of a bearing wall removal.

The floor joists shall be hung from the wall above using a tension tie screwed to the 600S200-68 wall stud above which will include a threaded tension rod through bolted to the deck (See Figure G-6) and including a plate washer to prevent the tension rod from pulling through the deck.

Connection of each floor slab will consist of multiple connectors so it shall be considered force controlled:

Floor loadings:

$$DL = 62 \text{ psf} * 18.67' * 2' = 2315 \text{ lbs}$$

$$LL = 40 \text{ psf} * 18.67' * 2' = 1494 \text{ lbs} \quad (\text{no live load reduction due to limited tributary area for each tension tie})$$

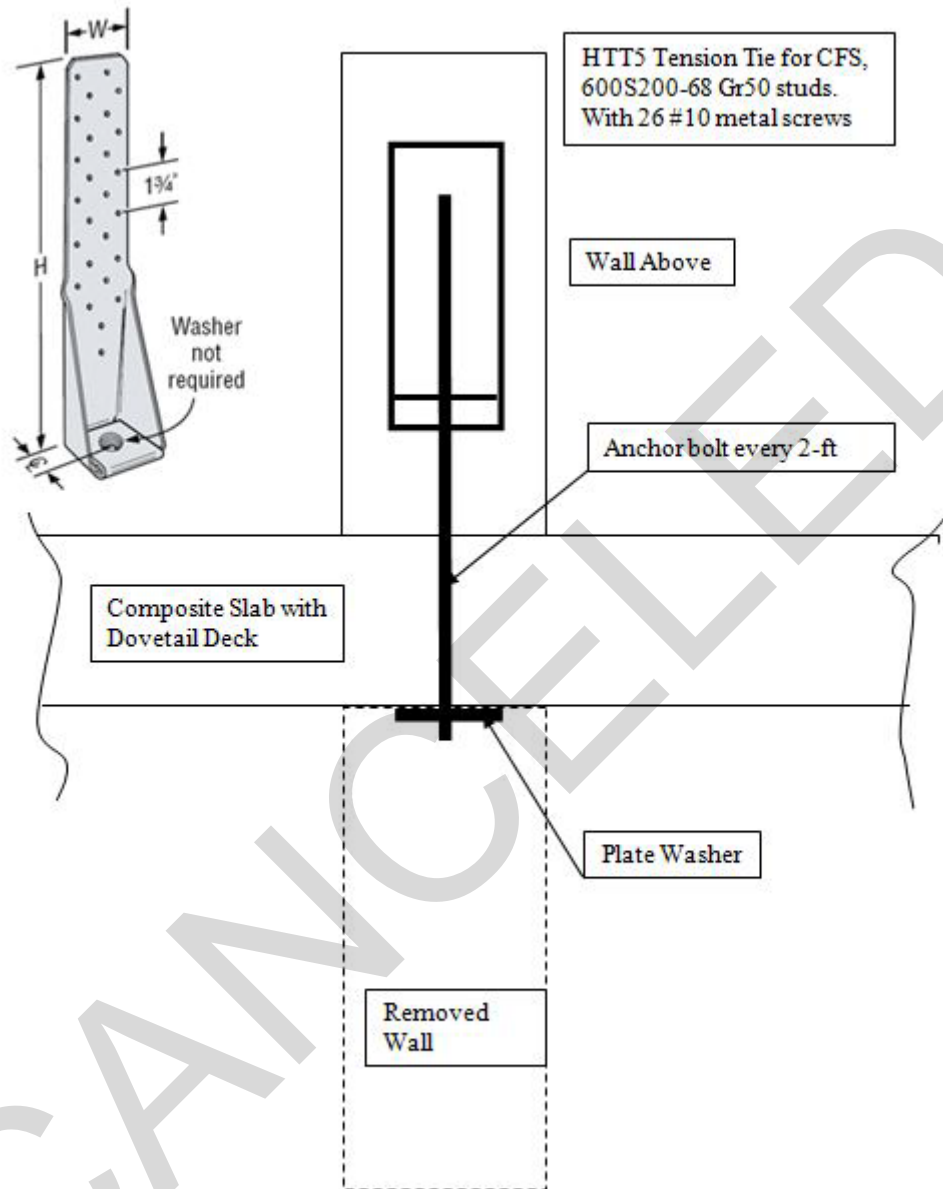
$$\Omega_{LD} = 2.0 \text{ (UFC Table 3-4)}$$

$$G_{LF} = 2.0*(1.2*DL + 0.5*LL)$$

$$G_{LF} = 2.0*(1.2*(2315 \text{ lbs}) + 0.5*(1494)) = 7050 \text{ lbs}$$

The tension tie capacity must be greater than 7050 lbs. Accordingly, provide one (1) Simpson HTT5 with 26 #10 screws into the wall stud above with a capacity of 6505 lbs. Note that 6505 lbs is less than the required capacity, however the nominal tension capacity of the hanger is 11,585 lbs. This capacity, in conjunction with a phi factor of 0.6 will indicate sufficient strength. The 600S200-68 stud has sufficient strength and thickness to support this tension hanger. See Figure G-6 for a detail showing the floor system connection to the wall above.

