Progressive Collapse Analysis and Design Guidelines

for

New Federal Office Buildings
and
Major Modernization Projects

June 2003

TABLE OF CONTENTS

Preface

The U.S. General Services Administration (GSA) developed the "Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects" to ensure that the potential for progressive collapse is addressed in the design, planning and construction of new buildings and major renovation projects. Mr. Bruce Hall, P.E., of the Office of the Chief Architect, initiated this work in 1999 and served as the GSA Project Manager. The Guidelines, initially released in November 2000, focused primarily on reinforced concrete structures. GSA subsequently identified the need to update the November 2000 Guidelines to address the progressive collapse potential of steel frame structures. Preparation of the updated Guidelines was performed by Applied Research Associates, Inc. with assistance provided by Myers, Houghton & Partners, Inc., Simpson, Gumpertz and Heger, Inc., the U.S. Army Corps of Engineers, and the U.S. Department of State.

Section 1. General Requirements

1.1 Purpose

The purpose of these Guidelines is to:

- Assist in the reduction of the potential for progressive collapse in new Federal Office Buildings
- Assist in the assessment of the potential for progressive collapse in existing Federal Office Buildings
- Assist in the development of potential upgrades to facilities if required

To meet this purpose, these Guidelines provide a *threat independent* methodology for minimizing the potential for progressive collapse in the design of new and upgraded buildings, and for assessing the potential for progressive collapse in existing buildings. It should be noted that these Guidelines are not an explicit part of a blast design or blast analysis, and the resulting design or analysis findings cannot be substituted for addressing blast design or blast analysis requirements. The requirements contained herein are an independent set of requirements for meeting the provisions of Interagency Security Committee (ISC) Security Criteria regarding progressive collapse. The procedures presented herein are required for the treatment of progressive collapse for U.S. General Services Administration (GSA) facilities.

The previous guidelines, "Progressive Collapse Analysis and Design for New Federal Office Buildings and Major Modernization Projects", November 2000 focused primarily on analysis and design for progressive collapse of reinforced concrete structures. This update includes lessons learned and adds a separate section pertaining to structural steel buildings.

1.2 Applicability

These Guidelines should be used by all professionals engaged in the planning and design of new facilities or building modernization projects for the GSA. It applies to in-house Government engineers, architectural/engineering (A/E) firms and professional consultants under contract to the GSA. The primary users of the document will be architects and structural engineers. While mandatory for GSA facilities, these Guidelines may also be used and/or adopted by any agency, organization, or private concern.

The exemption process contained in these Guidelines applies to the majority of the construction types currently in the GSA building inventory as described in Section 3. The analysis and design guidance for considering non-exempt, new or existing facilities is described in Sections 4 and 5.

The use of a simplified analysis approach (hereafter referred to in these Guidelines as a "*Linear Procedure*") should typically be limited to consideration of low-to-medium-rise facilities. A Linear Procedure implies the use of either a static or dynamic linear-elastic finite element analysis. Typically such facilities consist of buildings and specialty structures that are nominally 10 stories above grade or less. However, when analyzing buildings that have more than 10 stories above grade, and/or exhibit an atypical structural configuration (as defined in Section 2.1 and elaborated on in Appendix A), project engineers should consider using a more sophisticated analysis method hereinafter referred to as a "*Nonlinear Procedure*".

A Nonlinear Procedure implies the use of static or dynamic finite element analysis methods that capture both material and geometric nonlinearity. Special attention should be taken for facilities that contain atypical structural configurations and/or high-rise buildings that may exhibit complex response modes for the case where a primary vertical element is instantaneously removed. If a complex structural response to the analysis process contained in these Guidelines is anticipated, a Nonlinear Procedure may be required. It should be noted that if a Nonlinear Procedure is utilized, the approach must be based on the intent of these Guidelines and use the same allowable extents of collapse area as that presented in Section 4 and Section 5 in the evaluation of the potential for progressive collapse. As such, the assessment team will require experience and demonstrated expertise in structural dynamics, abnormal loading, and nonlinear structural response. Additionally, the applied procedure will require approval by the project Contracting Officer Technical Representative (COTR).

1.3 Guideline Philosophy

These Guidelines address the need to protect human life and prevent injury as well as the protection of Federal buildings, functions and assets. The Guidelines take a flexible and realistic approach to the reliability and safety of Federal buildings.

The ISC Security Criteria requires all newly constructed facilities to be designed with the intent of reducing the potential for progressive collapse, regardless of the required level of protection determined in the facility-specific risk assessment. Similarly, existing facilities shall be evaluated to determine the potential for progressive collapse.

The approach described below utilizes a flow-chart methodology to determine if the facility under consideration might be exempt from detailed consideration for progressive collapse, as illustrated in Figure 1.1. In other words, a series of questions must be answered that identify whether or not further progressive collapse considerations are required. This process is based on ascertaining certain critical documentation to ensure that resources are spent wisely regarding this issue. Critical documentation consists of identifying <u>all</u> of the following information:

- Building occupancy
- Building category (e.g., reinforced concrete building, steel frame building, etc.)
- Number of stories
- Seismic zone
- Detailed description of *local* structural attributes [discrete beam-to-beam continuity, connection redundancy, and connection resilience, as defined under Section 2.1]

- Description of significant *global* structural attributes [single point failure mechanism(s), structural irregularities, etc.]

The outcome of these answers leads to either (1) an *exemption* (no further consideration required) or (2) the need to further consider the potential for progressive collapse. The detailed analysis required in the latter case is intended to reduce the probability of progressive collapse for new construction and identify the potential for progressive collapse in existing construction.

These Guidelines present the methodology and performance criteria for these determinations without prescribing the exact manner of design or analyses. As such, the architect/engineer may apply methods appropriate to the facility at hand.

A threat independent approach is, however, prescribed as it is not feasible to rationally examine all potential sources of collapse initiation. The approach taken (i.e., the removal of a column or other vertical load bearing member) is not intended to reproduce or replicate any specific abnormal load or assault on the structure. Rather, member removal is simply used as a "load initiator" and serves as a means to introduce redundancy and resiliency into the structure. The objective is to prevent or mitigate the potential for progressive collapse, not necessarily to prevent collapse initiation from a specific cause. Regardless of other specific design requirements, (e.g., blast design, seismic design, impact design, fire design, etc.) there are always scenarios that will be capable of initiating a collapse. For example, say a building is rammed by an 18-wheeler taking out 3 columns and collapsing several structural bays over 2 floors. These Guidelines and the provisions made herein would undoubtedly reduce the extent of this initially collapsed area. More importantly, however, provisions of these Guidelines should serve to help arrest the progression of the collapse and should reduce the extent of the damage. The strategy places a premium on well designed continuity as well as post event capacity, ductility and robustness as compared with just using key element resistance. No special value is placed on using more robust columns that can survive a particular threat or the use of larger spans to avoid multiple column failure from a specific point threat.

1.4 How To Use This Document

The intent of this document is to provide guidance to reduce and/or assess the potential for progressive collapse of Federal buildings, for new or existing construction, respectively. It should be noted that the use of a Linear Procedure, as provided for in these Guidelines, is not intended for and not capable of predicting the detailed response or damage state that a building may experience when subjected to the instantaneous removal of a primary vertical element. However, a Linear Procedure, albeit a simplified methodology, may, with proper judgment, be used for determining the *potential* for progressive collapse (i.e., a high or low potential for progressive collapse), providing the acceptance criteria accounts for the uncertainties in behavior in the form of appropriate Demand-Capacity Ratios. The owner, architect, and project engineer should be thoroughly familiar with the provisions of these Guidelines, as it applies to their specific facility.

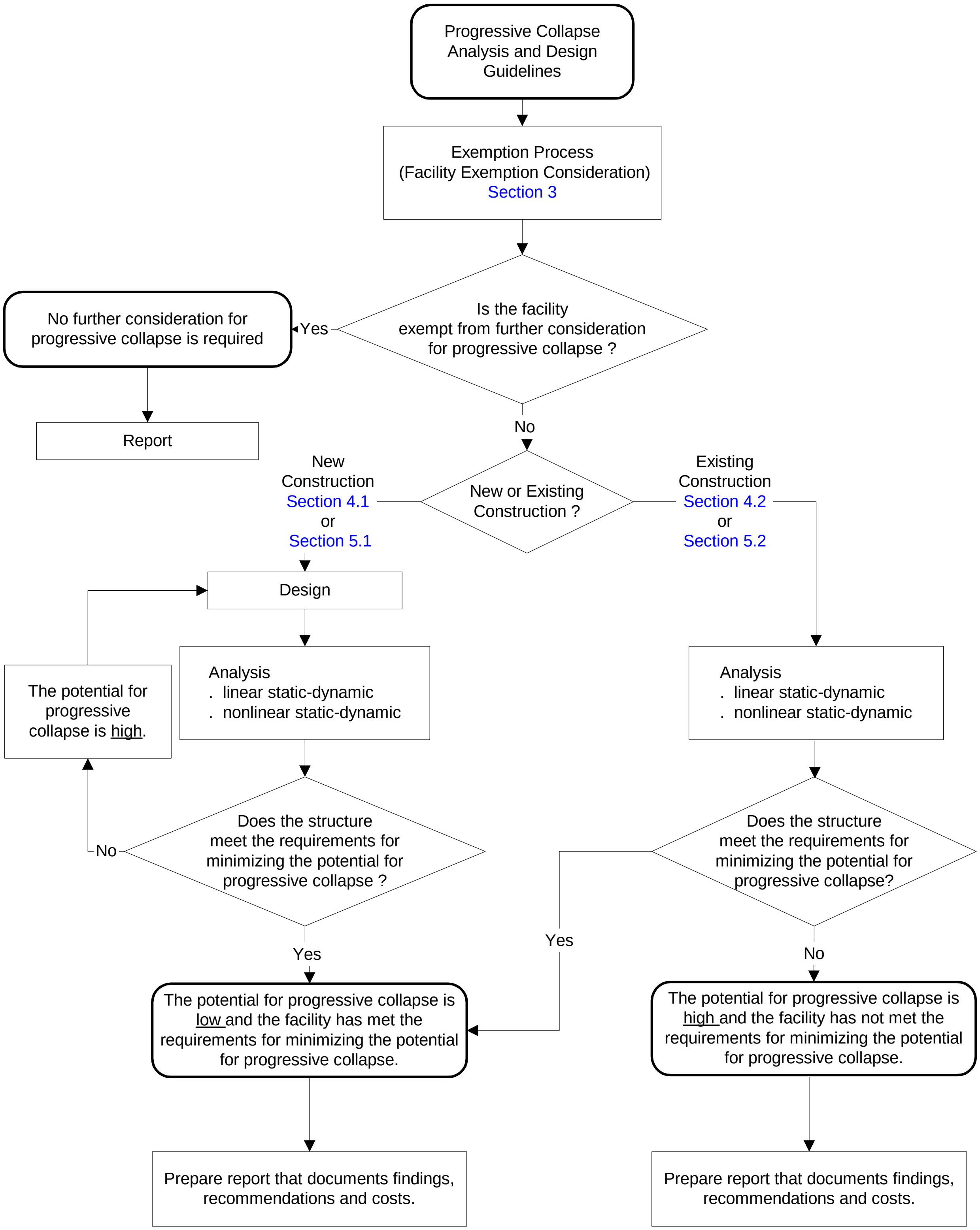


Figure 1.1. Overall flow for consideration of progressive collapse.

The first step of the process is to evaluate the facility using the methodology outlined in Section 3 of these Guidelines to determine if the facility might be exempt from further consideration for progressive collapse. If the facility is determined to be exempt, the process concludes with documentation of the exemption process.