In order to prevent the through bolt from tearing through the floor system, a plate washer shall be used. Grout packing will be needed at locations for which the washer must bear on the unfilled portion of the dovetail deck. Additionally, typical floor reinforcement in the direction of the deck span shall continue across load bearing wall supports and transverse reinforcement shall be provided above each interior load bearing wall to distribute forces horizontally between each tension hanger connection.



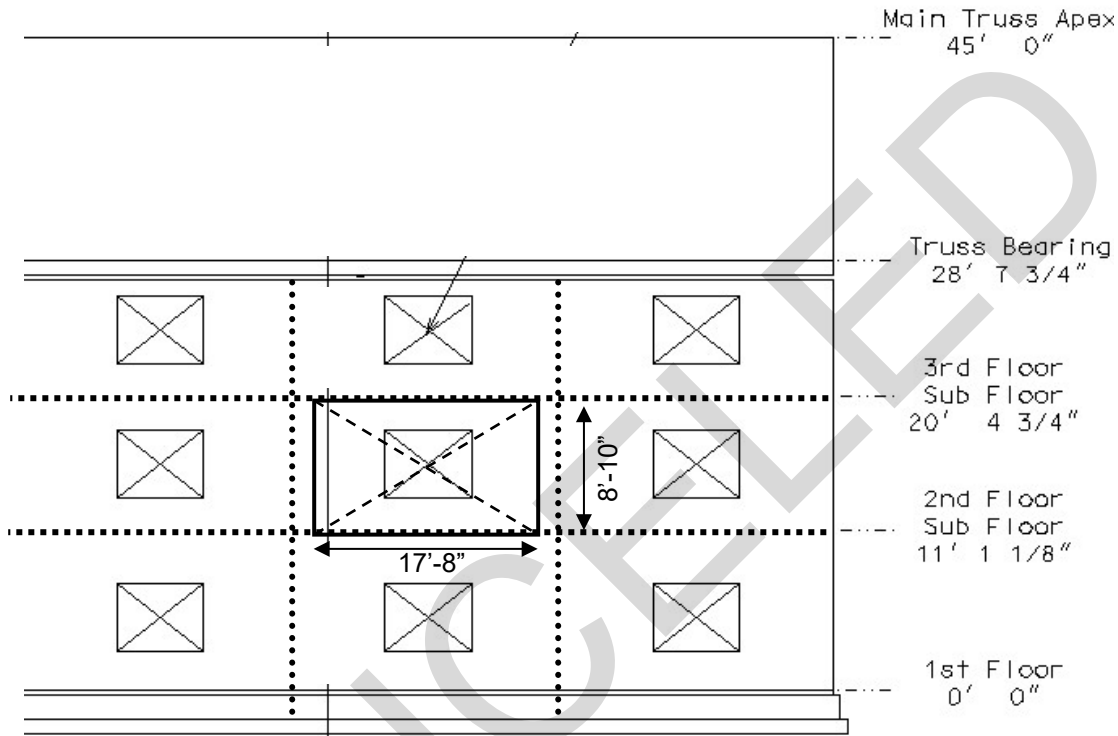
**Figure G-6. Floor System Support Detail**

Removal of a wall segment above the 2<sup>nd</sup> floor would produce similar analysis and results to those described above as the load resisted is roughly proportional to the available depth of remaining wall per level.

Similarly, removal of an exterior load bearing wall along the short side would produce results consistent with the interior removal as the tributary area of floor carried is roughly half that of an interior wall, and the provided capacity will be similar.

### G-3.3 AP Analysis of Removal of Exterior Load Bearing Walls (Long Direction)

As shown in Figure G-7, a segment of wall of length  $2H$  is removed between the second and third floors.  $H$  is taken as the distance between the 2<sup>nd</sup> floor sub-floor elevation and the bottom of the concrete deck at the 3<sup>rd</sup> floor ( $= 8'-10"$ ).



**Figure G-7. Exterior Wall Removal**

The chosen area of removal between the second and the third floor will allow for the shear wall above to span over and be supported on each side of the removed wall section. (Note: shifted removal locations could be addressed similar to the wood building examples of Appendix F).

The primary loading is that of the roof trusses:

Dead Load

Roof:  $25 \text{ psf} \times 25'-11" / 2 = 324 \text{ plf}$

Wall:  $7 \text{ psf} \times 8'-9" = 61 \text{ plf}$

Roof live load not applicable for UFC design combination

Snow Load

$7 \text{ psf} \times 25'-11" / 2 = 91 \text{ plf}$

Linear Static AP Load Case for Deformation Controlled Elements:

$$G_{LD} = \Omega_{LD} [1.2D + 0.2S]$$

From ASCE 41-13 Table 12-3:

for "Wood Structural Panel Sheathing or Siding"

$$h = 8'-10" \quad b = 8'-9"$$

$$h/b = 8.83/8.75 = 1.0 < 2.0$$

$$m = 3.8 \text{ for Life Safety Primary Element}$$

Connections to be screwed:

$$m = 1.8 \text{ for "Screws – Metal to Wood"}$$

$$m = 1.8 \text{ controls}$$

$$\Omega_{LD} = 2.0m = 3.6 \text{ (UFC Table 3-4)}$$

$$G_{LD} = 3.6 * (1.2 * (324 + 61) + 0.2 * (91)) = 3.6 * (480 \text{ plf}) = 1,644 \text{ plf}$$

Total force onto the removed section

$$F = 1644 \text{ lb/ft} * 17.67 \text{ ft} = 29,028 \text{ lbs}$$

$$\text{Shear each side } V = 29,028 \text{ lbs} / 2 = 14,514 \text{ lbs}$$

$$\text{Unit shear} = 14,514 \text{ lbs} / 8.83' = 1644 \text{ plf}$$

For "Wood Structural Panels – Sheathing", 7/16" rated OSB sheathing, fasteners at 4" O.C max at edges:

$$v_{wc} = 1410 \text{ plf (for one side of wall)}$$

Per IBC Table 2211.2, where gyp wall board is applied on opposite side of wall with fastener spacing less than 7" on center, shear strength may be increased by 30%.

$$\phi = 0.55 \text{ for LRFD per IBC 2211.2.1}$$

$$\phi V_s = 0.55 (1410 \text{ plf}) (1.3) = 1008 \text{ plf}$$

$$\phi Q_{CE} = 1.5 (\phi V_s) = 1.5 (1008) = 1512 \text{ plf}$$

Check UFC Equation 3-13:

$$\phi m Q_{CE} \geq Q_{UD}$$

$$1512 \text{ plf} (3.8) = 5746 \text{ plf} > 1644 \text{ plf} \quad \text{OK}$$



For force controlled actions:

$$\Omega_{LD} = 2.0 \text{ (UFC Table 3-4)}$$

$$G_{LF} = 2.0 * (1.2 * (324 + 61) + 0.2 * (91)) = 2.0 * (481 \text{ plf}) = 961 \text{ plf}$$

$$\text{Wall moment} = G_{LF} * L^2 / 8 = 961 \text{ plf} * (17.67')^2 / 8 = 37,507 \text{ lb-ft}$$

$$\text{Chord force} = M/d = 37,507 \text{ lbs} / 8.25' = 4,546 \text{ pounds} = Q_{UF}$$

$$\phi Q_{CL} = 24,025 \text{ pounds for (1) 600T200-54 as established in G-3.2} > 4,546 \text{ pounds OK}$$

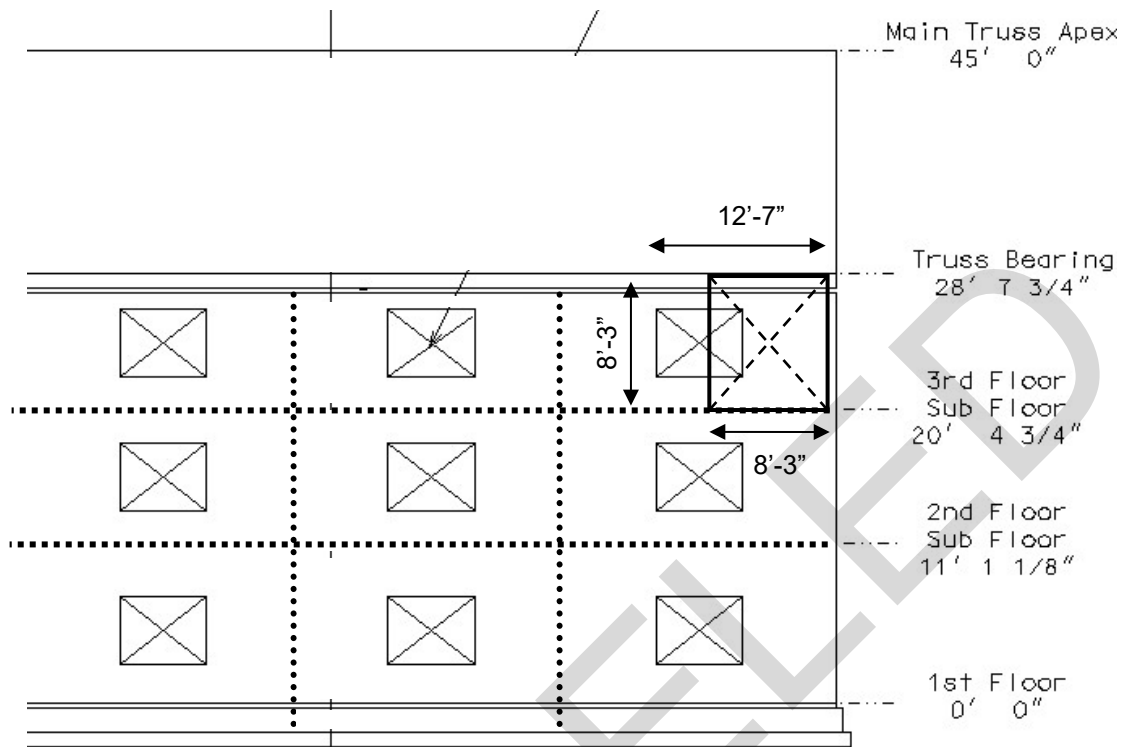
For stud bearing to support bridging walls:

$$Q_{UF} = G_{LF} * L / 2 = 961 \text{ lb/ft} * 17.67 \text{ ft} / 2 = 33,962 \text{ lbs at each side of removed wall}$$

Similar to G-3.2, compression strength of bearing walls is adequate to support bridging wall.

#### **G-3.4 AP Analysis of Removal of Exterior Load Bearing Walls Below Third Floor (Long Direction)**

As shown in Figure G-8, a segment of wall of length H is removed at exterior corners in each direction between the second and third floors. H is taken as the distance between the 3<sup>rd</sup> floor sub-floor elevation and the truss bearing location (=8'-3").



**Figure G-8. Exterior Corner Wall Removal**

The primary loading is that of the roof trusses:

Dead Load

Roof:  $25 \text{ psf} * 25'-11" / 2 = 324 \text{ plf}$

Roof live load not applicable for UFC design combination

Snow Load

$7 \text{ psf} * 25'-11" / 2 = 91 \text{ plf}$

Linear Static AP Load Case for Deformation Controlled Elements:

$$G_{LD} = \Omega_{LD} [1.2D + 0.2S]$$

Resistance will be provided by steel beam(s) at roof level braced at each roof truss at 4'-0" on center as shown in Figure G-9.

From ASCE 41-13 Table 9-5:

for "Frame Components Subjected to Axial Tension and/or Bending"

$m = 8$  for Collapse Prevention Primary Element

Connection for backspan to be similar to double angle shear tab  
Per UFC Table 5-1

$$m = 5.8 - 0.107 d_{bg} \quad d_{bg} = 3''$$

$$m = 5.8 - 0.107 * 3 = 5.5$$

$$m = 5.5 \text{ controls}$$

$$\Omega_{LD} = 0.9 m + 1.1 = 6.0 \text{ (UFC Table 3-4)}$$

$$G_{LD} = 6.0 * (1.2 * (324) + 0.2 * (91)) = 2,463 \text{ plf}$$

Cantilever span = L = 12'-7"

$$Q_{UD} = G_{LD} * L^2 / 2 = 2,463 \text{ plf} * (12.6 \text{ ft})^2 / 2 = 195,513 \text{ lb-ft (2347 k-in)}$$

Use Steel HSS beam  $F_{ye} = 1.1 * 46 \text{ ksi} = 50.6 \text{ ksi}$

Beam supported against lateral and rotational displacement by bolting at each truss bearing post at 48" on center as shown in Figure G-9).