For *new construction*, if the facility is determined not to be exempt from further consideration for progressive collapse, the methodology for new construction outlined in Section 4.1 or Section 5.1, as applicable, shall be executed. This process provides design guidance and evaluation guidance for determining the potential for progressive collapse. If the potential for progressive collapse is found to be high for a given design, a redesign must be executed. When the criteria are satisfied (i.e., the potential for progressive collapse is low), the process concludes with documentation of the analysis procedure and results.

For *existing construction*, if the facility is determined not to be exempt from further consideration for progressive collapse, the methodology for existing construction outlined in Section 4.2 or 5.2, as applicable, shall be executed. The potential for progressive collapse determined in this process (whether low or high) must be quantified and the analysis procedure and results documented.

1.5 Documentation Requirements

The entire evaluation process shall be documented to a level such that the conclusions can be independently verified by in-house designers or outside firms and professionals under contract to the GSA. This generally will involve providing adequate support for the "answers" to each question in the process in the form of a report. The ***STANDGARD*** (**Stand**ard **G**SA **A**ssessment **R**eporter & **D**atabase) software program shall be used for preparing progressive collapse assessment reports for all existing GSA buildings, to ensure that such reports contain the same type of information for each building assessed, and can be easily compared to other assessed buildings as part of a common database. ***STANDGARD*** may also be used for reporting on new and upgraded building designs. Information on ***STANDGARD*** and access to the program may be obtained at the website www.oca.gsa.gov.

For the exemption process, the answer to each question in the flow diagram should be supported by a written description and graphics (as may be appropriate) to fully describe the conclusion. For example, the initial consideration in the exemption process will generally require the following supporting information: (1) the site plan showing minimum defended standoff distances, (2) a description of the overall construction type, (3) a description of both the *global* and *local* structural attributes as they may affect progressive collapse and (4) the level of protection required. From this information, the conclusion as to whether or not adequate standoff is sufficient to justify an exemption from further consideration of progressive can be determined from Table 3.1.

Answers to subsequent questions in the flow diagram will require either a similar or even a higher level of supporting detail, depending on the material category of a given building (e.g., reinforced concrete building vs. steel frame building), particularly as to structural considerations.

The written description of global attributes shall include column spacing, building height and number of stories, and any significant structural irregularities. The written description of local attributes shall include a detailed summary of essential connection elements and a detailed description and sketch of the *geometry* of those elements as they affect the ability to maintain structural continuity across a removed vertical element, and to achieve a resilient and robust design. In particular, the project engineer is required to explicitly describe how the essential attributes of *discrete beam-to-beam continuity* across a column, *connection redundancy,* and *connection resilience* (as defined in Section 2.1) are achieved in any proposed new or retrofit upgrade design, as well as how they are achieved or not achieved individually and collectively for assessments of existing buildings.

In particular, for steel frame buildings, the reporting analyst shall first describe the beam-to-column *connection type*, using a general description such as those used in Appendix D. The reporting analyst shall then describe those connection attributes (favorable or otherwise) that directly affect the connection's ability to maintain independent structural beam-to-beam continuity across a removed column, similar to the language used to characterize the term "symmetric reinforcement" in Section 2.1 for reinforced concrete buildings. Favorable attributes include the use of creative detailing of the geometry of connection elements (i.e., judicious configuring of weld orientations, selection of weld types, and/or orientation of bolt groups) to achieve *discrete beam-to-beam continuity* across a column, coupled with *connection redundancy* to ensure a multiplicity of clearly defined load paths, and with increased torsional strength and minor-axis bending strength to provide overall *connection resilience.*

Section 2. Definitions

The following definitions apply to terms used throughout these Guidelines. General terms are defined in Section 2.1, frangible/non-frangible façade is described in detail in Section 2.2, and alternate analysis methods are discussed in Section 2.3.

2.1 General Terms

Abnormal Loads – Loads other than conventional design loads (dead, live, wind, seismic, etc.) for structures such as air blast pressures generated by an explosion or impact by vehicles, etc.

Allowable Extent of Collapse (Exterior Consideration) – The extent of damage resulting from the loss in support of an exterior primary vertical load-bearing member that extends one floor above grade (one story) shall be limited. Explicit limitations for damage to primary and secondary structural components are defined in Sections 4 and 5.

Allowable Extent of Collapse (Interior Consideration) - The extent of damage resulting from the loss in support of an interior primary vertical load-bearing member that extends one floor above grade (one story) shall be limited. Explicit limitations for damage to primary and secondary structural components are defined in Sections 4 and 5.

Alternate Analysis Techniques – Sophisticated analysis methods (e.g., nonlinear, dynamic finite element analysis, etc.) that may be used to determine the potential for progressive collapse in a given facility. Requirements and further discussion of this topic is included in Section 2.3.

Atypical Structural Configuration – A structural configuration that has distinguishing features or details. A detailed discussion of atypical structural configurations is presented in Appendix A.

Connection Redundancy – A beam-to-column connection that provides direct, multiple load paths through the connection.

Connection Resilience – A beam-to-column connection exhibiting the ability to withstand rigorous and destructive loading conditions that accompany a column removal, without rupture. This ability is facilitated by the connection's torsional and weak-axis flexural strength, its robustness, and its primary use of proven ductile properties of a given construction material.

Defended Standoff Distance – The defended standoff distance is the range between a point along the defended perimeter and the nearest structural element.

Defended Perimeter – The defended perimeter is the line that defines the boundaries of defended standoff zones (Figure 2.1). Parking within this defended zone must be limited to cleared employees or other controlled parking as defined by the ISC Security Criteria. In addition, security countermeasures (i.e., an automatic vehicle identification (AVI). system, a prescreening system, etc.) must be in place, if parking is allowed in this zone, to reduce the potential for the delivery of an explosive device into this defended area. In order for the perimeter to be considered defended, as a minimum, vehicle barriers capable of stopping the Medium Level Protection vehicle explosive threat (defined in the ISC Security Criteria) must be in place, unless a Higher Level of Protection is specified. Vehicle barriers such as bollards, planters, retaining walls, landscaping, etc., can be designed to stop a vehicle of the specified weight and speed consistent with the criteria.

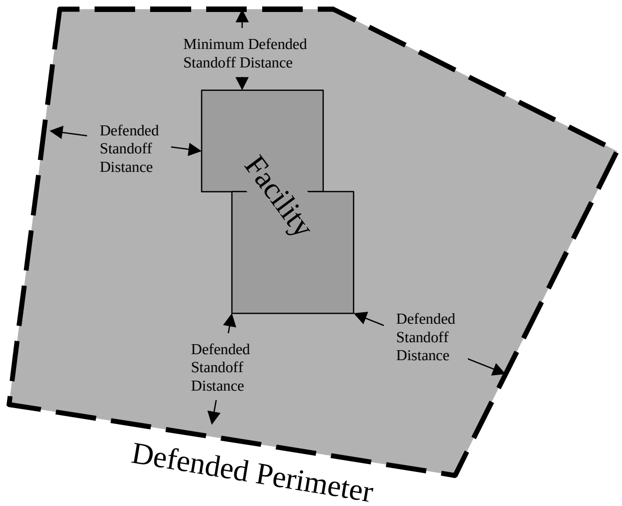


Figure 2.1. Illustration depicting defended perimeter and defended standoff distances.

Discrete Beam-to-Beam Continuity – A distinct, clearly defined beam-to-beam continuity link across a column, for steel frame beam-to-column connection applications, that is capable of independently transferring gravity loads for a removed column condition, regardless of the actual or potential damage state of the column.

Exemption Procedure - A facility exemption process is offered for both new and existing construction. This process presents the designer/analyst with an outlet to further consideration of progressive collapse if the facility possesses structural and/or site characteristics that enable the facility to be considered a low potential for progressive collapse.

Frangible Façade - An exterior façade system (wall systems, window systems, etc.) that has an ultimate, unfactored flexural capacity that is less than 1.0 psi. Refer to Section 2.2 for a more detailed description.

High Potential for Progressive Collapse – The facility is considered to have a high potential for progressive collapse if analysis results indicate that the structural member(s) and/or connections are not in compliance with the appropriate progressive collapse analysis acceptance criteria.

ISC Higher Level Protection - Minor damage, repairable. The facility or protected space may globally sustain minor damage with some local significant damage possible. Occupants may incur some injury, and assets may receive minor damage.

ISC Low and Medium/Low Level Protection - Major damage. The facility or protected space will sustain a high level of damage without progressive collapse. Casualties will occur and assets will be damaged. Building components, including structural members, will require replacement, or the building may be completely unrepairable, requiring demolition and replacement.

ISC Medium Level Protection - Moderate damage, repairable. The facility or protected space will sustain a significant degree of damage, but the structure should be reusable. Some casualties may occur and assets may be damaged. Building elements other than major structural members may require replacement.

Linear Procedure - A Linear Procedure is a simplified analysis approach, and implies the use of either a static or dynamic linear-elastic finite element analysis.

Low Potential for Progressive Collapse –The facility is considered to have a low potential for progressive collapse if analysis results indicate that the structural member(s) and/or connections are in compliance with the appropriate progressive collapse analysis acceptance criteria. Such facilities may be exempt from any further consideration of progressive collapse.

Non-Frangible Facade – An exterior façade system (wall systems, window systems, etc.) that has an ultimate, unfactored flexural capacity that is greater than or equal to 1.0 psi. Refer to Section 2.2 for a more detailed description.

Nonlinear Procedure - A Nonlinear Procedure is a more sophisticated analysis approach, and implies the use of either static or dynamic elasto-plastic finite element analysis methods that capture both material and geometric nonlinearity. It is generally a more accurate analysis approach than are Linear Procedures to characterizing the damage state of a structure. When such procedures are used, less restrictive acceptance criteria (Table 2.1) are permitted, in recognition of the improved response information that can be obtained from such procedures when employed by highly trained analysts.

Primary Structural Elements – As defined by the ISC Security Criteria, the primary structural elements are the essential parts of the building's resistance to abnormal loads and progressive collapse, including columns, girders, roof beams, and the main lateral resistance system.

Primary Non-Structural Elements – As defined by the ISC Security Criteria, the primary non-structural elements that are considered are all elements (including their attachments) that are essential for life safety systems or elements that can cause substantial injury if failure occurs, including, but not limited to, ceilings or heavy suspended mechanical units.

Progressive Collapse - Progressive collapse is a situation where local failure of a primary structural component leads to the collapse of adjoining members which, in turn, leads to additional collapse. Hence, the total damage is disproportionate to the original cause.

Qualified Blast Engineer/Consultant – Consultant should have formal training in structural dynamics, and demonstrated experience with accepted design practices for blast resistant design and with referenced technical manuals (Figure 3.3). To be considered qualified, a blast consultant should have a minimum of 5 years of demonstrated experience in the design and assessment of facilities subjected to blast loads as well as in the testing and evaluation of hazard mitigating products.

Robustness – Ability of a structure or structural components to resist damage without premature and/or brittle failure due to events like explosions, impacts, fire or consequences of human error, due to its vigorous strength and toughness.

Secondary Structural Elements – As defined by the ISC Security Criteria, the secondary structural elements are all other load bearing members (not included in the primary structural elements category), such as floor beams, slabs, etc.

Secondary Non-Structural Elements – As defined by the ISC Security Criteria, the secondary non-structural elements are all elements not covered in primary non-structural elements, such as partitions, furniture, and light fixtures.

Single Point Failure Mechanism – A structural feature in which a localized structural failure can lead to a widespread collapse of the structure. A primary example includes the use of transfer girders (i.e., beams or girders that typically provide vertical support for intermediate columns or load bearing members located above, hence, transferring load to the load bearing members supporting the girder). Another example includes exposed perimeter columns.

Symmetric Reinforcement – Symmetric reinforcement is defined here as having continuous (i.e., no lap splices across a column) and equal amounts of main reinforcing steel in both the compressive and tension faces of a reinforced concrete girder or beam, across a column.

Traditional Moment Connection – A 'traditional' moment connection is defined here as a steel frame moment-resisting beam-to-column connection that 1) typically joins beam or girder flanges directly to the face of a column flange in the field by using either a complete joint penetration (CJP) groove weld in a T-joint configuration, and/or 2) may significantly depend on panel zone participation from the column's web to achieve its rotational capacity.

Typical Structural Configuration – A typical structural configuration consists of a structural layout that is generally simple and contains no atypical structural configuration arrangements.

Ultimate, Unfactored Capacity – The calculated flexural, shear, and axial capacities with no use of a capacity reduction factor (i.e., $\phi = 1.0$).

Uncontrolled Public Areas – These are areas located at the ground floor or entry level that are utilized by retail and other users and have no, or inadequate, operational security countermeasures in place. The concern for this situation is that an explosive device could be brought into the facility and placed at a vulnerable location, such as next to a column.

Uncontrolled Parking – Public parking or a parking area located within the footprint of the building under consideration, where no operational security countermeasures are in place to screen vehicles that could enter this area with an explosive device.

2.2 Frangible/Non-frangible Façade

Façade systems that constitute at least 25% of the wall area per structural bay shall be evaluated for flexural capacity. The façade system that has the largest capacity shall be used to specify the type of façade system (i.e., *frangible* or *non-frangible*). Any façade system that occupies less than 25% of the wall area per structural bay shall be disregarded for this consideration.

A *non-frangible façade* system is quantified by having a static flexural capacity equal to 1.0 psi or greater, based on a uniform distributed load acting inward (towards the interior of the building).

A *frangible façade* system is quantified by having a static flexural capacity that is less than 1.0 psi, based on a uniform distributed load acting inward.

The procedure (i.e., action, boundary conditions, etc.) for determining the flexural capacity of the façade system should correspond with the construction details of the actual façade system. Unfactored, ultimate strengths should be used in the determination of the capacity.

Examples of this process are shown in Appendix C.

2.3 Alternate Analysis Techniques

Nonlinear Procedure

A Nonlinear Procedure implies the use of either static or dynamic finite element analysis methods that capture both material and geometric nonlinearity. It is generally a more sophisticated analysis approach than are Linear Procedures in characterizing the performance of a structure. When such procedures are used, less restrictive acceptance criteria are permitted, recognizing the improved results that can be obtained from such procedures. Caution, however, must be exercised when using Nonlinear Procedures because of potential numerical convergence problems that may be encountered during the execution of the analysis, sensitivities to assumptions for boundary conditions, geometry and material models, as well as other possible complications due to the size of the structure. Accordingly, it is imperative that only experienced structural engineering analysts with advanced structural engineering knowledge be allowed to implement these sophisticated analysis tools and judgment must be used in interpreting the results. The qualifications and experience of those proposed to perform the Nonlinear Procedure shall be reviewed and approved by the project manager, prior to starting the work.

Empirically determined damage criteria must be utilized to predict the potential collapse of a structural element. One such set of damage criteria that may be utilized in conjunction with a nonlinear analysis approach is outlined in an interim Department of Defense Construction Standard (Department of Defense, Interim Antiterrorism/Force Protection Construction Standards, Guidance on Structural Requirements (Draft), March 2001) and is included in Table 2.1. Table 2.1 provides the maximum allowable ductility and/or rotation limits for many structural component and construction types to limit the possibility of collapse. The values listed are for typical elements in conventional construction (i.e., construction that has not been hardened to resist abnormal loading). At the time of this writing the Department of Defense was considering modifications to their guidance. Unless explicitly accepted by the GSA, the guidance and criteria in the March 2001 document, as included in Table 2.1, shall be used.

Because of the inherent challenges, complexities and costs involved, Nonlinear Procedures have been used less frequently for progressive collapse analyses than have Linear Procedures. In addition, infrequent usage of Nonlinear Procedures was, until only recently, reinforced by limitations in computer hardware and analysis software. However, advancements in computer hardware and general-purpose analysis software packages over the past few years have now made it possible to employ sophisticated structural assessment techniques on large and complex structures, including dynamic time history nonlinear response of high-rise structures containing thousands of members and connections covering a wide range of inelastic constitutive relations for the purpose of practical design applications. Structural engineers, with proper experience and knowledge in structural dynamics, can now construct a global model of the whole structure to capture both material and geometric non-linearity, and to perform the required dynamic time-history non-linear analyses of the entire structure.

Table 2.1. Acceptance criteria for nonlinear analysis[1].

COMPONENT	DUCTILITY (μ)[3]	ROTATION Degrees (θ)[4]	ROTATION %Radians (θ)[4]	NOTES
Reinforced Concrete (R/C) Beam[5]		6	10.5	
R/C One-way Slabs w/o tension membrane[5]		6	10.5	
R/C One-way Slabs w/ tension membrane[5]		12	21	
R/C Two-way slabs w/o tension membrane[5]		6	10.5	
R/C Two-way Slabs w/ tension membrane[5]		12	21	
R/C Columns (tension controls) [5]		6	10.5	
R/C Columns (compression controls)	1			
R/C Frames		2	3.5	H/25 Max sidesway
Prestressed Beams	2			
Steel Beams	20	12	21	
Metal Stud Walls	7			
Open Web Steel Joist (based on flexural tensile stress in bottom chord)	6			
Metal Deck	20	12	21	
Steel Columns (tension controls)	20	12	21	
Steel Columns (compression controls)	1			
Steel Frames		2	3.5	H/25 Max sidesway
Steel Frame Connections; Fully Restrained • Welded Beam Flange or Coverplated (all types) • Reduced Beam Section		1.5 2	2.5 3.5	See Appendix D
Steel Frame Connections; Proprietary[6]		2 to 2.5	3.5 to 4.5	See Appendix D
Steel Frame Connections; Partially Restrained • Limit State governed by rivet shear or flexural yielding of plate, angle or T-section • Limit State governed by high strength bolt shear, tension failure of rivet or bolt, or tension failure of plate, angle or T-section		1.5 1	2.5 1.5	See Appendix D
One-way Unreinforced Masonry (unarched)	1			
One-way Unreinforced Masonry (compression membrane)	1			
Two-way Unreinforced Masonry (compression membrane)	1			
One-way reinforced Masonry		2	3.5	
Two-way Reinforced Masonry		2	3.5	
Masonry Pilasters (tension controls)		2	3.5	
Masonry Pilasters (compression controls)	1			
Wood Stud Walls	2			
Wood Trusses or Joist	2			
Wood Beams	2			
Wood Exterior Columns (bending)	2			
Wood Interior Columns (buckling)	1			

* Notes provided on following page.

1. COTR approval must be obtained for the use of updated tables.
2. Ductility is defined as the ratio of ultimate deflection to elastic deflection (Xu/Xe).
3. Rotation for members or frames can be determined using Figures 2.2 and 2.3 provided below.
4. Concrete having more than 2-degrees rotation must include shear stirrups per requirements of DAHSCWE Manual (See Reference 3, Page 6-1).
5. Proprietary connections must have documented test results justifying the use of higher rotational limits.

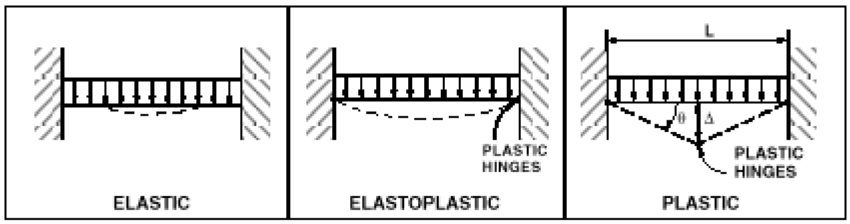


Figure 2.2. Measurement of θ after formation of plastic hinges.

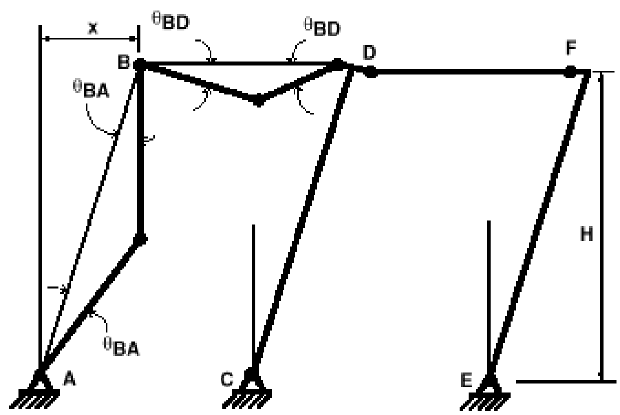


Figure 2.3. Sidesway and member end rotations (θ) for frames.

Additionally, it is recommended that the following downward loads be applied when assessing the potential for progressive collapse as presented in this Guideline.

Static Analysis Loading

For static analysis purposes the following vertical load shall be applied downward to the structure under investigation:

$$\text{Load} = 2(DL + 0.25LL) \tag{2.1}$$

Dynamic Analysis Loading

For dynamic analysis purposes the following vertical load shall be applied downward to the structure under investigation:

$$\text{Load} = DL + 0.25LL \tag{2.2}$$

where,

DL = dead load
LL = live load (higher of the design live load or the code live load).

Section 3. Exemption Process

The following procedure provides a process for evaluating the potential for progressive collapse for reinforced concrete and steel framed buildings, resulting from an abnormal loading situation. If the facility is at an extremely low risk for progressive collapse or if the human occupancy is extremely low (as determined in this process), the facility may be exempt from any further consideration of progressive collapse.

This process does not preclude a building from being evaluated for progressive collapse potential by other well-established procedures based on rational methods of analysis that are approved, in writing, by the GSA on a case-by-case basis.

The facility should be evaluated for the possibility of being exempt from further consideration of progressive collapse using the included computer program (an automated version of the exemption process) or by the following manual procedure. To begin the automated version of the exemption process, click on the 'Begin Exemption Process' button (at right).

Procedure:

Step 1. Follow the steps in Flowchart 1, depicted in Figure 3.1, to determine the potential for total exemption to the remaining methodology.

Step 2. Using Table 3.1, determine the minimum defended standoff distance consistent with the construction type and required level of protection (as determined by the GSA) of the facility under consideration. If the type of construction is not listed in Table 3.1, go directly to Step 3. Otherwise, follow the steps in Flowchart 2, depicted in Figure 3.2, to determine the potential for total or partial exemption to the remaining methodology. Note that defended standoff is only considered as one factor in determining if a facility is exempt. If a facility is not exempt and an analysis is required, the analysis process is threat independent.

Step 3. This step offers a more detailed consideration of the facility if the requirements set forth in Step 2 are not achievable or the construction type is not included in Table 3.1. The user shall begin with Flowchart 3 (Figure 3.3) to determine the potential for total exemption. The user will then continue to Flowchart 4 or 5 (Figures 3.4 or 3.5, respectively) for concrete structures, or Flowchart 4 or 6 (Figures 3.4 or 3.6, respectively) for steel frame structures as indicated.

Step 4. The results determined in the exemption process shall be documented by the project engineer and submitted to the GSA Project Manager for review. This process is documented in all ***STANDGARD*** generated progressive collapse assessment reports.