$$\phi m Q_{CE} \geq Q_{UD}$$

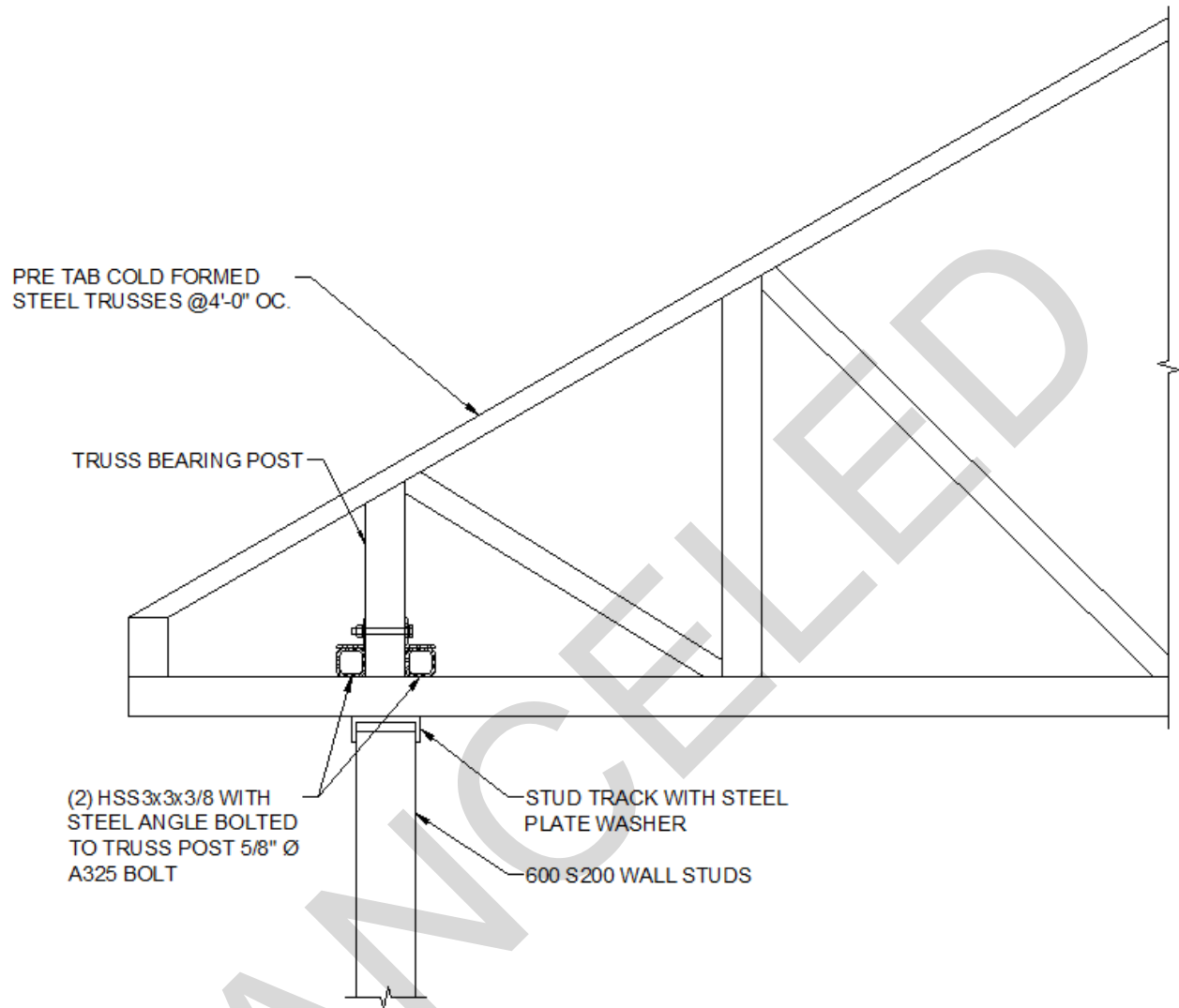
$$\phi m F_{ye} Z = 0.9 * 8 * 50.6 \text{ ksi} * Z \geq 2347 \text{ k-in}$$

$$Z \geq 6.5 \text{ in}^3 \quad \text{Use (2) HSS3x3x3/8"} \quad Z_x = 3.25 \text{ in}^3$$

$$Q_{UD} = 161,539 \text{ lb-ft}$$

$$Q_{UD} / \phi m Q_{CE} = 1.97 \text{ therefore require two (2) HSS3x3x3/8"}$$

Provide (1) HSS3x3x3/8 beam at each side of roof truss vertical bearing post. Stagger the splice of these beams and provide a third identical beam between vertical truss posts at splice location or use a full moment splice at any splice locations.



**Figure G-9. Exterior Roof Beams**

Anchorage of beams required to prevent uplift of the cantilever backspan at 11.4'. Attach beams using double angle connection to truss posts, and utilize hold-down anchors, such as Simpson HD7B at each side of truss post to resist uplift.

Double angle connection between HSS beams and truss shall be deformation controlled with  $m = 6$

$M_{LD} = 195,513 \text{ lb-ft}$   
 $V_{\text{uplift}} = 195,513 \text{ lb-ft} / 11.4' = 17,150 \text{ lbs}$   
 Bolt in double shear – 5/8" diameter A325 bolt  
 Per AISC table 7-1  $\phi r_n = 11 \text{ kips}$   
 $m\phi r_n = 6 * 11 \text{ kips} = 66 \text{ kips} \geq 17.15 \text{ kips}$

Two tie downs shall be used so in accordance with ASCE 41-13 section 12.3.3:  
“demands on nails, screws, lags, bolts, split rings and shear plates used to link wood components to other wood or metal components shall be considered deformation controlled actions. Demands on bodies of connections and bodies of connection hardware shall be considered force controlled actions”

Accordingly, since 2 tie downs will be used, the action shall be considered force controlled.

$$\Omega_{LD} = 2.0 \text{ (UFC Table 3-4)}$$

$$G_{LF} = 2.0 * (1.2 * (324) + 0.2 * (91)) = 2.0 * (407 \text{ plf}) = 814 \text{ plf}$$

$$\text{Cantilever moment} = G_{LF} * L^2 / 2 = 814 \text{ plf} * (12.6 \text{ ft})^2 / 2 = 64615.3 \text{ lb-ft}$$

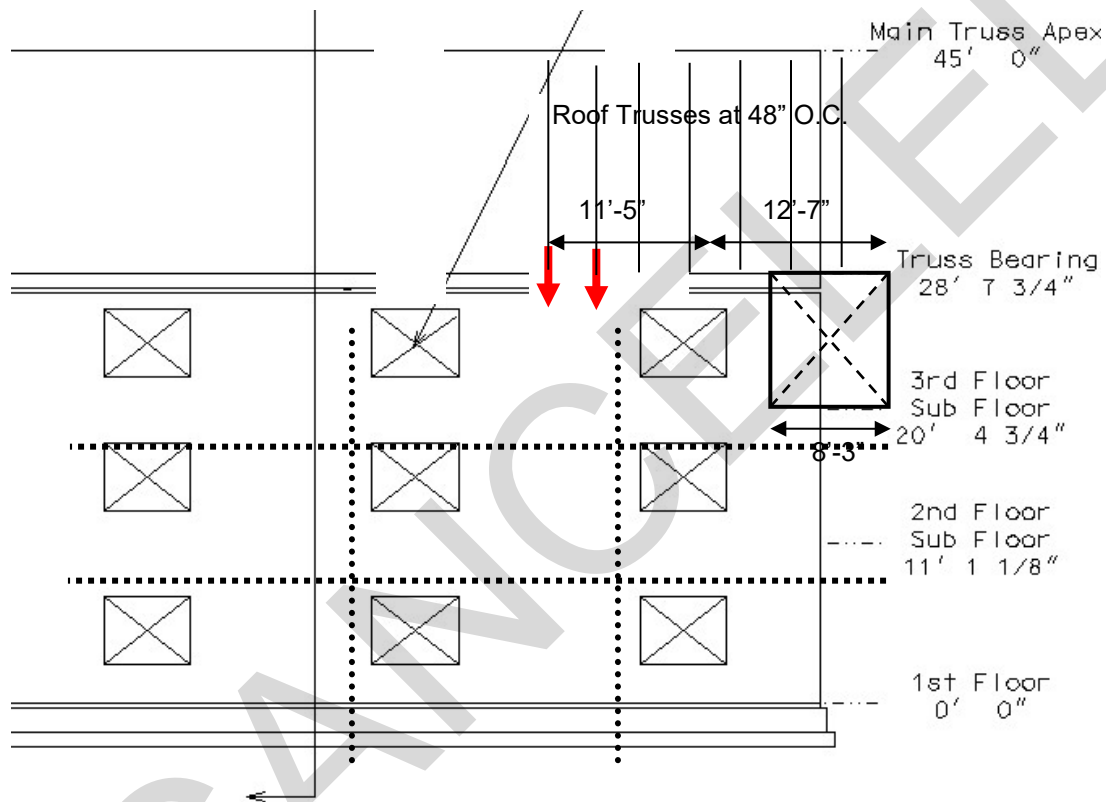
$$\text{Backspan distance} = 11'-5" = 11.4 \text{ ft}$$

$$\text{Uplift reaction} = 64615.3 \text{ lb-ft} / 11.4 \text{ ft} = 5668.8 \text{ lbs}$$

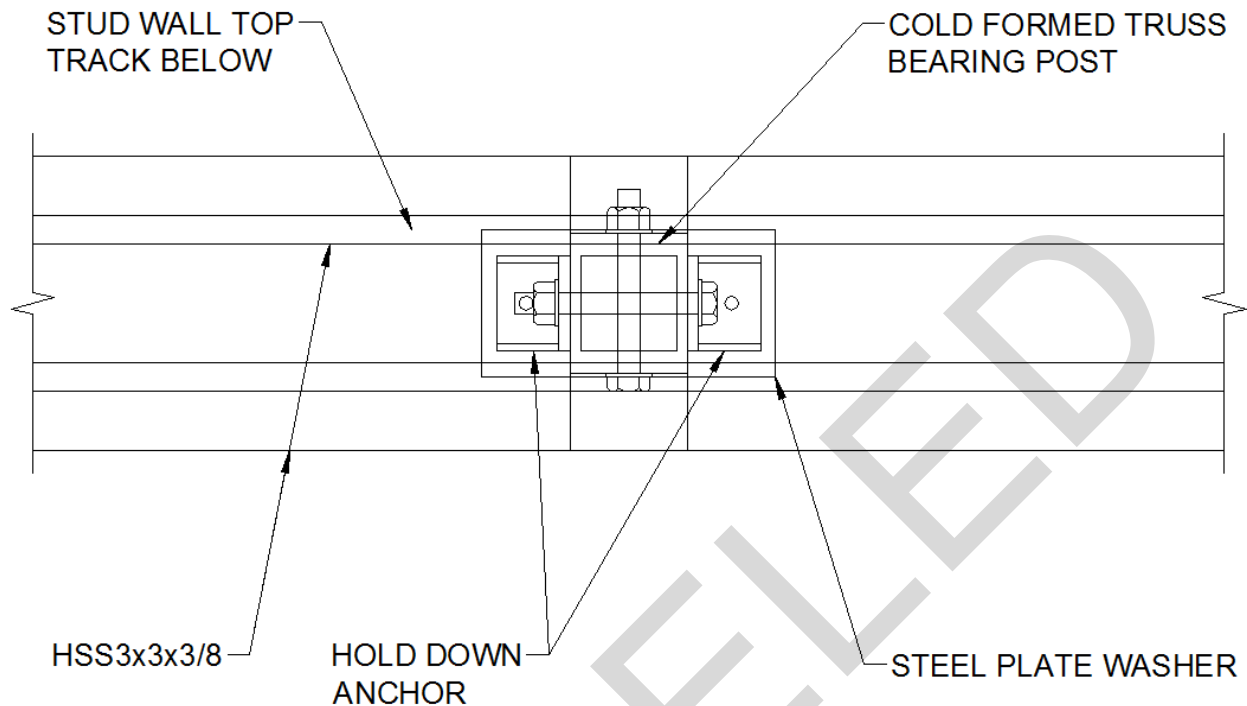
$$\text{Capacity of Simpson HD7B} = 6645 \text{ lbs}$$