In newly constructed facilities, the project manager is ultimately responsible for verifying that the site and structural characteristics used in this procedure are consistent from conceptual through 100% plans (including architectural, structural and site drawings). Should the project characteristics change or if the GSA Project Manager disagrees with the assessment of the characteristics used in the exemption process, further progressive collapse consideration may be required.

For existing facilities, the GSA Project Manager shall review the results of this procedure documented by the project engineer. If the GSA Project Manager disagrees with the assessment of characteristics used in the procedure, further progressive collapse consideration may be required.

It should be noted that limited test data currently exists for steel frame beam-to-column connections subjected to the type of loading conditions that accompany removal of a column. As a result, the exemption process criteria have been designed to be conservative and therefore, there will be very few exemptions for steel frame structures.

Table 3.1. Minimum defended standoff distances for various types of construction.

Construction Type	Minimum Defended Standoff Distance (ft)*		
	ISC Required Level of Protection		
	Low and Medium/low	Medium	Higher
Reinforced Concrete Construction			
Rigid frame structure with a ***non-frangible facade*** (FEMA 310 Building Type: C1, C2)	25	40	130
Rigid frame structure with a ***frangible facade*** (FEMA 310 Building Type: C1, C3, RM2)	25	35	100
Flat slab structure with a ***non-frangible facade*** (FEMA 310 Building Type: C2)	25	40	130
Flat slab structure with a ***frangible facade*** (FEMA 310 Building Type: C3, RM2)	25	35	100
Shear wall structure (FEMA 310 Building Type: C2)	25	35	100
Steel Construction			
Rigid frame structure with a ***non-frangible facade*** (FEMA 310 Building Type: S4)	25	40	130
Rigid frame structure with a ***frangible facade*** (FEMA 310 Building Type: S1, S5, RM2)	25	35	100
Lightweight steel framed structures (i.e., Butler style buildings, etc.) (FEMA 310 Building Type: S1A, S2, S2A, S3, S5A)	55	105	165
Masonry Construction			
Reinforced masonry wall with steel or r/c concrete frame (FEMA 310 Building Type: C3, RM2)	25	35	100
Masonry bearing walls with reinforced CMU pilasters (FEMA 310 Building Type: RM1, URM, URMA)	65	105	290
Precast Construction			
Precast concrete frame (FEMA 310 Building Type: PC1, PC1A, PC2, PC2A)	55	105	165
Wood Construction			
Wooden frame (FEMA 310 Building Type: C2A, C3A, W1, W1A, W2)	95	120	360

[*] These distances are used in the progressive collapse exemption process only and are not directly related to general standoff distances cited in the ISC Security Criteria.

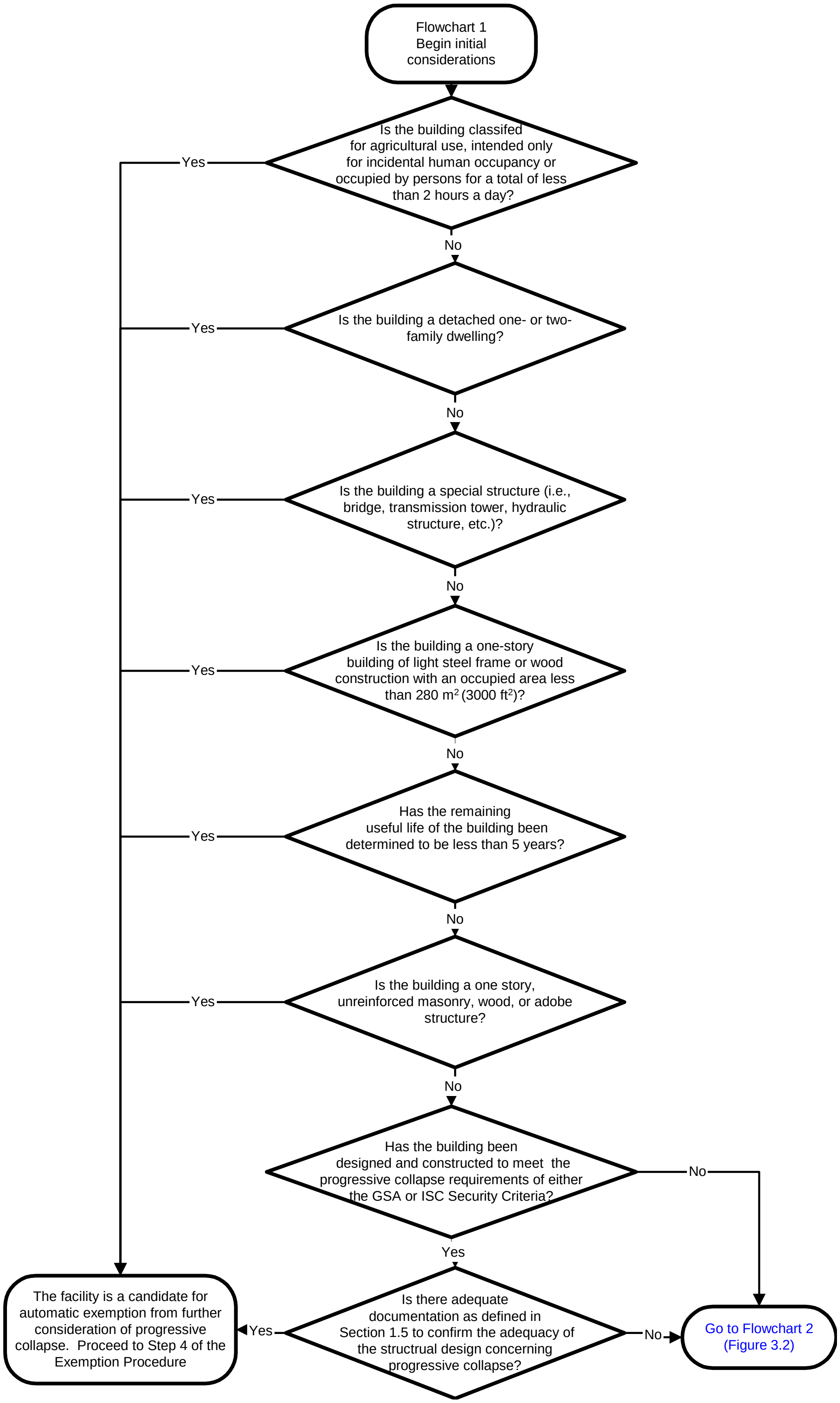
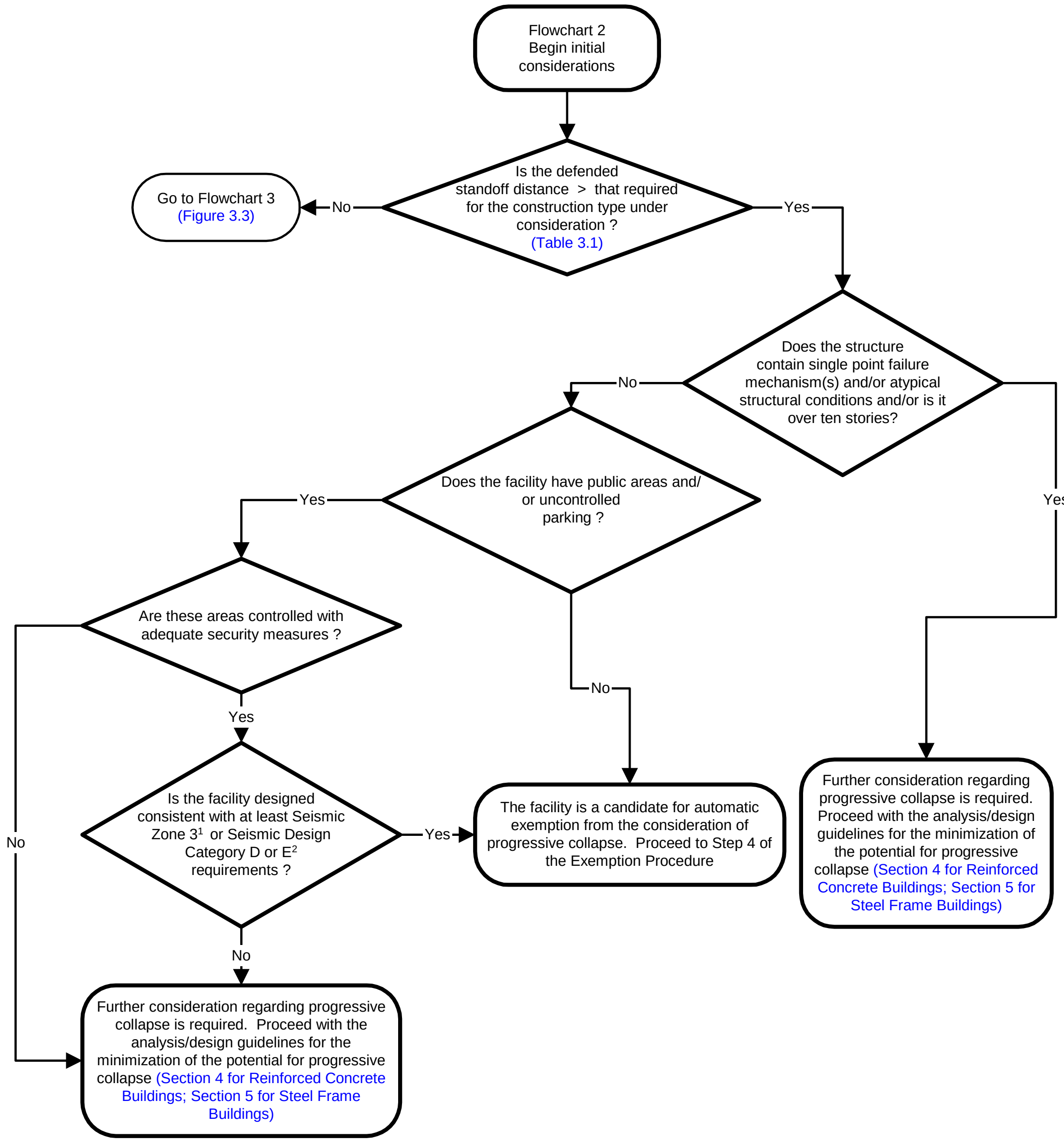
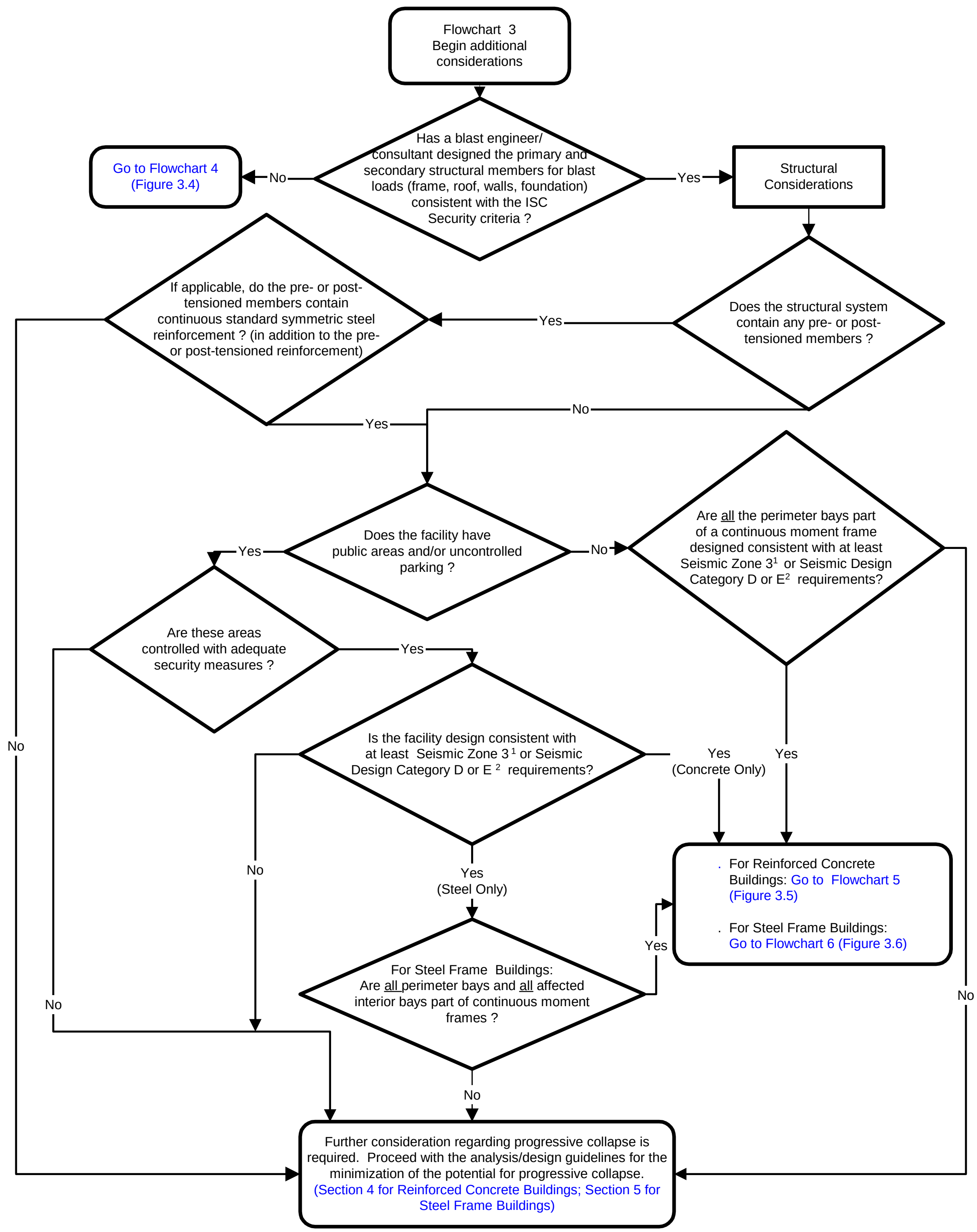