A single anchor would suffice; however provide 2 anchors to prevent connection eccentricity. Include a steel plate washer at wall header track to prevent anchor tension pullout.

See Figure G-10 for tie down regions and Figure G-11 for anchorage configuration.



**Figure G-10. Exterior Corner Wall Removal Tie Down Locations**



**Figure G-11. Roof Beam Tie Down Anchorage, Plan View**

Stud bearing to support bridging beam shall be force controlled:

Stud compression in lowest level:

$$Q_{UF} = G_{LF} * L + \text{Uplift reaction} + \text{additional wall load}$$

Additional wall load carried by each stud to the foundation

$$h = 28'-8" = 28.67' \quad \text{stud spacing} = 2 \text{ ft} \quad \text{wall DL} = 7 \text{ psf}$$

$$\text{Additional wall load} = 2.0 * (0.5 * 7 \text{ psf} * 28.67') = 201 \text{ lbs / ft} * 2 \text{ ft} = 402 \text{ lbs}$$

$$Q_{UF} = 814 \text{ lb/ft} * 12.6 \text{ ft} + 5669 \text{ lbs} + 402 \text{ lbs} = 16,328 \text{ lbs}$$

Compression carried by 600S200-68 studs at studs at 2' on center with in-plane bracing provided by sheathing and blocking. Check as column for out-of-plane strength of lower story

Per G-3.2 Stud strength is 15,200 lbs / stud

$$16,328 \text{ lbs} / 15,200 = 1.1 \text{ studs}$$

2 studs available at each window jamb **OK**

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## APPENDIX H /2/INTERNATIONAL BUILDING CODE MODIFICATIONS FOR CONSTRUCTION OF BUILDINGS TO RESIST PROGRESSIVE COLLAPSE

The following narrative identifies required modifications to the provisions of the *International Building Code* (IBC 2015) addressing construction documents, structural tests and special inspections for buildings that have been designed to resist progressive collapse. The modifications reference specific sections in the IBC that require modification. Apply IBC requirements except as modified herein. The required IBC modifications are one of two actions, according to the following legend:

### LEGEND FOR ACTIONS

**[Addition]** -- New section added, includes new section number not shown in IBC.

**[Replacement]** -- Delete referenced IBC section and replace it with the narrative shown.

## Chapter 16 Structural Design

### 1603 Construction Documents

**1603.1.9 [Addition] Progressive Collapse design data.** The following information shall be indicated on the construction documents:

1. General note stating the follow:

Design of the building is in accordance with UFC 4-023-03, DD/MM/YYYY. Future additions or alterations to this structure shall not jeopardize the requirements for progressive collapse resistance.

2. \3\ Risk Category /3/ II, III or IV.
3. Method of progressive collapse resistance (Tie Force, Alternate Path, Enhanced Local Resistance or combinations thereof).

**1603.1.10 [Addition] Systems and components requiring special inspections for progressive collapse resistance.** Construction documents or specifications shall be prepared for those systems and components requiring special inspection for progressive collapse resistance and shall be submitted for approval as specified in section 1705.11 by the registered design professional responsible for their design on 107.1.

## Chapter 17 Structural Tests and Special Inspections

**1701.1 [Replacement] Scope** The provisions of this chapter shall govern the quality, workmanship and requirements for materials covered. Materials of construction and tests shall conform to the applicable standards listed in this code.