Figure 3.1. Flowchart 1. To be used with Step 1 of the exemption process.



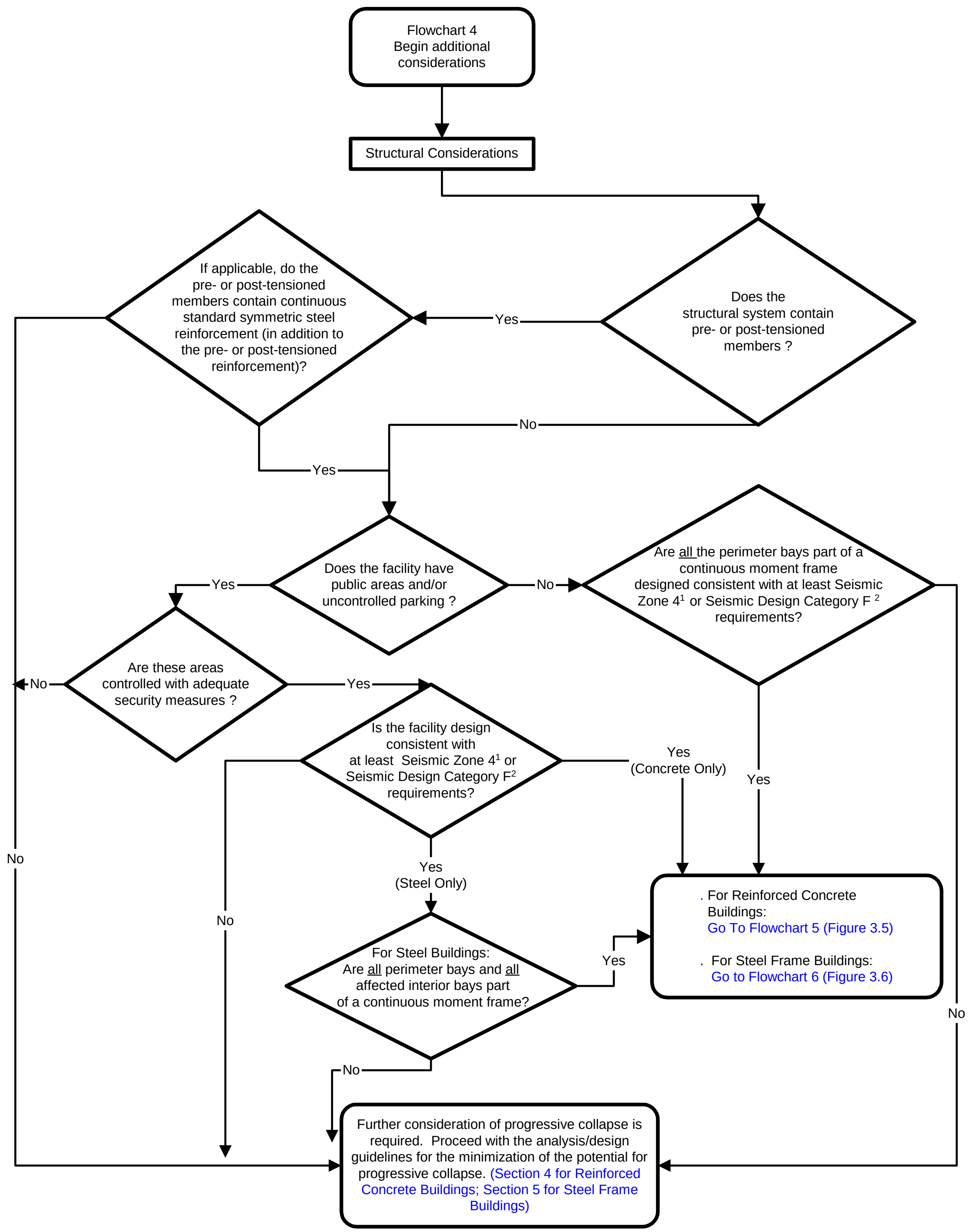
1 As defined in the 1997 Uniform Building Code
2 As defined in the 2000 International Building Code

Figure 3.2. Flowchart 2. To be used with Step 2 of the exemption process.



1 As defined in the 1997 Uniform Building Code
2 As defined in the 2000 International Building Code

Figure 3.3. Flowchart 3. To be used with Step 3 of the exemption process.



1 As defined in the 1997 Uniform Building Code
2 As defined in the 2000 International Building Code

Figure 3.4. Flowchart 4. To be used with Step 3 of the exemption process.

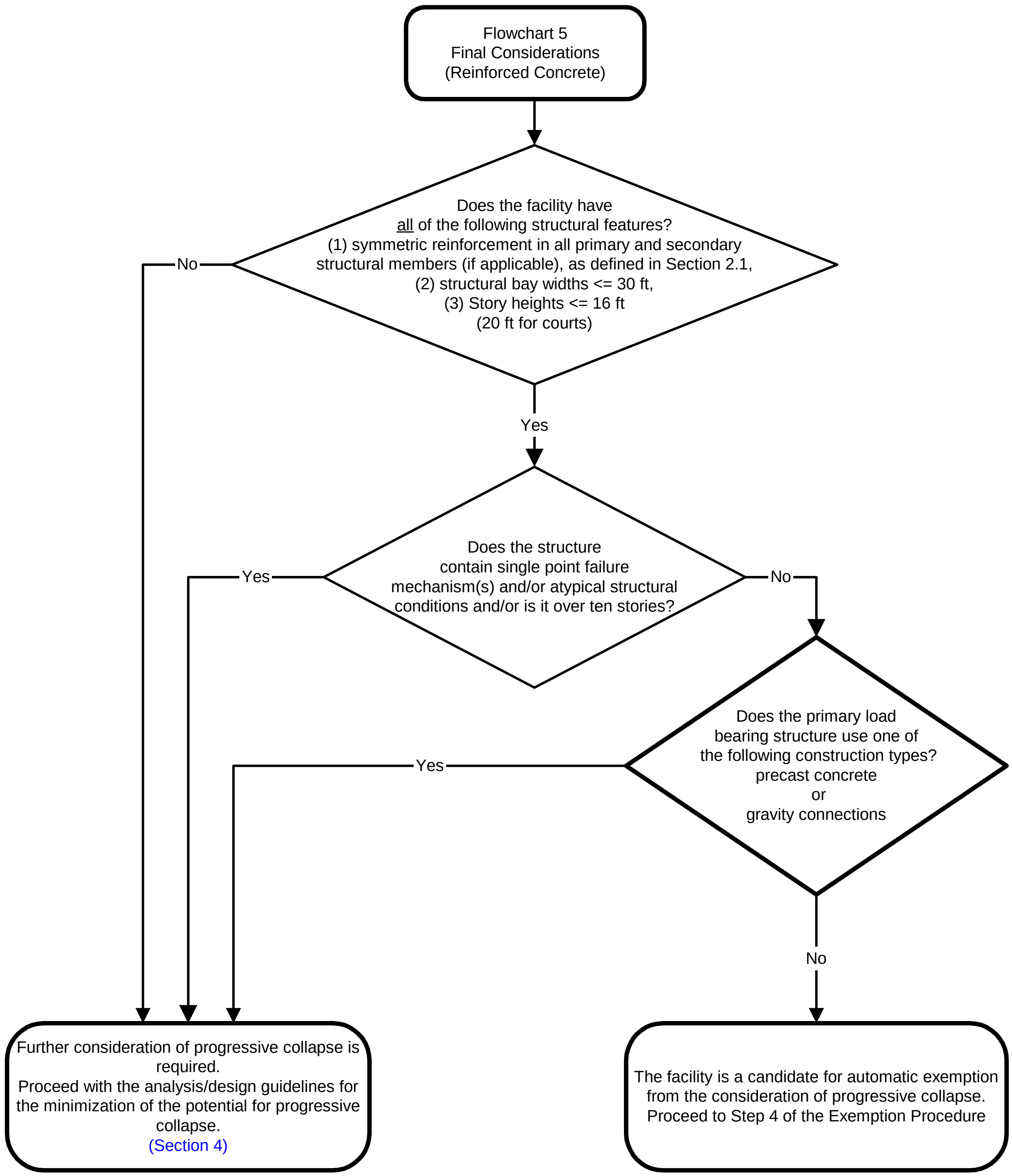


Figure 3.5. Flowchart 5. To be used with Step 3 of the exemption process.

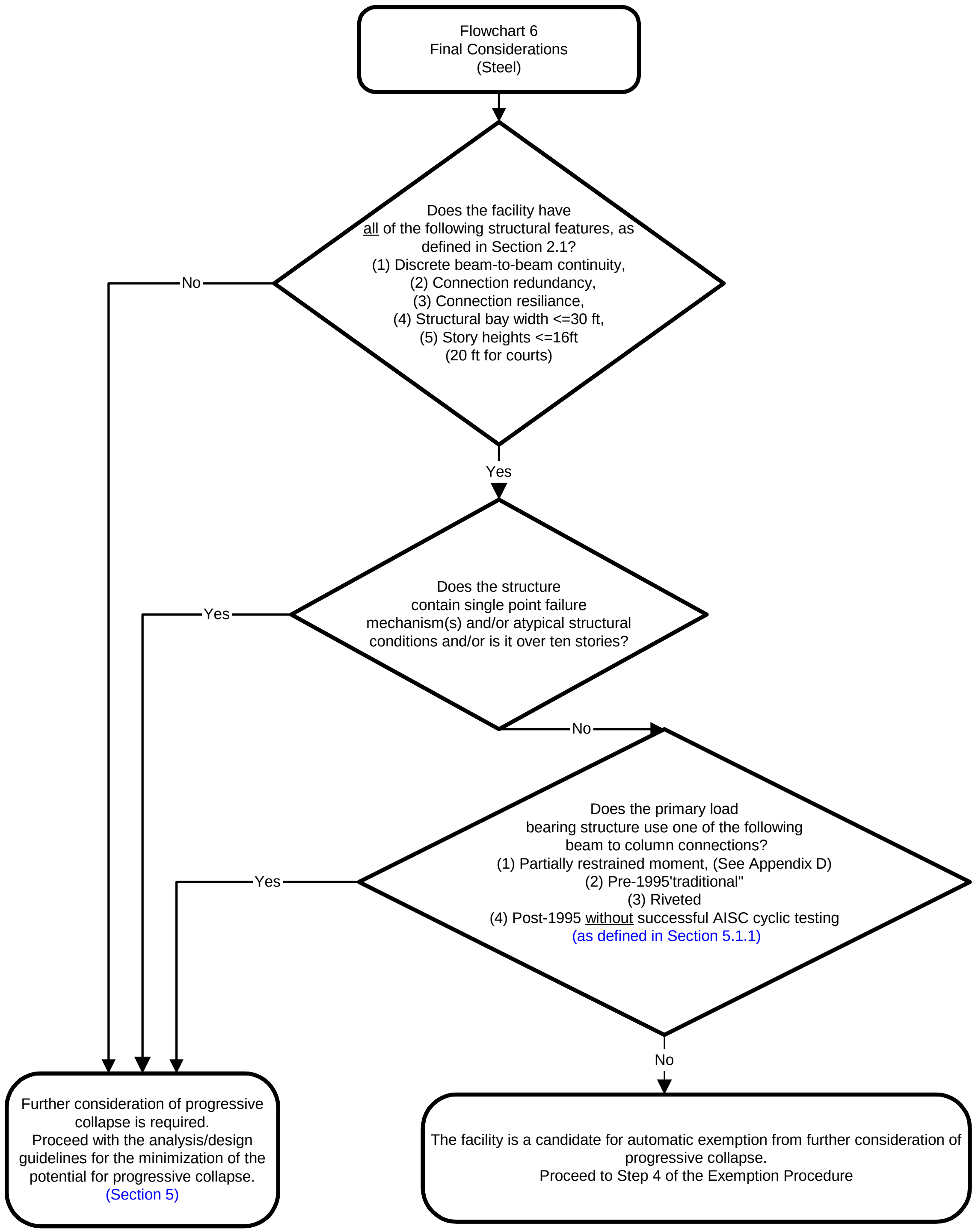


Figure 3.6. Flowchart 6. To be used with Step 3 of the exemption process.

Section 4. Reinforced Concrete Building Analysis and Design

4.1 New Construction

All newly constructed facilities shall be designed with the intent of reducing the potential for progressive collapse as a result of an abnormal loading event, regardless of the required level of protection. The process presented in these Guidelines consists of an analysis/redesign approach. This method is intended to enhance the probability that if localized damage occurs as the result of an abnormal loading event, the structure will not progressively collapse or be damaged to an extent disproportionate to the original cause of the damage. The flowchart, shown in Figure 4.1, outlines this process for reducing the potential for progressive collapse in newly constructed facilities.

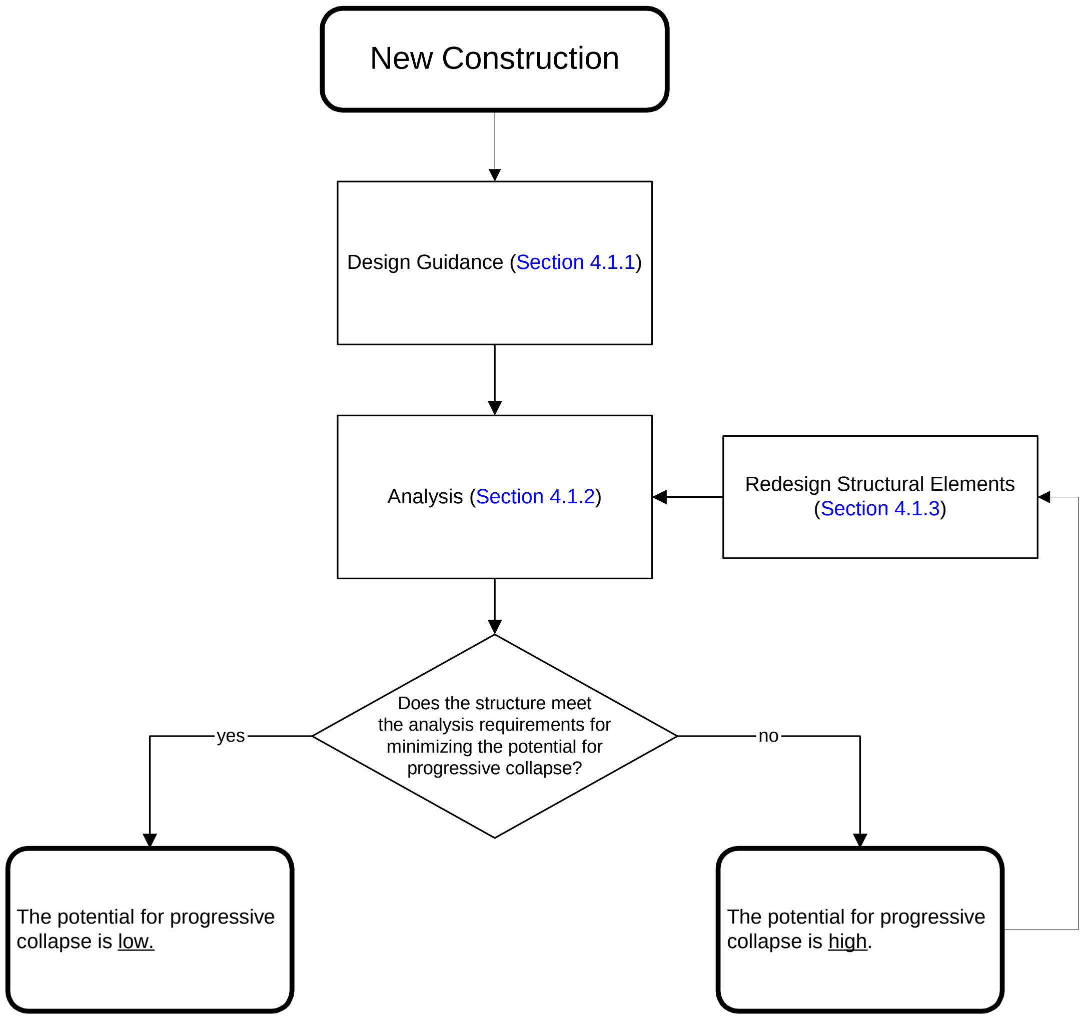


Figure 4.1. Process for reducing the potential for progressive collapse in new construction.

4.1.1 Design Guidance

Structural design guidance, although not a requirement of these Guidelines, is provided for consideration during the initial structural design phase and prior to performing the progressive collapse analysis outlined in Section 4.1.2 to minimize the impact on the building's final design. These Guidelines should act as a supplement to the Interagency Security Committee (ISC) Security Design Criteria for New Federal Office Buildings and Major Modernization Projects, which states that mitigation of progressive collapse be addressed in the design of new structures.

It is recommended that the following structural characteristics be considered in the initial phases of structural design. The incorporation of these features will provide for a much more robust structure and increase the probability of achieving a low potential for progressive collapse when performing the analysis procedure in Section 4.1.2.

Redundancy - The use of redundant lateral and vertical force resisting systems are highly encouraged when considering progressive collapse. Redundancy tends to promote an overall more robust structure and helps to ensure that alternate load paths are available in the case of a structural element(s) failure. Additionally, redundancy generally provides multiple locations for yielding to occur, which increases the probability that damage may be constrained.

The use of detailing to provide structural continuity and ductility - It is critical that the primary structural elements (i.e., girders and beams) be capable of spanning two full spans (i.e., two full bays). This requires both beam-to-beam structural continuity across the removed column, as well as the ability of both primary and secondary elements to deform flexurally well beyond the elastic limit without experiencing structural collapse. Hence, correct detailing of connections shall be required in the design to ensure discrete *beam-to-beam continuity* across a column, and to ensure *connection redundancy and resilience.* For concrete structures, configuring connection reinforcing steel in structural elements (i.e., girder, beams and columns) such that the concrete material can behave in a ductile manner is critical. Having the capability of achieving a ductile response is imperative when considering an extreme redistribution of loading such as that encountered for the case of a structural element(s) failure.

Capacity for resisting load reversals - It is recommended that both the primary and secondary structural elements be designed such that these components are capable of resisting load reversals for the case of a structural element(s) failure.

An example illustrating the importance of having the capability to resist load reversals follows. Consider a reinforced concrete building designed for gravity loads only (i.e., dead and live loads). It is possible that many of the structural members will not be able to resist load reversals. While the columns may contain reinforcement in all faces and be capable of exhibiting substantial capacity in all directions, the horizontal structural components (i.e., beams, slabs, etc.) may only contain reinforcement needed for resisting the downward loading caused by gravity.

Consider the structural configuration shown in Figure 4.2. Along the length of the beam, negative reinforcing steel (top steel) is provided in areas where negative moments are induced by the downward loading. Likewise, positive reinforcing steel (bottom steel) is provided in areas where positive moments are induced by the downward loading. ACI 318 includes a provision for structural integrity reinforcement that requires some top and bottom reinforcement to be continuous for beams such as those shown in Figure 4.2 The amount of reinforcement that ACI 318 requires to be continuous may not be sufficient to prevent progressive collapse for instantaneous removal of a column.

It is not likely that the structural configuration illustrated in Figure 4.2 will be capable of effectively redistributing loads when a primary support column is removed, as shown in Figure 4.3. Not only does the unsupported span length double, but the loss in support induces forces into the beam that were not considered in the original design. Specifically, the region of the beam designed for resisting negative moment forces is suddenly subjected to a positive moment and a substantial increase in vertical load. Due to the reinforcement configuration, the beam has very little resistance regarding the redistributed loading and will likely fail in a non-ductile manner, which could potentially lead to a propagation of additional structural failures.

Capacity for resisting shear failure - It is essential that the primary structural elements maintain sufficient strength and ductility under an abnormal loading event to preclude a shear failure such as in the case of a structural element(s) failure. When the shear capacity is reached before the flexural capacity, the possibility of a sudden, non-ductile failure of the element exists which could potentially lead to a progressive collapse of the structure.