### **1710 [Addition] QUALITY ASSURANCE FOR PROGRESSIVE COLLAPSE REQUIREMENTS**

**1710.1 [Addition] Scope** A quality assurance plan shall be provided in accordance with Section 1710.1.1.

**1710.1.1 [Addition] When required.** A quality assurance plan for progressive collapse requirements shall be provided for the following structures designed for various Risk categories as follows:

1. Structures designed for \3\ Risk Category /3/ II, where either 1) structural elements provide horizontal and vertical tie force capacity as well as additional ductility requirements in which the shear resistance of the corner and penultimate first story walls and columns is greater than the flexural resistance for lateral loads, or, 2) the alternate path method is used to provide bridging over the deficient elements.
2. Structures designed for \3\ Risk Category /3/ III, with horizontal and vertical tie forces, alternate path design, and enhanced local resistance where the columns or walls in the first two perimeter stories are designed for increased flexural and shear resistance.
3. Structures designed to \3\ Risk Category /3/ IV, which requires a design based on the results of a systematic risk assessment of the building.

**1710.1.2 [Addition] Detailed requirements.** When required by Section 1710.1.1, a quality assurance plan shall provide for the following:

1. Horizontal and vertical tie force connections as required based on material type.
2. Roof and floor diaphragm systems including transverse, longitudinal, and peripheral ties.
3. Vertical progressive collapse resisting systems including vertical ties and bridging connections.
4. Perimeter ground floor columns and walls with enhanced ductility requirements to ensure shear strength is greater than the flexural strength

**1710.2 [Addition] Quality assurance plan preparation.** The design of each designated progressive collapse resisting system shall include a quality assurance plan prepared by the registered design professional. The quality assurance plan shall identify the following:

1. The designated progressive collapse resisting systems and elements that are subject to quality assurance in accordance with 1710.1.
2. The special inspections and testing to be provided as required by sections 1704 and other applicable sections of this code, including the applicable standards reference by this code.
3. The type and frequency of testing required.
4. The type and frequency of special inspections required.
5. The required frequency and distribution of testing and special inspection reports.
6. The structural observations to be performed.
7. The required frequency and distribution of structural observation reports.

**1710.3 [Addition] Contractor responsibility.** Each contractor responsible for the construction of the progressive collapse resisting system or progressive collapse component listed in the quality assurance plan shall submit a written contractor's statement of responsibility to the contracting officer prior to the commencement of work on the system or component. The contractor's statement of responsibility shall contain the following.

1. Acknowledgement of awareness of the special requirements contained in the quality assurance plan;
2. Acknowledgement that control will be exercised to obtain conformance with the construction documents approved by the building official;
3. Procedures for exercising control within the contractors organization, the method and frequency of reporting the distribution of reports; and
4. Identification and qualification of the person(s) exercising such control and their position(s) in the organization.

## **1711 [Addition] SPECIAL INSPECTIONS FOR PROGRESSIVE COLLAPSE RESISTANCE**

**1711.1 [Addition] General.** Special inspections for progressive collapse resistance shall follow the requirements of Section 1704.1. Special inspections itemized in Sections 1717.2 through 1717.4 are required for the following:

1. Structures designed for Occupancy Category II, where either 1) structural elements provide horizontal and vertical tie force strength as well as additional ductility requirements in which the shear resistance of the corner and penultimate ground floor walls and columns is greater than the flexural resistance for lateral loads, or, 2) the alternate path method is used to provide bridging over the deficient elements.
2. Structures designed for Occupancy Category III, with horizontal and vertical tie forces, alternate path design, and enhanced local resistance where the columns or walls in the first two perimeter stories are designed for increased flexural and shear resistance.
3. Structures designed to Occupancy Category IV, which requires a design based on the results of a systematic risk assessment of the building.

**1711.2 [Addition] Structural steel.** Continuous special inspection for structural welding in accordance with AWS D1.1, including floor and roof deck welding.

**Exemptions:**

1. Single pass fillet welds not exceeding 5/16" (7.9mm) in size.

**1711.3 [Addition] Structural Wood.** Periodic special inspections during nailing, bolting, anchoring and other fastening of components within the progressive collapse resisting system, including horizontal tie force elements, vertical tie force elements and bridging elements.

**1711.4 [Addition] Cold-formed steel framing.** Periodic special inspections during welding operations, screw attachment, bolting, anchoring and other fastening of components within the progressive collapse resisting system, including horizontal tie force elements, vertical tie force elements and bridging elements.

**1711.5 [Addition] Cast-in-place concrete.** Continuous special inspection for reinforcing steel placement with a particular emphasis on reinforcing steel anchorages, laps and other details within the progressive collapse resisting system, including horizontal tie force elements, vertical tie force elements and bridging elements.

## **1712 STRUCTURAL OBSERVATIONS**

**1712.1 [Addition] Structural observations.** Structural observations shall be provided for the progressive collapse resisting systems as follows:

1. When the contracting officer requires such observation.
2. In structures designed to \3\ Risk Category /3/ IV