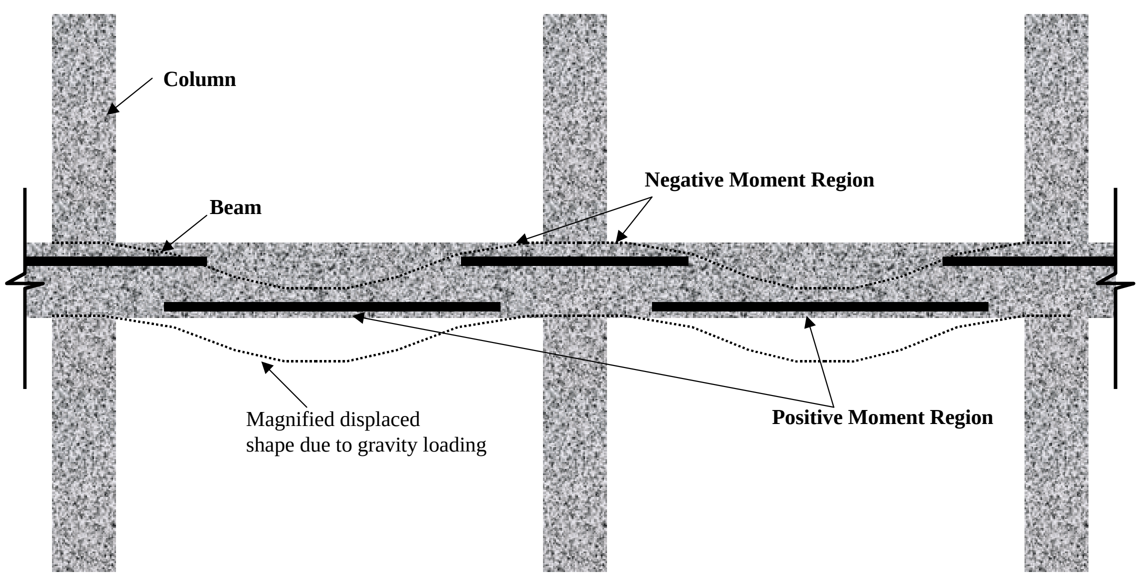


Figure 4.2. A sketch depicting the reinforcement scheme for a beam designed for gravity loads only.

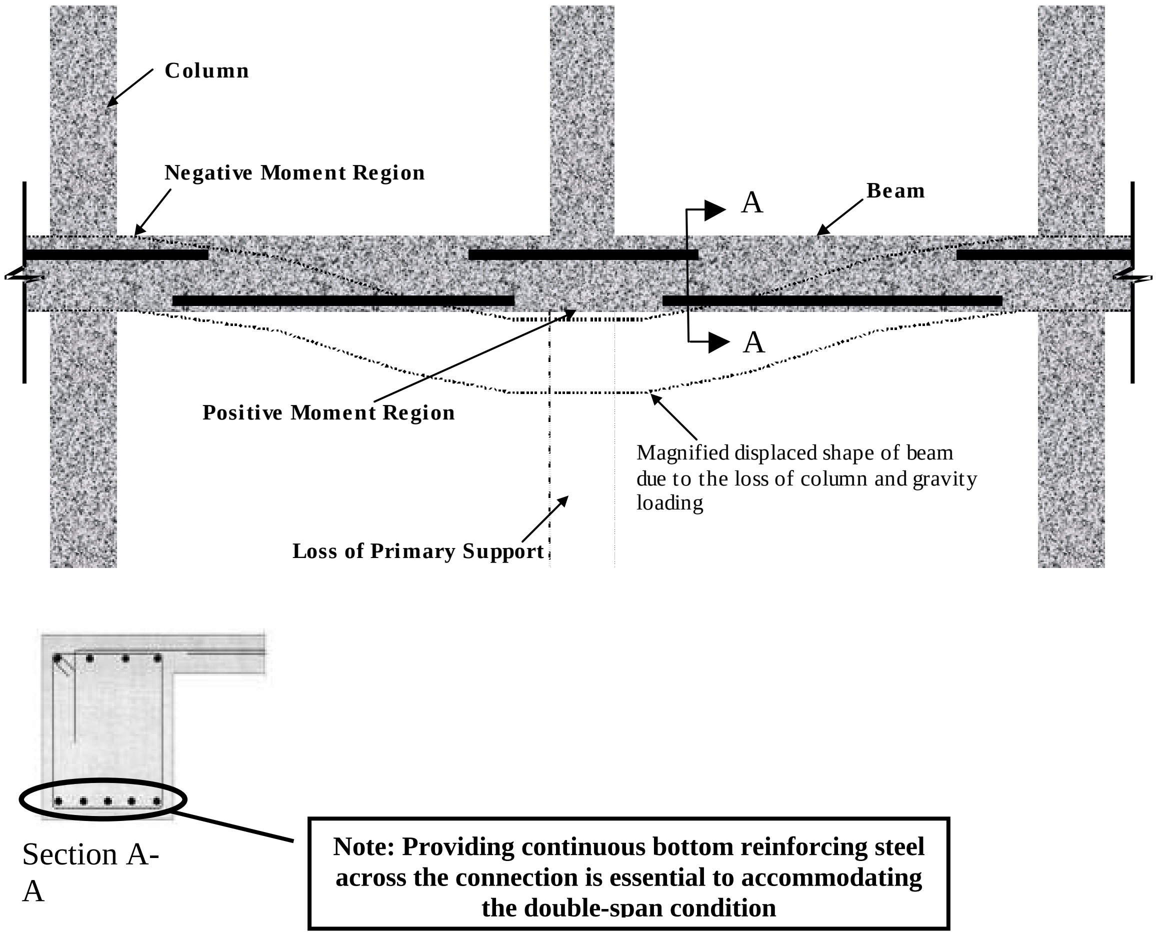


Figure 4.3. Response of the beam shown in Figure 4.2 after the loss of primary column support, shows the inability to protect against progressive collapse.

Note:

A set of design procedures for preliminarily sizing structural components is included in Appendix B. These procedures are not required by these Guidelines. However, these procedures can be used to preliminarily size and detail elements prior to performing the progressive collapse analysis presented in Section 4.1.2, if desired by the project engineer.

4.1.2 Analysis

The following static linear elastic analysis approach may be used to assess the potential for progressive collapse in all newly constructed facilities. Other analysis approaches may also be used, such as those discussed in Section 2.3, but the analysis considerations (Section 4.1.2.3) and allowable extents of collapse (Section 4.1.2.4), must be used in the assessment of the potential for progressive collapse.

The following procedure uses a static linear elastic, approach coupled with the following:

- Criteria for assessing the analysis results
- A suite of analysis cases
- Specific loading criteria to be used in the analysis

4.1.2.1 Analysis Techniques

The following analysis procedure shall be performed using well-established linear elastic, static analysis techniques. It is recommended that 3-dimensional analytic models be used to account for potential 3-dimensional effects and avoid overly conservative solutions. Nevertheless, 2-dimensional models may be used provided that the general response and 3-dimensional effects can be adequately accounted for.

4.1.2.2 Procedure

The potential for progressive collapse can be determined by the following procedure.

Step 1. The components and connections of both the primary and secondary structural elements shall be analyzed for the case of an instantaneous loss in primary vertical support. The applied downward loading shall be consistent with that presented in Section 4.1.2.3.

Step 2. The results from the analyses performed in Step 1 shall be evaluated by utilizing the analysis criteria defined in Section 4.1.2.4.

Note:

If the analysis results show that the structural member(s) and/or connections/joints <u>are not in compliance</u> with the analysis criteria presented in Section 4.1.2.4 (i.e., the member and/or connection capacities are greatly exceeded and it is unlikely that the structure is capable of effectively redistributing loads), <u>the facility exhibits a high potential for progressive collapse</u> and the user shall redesign the members and/or connections/joints consistent with the procedure outlined in Section 4.1.3. However, if the analysis results show that the structural member(s) and/or connections/joints <u>are in compliance</u> with the analysis criteria presented in Section 4.1.2.4, <u>the facility exhibits a low potential for progressive collapse</u> and requires no further progressive collapse considerations.

4.1.2.3 Analysis Considerations and Loading Criteria

The following analysis considerations shall be used in the assessment for progressive collapse for typical structural configurations. Atypical structural configurations are addressed in Section 4.1.2.3.2.

4.1.2.3.1 Typical Structural Configurations.

Facilities that have a relatively simple layout with <u>no atypical structural configurations</u> shall use the following analysis scenarios:

<u>Framed or Flat Plate Structures</u>

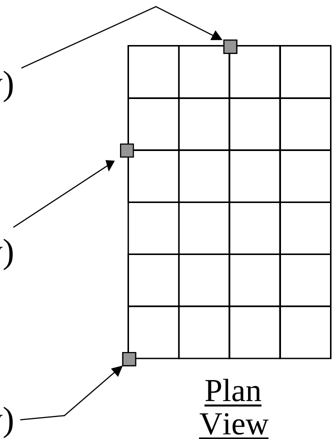
Exterior Considerations

The following exterior analysis cases shall be considered in the procedure outlined in Section 4.1.2.2.

1 Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at or near the middle of the short side of the building.

2 Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at or near the middle of the long side of the building.

3 Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at the corner of the building.

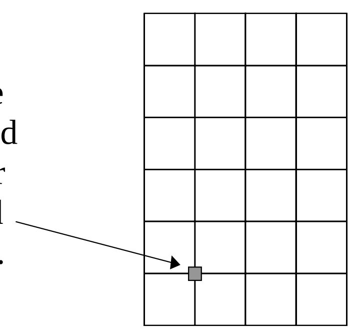


<u>Plan View</u>

Interior Considerations

Facilities that have underground parking and/or uncontrolled public ground floor areas shall use the following interior analysis case(s) in the procedure outlined in Section 4.1.2.2.

1 Analyze for the instantaneous loss of 1 column that extends from the floor of the underground parking area or uncontrolled public ground floor area to the next floor (1 story). The column considered should be interior to the perimeter column lines.



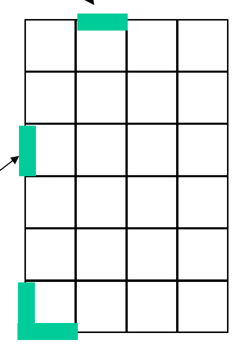
<u>Plan View</u>

Shear/Load Bearing Wall Structures

Exterior Considerations

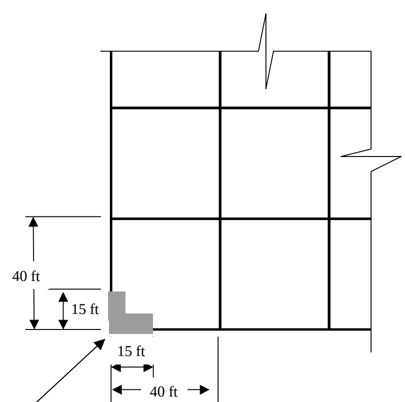
The following exterior analysis cases shall be considered in the procedure outlined in Section 4.1.2.2.

1. Analyze for the instantaneous loss of one structural bay or 30 linear feet of an exterior wall section (whichever is less) for one floor above grade, located at or near the middle of the short side of the building.
2. Analyze for the instantaneous loss of one structural bay or 30 linear feet of an exterior wall section (whichever is less) for one floor above grade, located at or near the middle of the long side of the building.
3. Analyze for the instantaneous loss of the entire bearing wall along the perimeter at the corner structural bay or for the loss of 30 linear feet of the wall (15 ft in each major direction) (whichever is less) for one floor above grade*.



Plan View

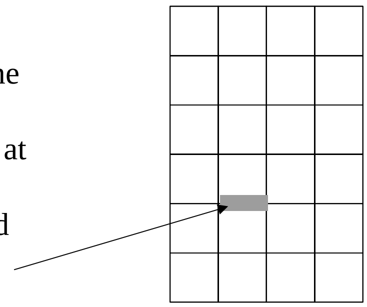
* The loss wall section for the corner consideration must be continuous and include the corner. For example, if the structural bay of a facility is 40 ft by 40 ft, the wall section that would require removal consists of 30 ft of the wall beginning at the corner and extending 15 ft in each major direction.



Interior Considerations

Facilities that have underground parking and/or uncontrolled public ground floor areas shall use the following interior analysis cases in the procedure outlined in Section 4.1.2.2.

1 Analyze for the instantaneous loss of one structural bay or 30 linear feet of an interior wall section (whichever is less) at the floor level of the underground parking area and/or uncontrolled ground floor area. The wall section considered should be interior to the perimeter bearing wall line.



Plan View

Analysis Loading

For static analysis purposes the following vertical load shall be applied downward to the structure under investigation:

$$\text{Load} = 2(\text{DL} + 0.25\text{LL}) \tag{4.1}$$

where,

DL = dead load
LL = live load

Note:

Depending on the facility characteristics and/or the outcome of the exemption process, the user may only be required to perform one of the analysis cases. For example, if the facility does not contain any uncontrolled parking areas and/or public areas, the user will not be required to perform the analyses for the interior considerations. Additional analysis cases should be considered, however, if there are significant changes in column or other load bearing member strength or configuration along any portion of the facility.

4.1.2.3.2 Atypical Structural Configurations

All structures are generally unique and are often not *typical* (i.e., buildings often contain distinguishing structural features or details), hence, developing a set of analysis considerations that applies to every facility is impractical. Thus, the user of these Guidelines must use engineering judgment to determine critical analysis scenarios that should be assessed, in addition to the situations presented in Section 4.1.2.3.1. The intent of these provisions should be reflected in these analysis scenarios. Specifically, the scenarios should consider cases where loss of a vertical support (column or wall) could lead to disproportionate damage. Possible structural configurations that may result in an atypical structural arrangement include, but are not limited to, the following configurations:

- Combination Structures
- Vertical Discontinuities/Transfer Girders
- Variations in Bay Size/Extreme Bay Sizes
- Plan Irregularities
- Closely Spaced Columns

These atypical structural configurations are described in more detail in. Appendix A

4.1.2.4 Analysis Criteria

Structural collapse resulting from the instantaneous removal of a primary vertical support shall be limited. Typically, the allowable collapse area for a building will be based on the structural bay size. However, to account for structural configurations that have abnormally large structural bay sizes, the collapsed region will also be limited to a reasonably sized area. The allowable extent of collapse for the instantaneous removal of a primary vertical support member along the exterior and within the interior of a building is defined as follows.

Exterior Considerations

The maximum allowable extents of collapse resulting from the instantaneous removal of an exterior primary vertical support member one floor above grade shall be confined to:

1. the structural bays directly associated with the instantaneously removed vertical member in the floor level directly above the instantaneously removed vertical member.

or

2. 1,800 ft^2 at the floor level directly above the instantaneously removed vertical member

whichever is the smaller area (Figure 4.4.a).

Interior Considerations

The allowable extents of collapse resulting from the instantaneous removal of an interior primary vertical support member in an uncontrolled ground floor area and/or an underground parking area for one floor level shall be confined to:

1. the structural bays directly associated with the instantaneously removed vertical member

or

2. 3,600 ft^2 at the floor level directly above the instantaneously removed vertical member

whichever is the smaller area (Figure 4.4.b). If there is no uncontrolled ground floor area and/or an underground parking area present in the facility under evaluation, the internal consideration is not required.

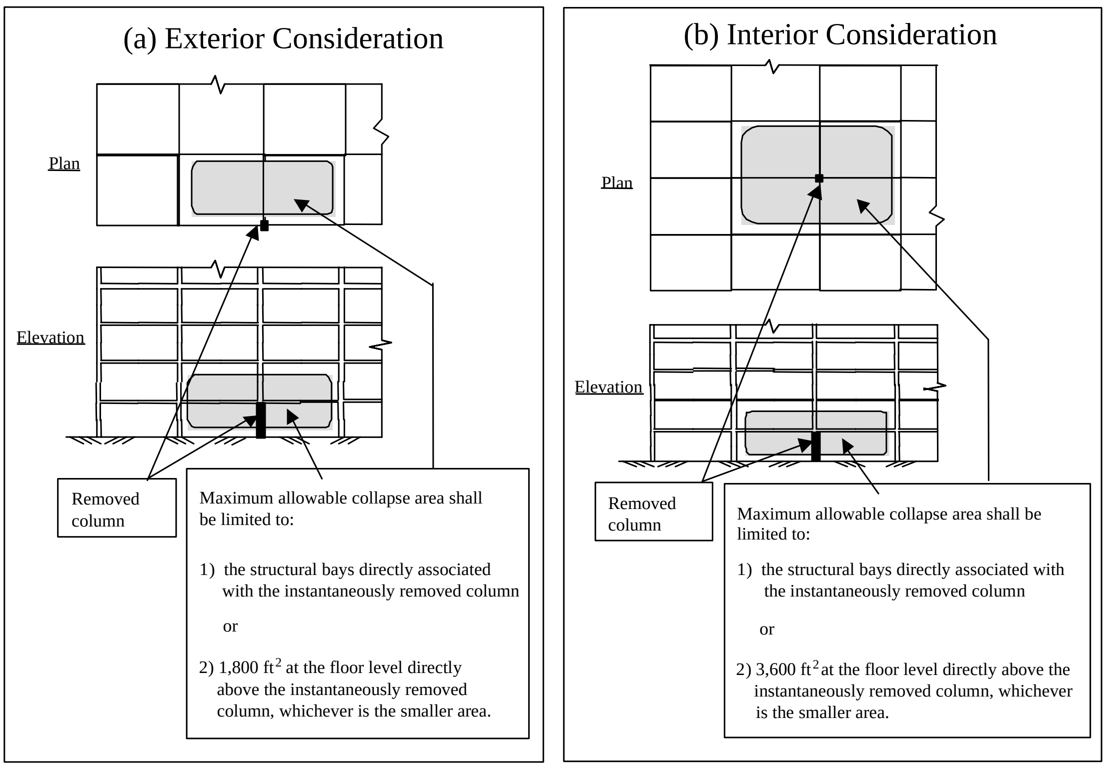


Figure 4.4. An example of maximum allowable collapse areas for a structure that uses columns for the primary vertical support system.

Acceptance Criteria

An examination of the linear elastic analysis results shall be performed to identify the magnitudes and distribution of potential demands on both the primary and secondary structural elements for quantifying potential collapse areas. The magnitude and distribution of these demands will be indicated by **D**emand-**C**apacity **R**atios (*DCR*). These values and approaches are based, in part, on the methodology presented in the following references:

- NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings (FEMA 274). Issued by Federal Emergency Management Agency, October 1997.
- Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356). Issued by Federal Emergency Management Agency, November 2000.
- Interim Antiterrorism/Force Protection Construction Standards, Guidance on Structural Requirements (Draft). Issued by Department of Defense, March 2001.
- Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects. U.S. General Services Administration and Applied Research Associates, Inc. November 2000.

Acceptance criteria for the primary and secondary structural components shall be determined as:

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{4.2}$$

where,

Q_{UD} = Acting force (demand) determined in component or connection/joint (moment, axial force, shear, and possible combined forces)

Q_{CE} = Expected ultimate, un-factored capacity of the component and/or connection/joint (moment, axial force, shear and possible combined forces)

Using the *DCR* criteria of the linear elastic approach, structural elements and connections that have *DCR* values that exceed the following allowable values are considered to be severely damaged or collapsed.

The allowable *DCR* values for primary and secondary structural elements are:

- $DCR \leq 2.0$ for typical structural configurations (Section 4.1.2.3.1)
- $DCR \leq 1.5$ for atypical structural configurations (Section 4.1.2.3.2)

Note:

The criteria for atypical structural configurations (i.e., DCR ≤ 1.5) may be limited to the 'atypical' region if this is localized. For example, consider a structure that uses transfer girders along one face of the perimeter and a typical structural configuration for the remainder of the structure. The perimeter structural bays along the side of the building that utilizes transfer girders shall use a DCR that is less than or equal to 1.5, *but the remainder of the building shall use a DCR that is less than or equal to 2.0 for the assessment of the potential for progressive collapse.*

The approach used in estimating the magnitude and distribution of the potential inelastic demands and displacements used in these Guidelines is similar to the 'm-factor' approaches currently employed in FEMA 273 and 356 for linear elastic analysis methods.

The step-by-step procedure for conducting the linear elastic, static analysis follows.

Step 1. Remove a vertical support from the location being considered and conduct a linear-static analysis of the structure as indicated in Section 4.1.2.2. Load the model with 2(DL + 0.25LL).

Step 2. Determine which members and connections have DCR values that exceed the acceptance criteria. <u>If the DCR for any member end connection is exceeded based upon shear force, the member is to be considered a failed member. In addition, if the flexural DCR values for both ends of a member or its connections, as well as the span itself, are exceeded (creating a three hinged failure mechanism – Figure 2.2), the member is to be considered a failed member.</u> Failed members should be removed from the model, and all dead and live loads associated with failed members should be redistributed to other members in adjacent bays.

Step 3. For a member or connection whose Q_{UD}/Q_{CE} ratio exceeds the applicable flexural DCR values, place a hinge at the member end or connection to release the moment. This hinge should be located at the center of flexural yielding for the member or connection. Use rigid offsets and/or stub members from the connecting member as needed to model the hinge in the proper location. For yielding at the end of a member the center of flexural yielding should not be taken to be more than ½ the depth of the member from the face of the intersecting member, which is usually a column (Figure 4.5).

Step 4. At each inserted hinge, apply equal-but-opposite moments to the stub/offset and member end to each side of the hinge. The magnitude of the moments should equal the expected flexural strength of the moment or connection, and the direction of the moments should be consistent with direction of the moments in the analysis performed in Step 1.

Step 5. Re-run the analysis and repeat Steps 1 through 4. Continue this process until no DCR values are exceeded. <u>**If moments have been re-distributed throughout the entire building and DCR values are still exceeded in areas outside of the allowable collapse region, the structure will be considered to have a high potential for progressive collapse.**</u>

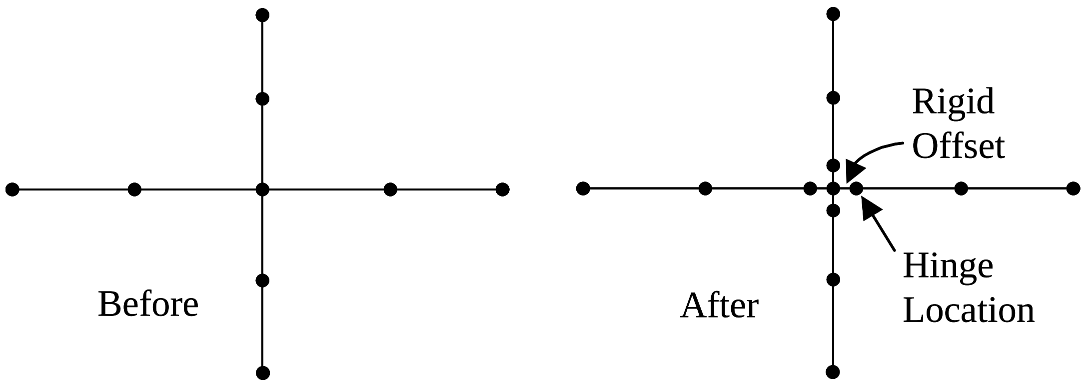


Figure 4.5. Rigid offset placement.

4.1.2.5 Material Properties

For these Guidelines the design material strengths may be increased by a strength-increase factor to determine the expected material strength (for determining capacities, etc.). These should be used only in cases where the designer or analyst is confident in the actual state of the facility's materials. These values are given in Table 4.2.

Table 4.2. Strength-increase factors for various construction materials.

Construction Material	**Strength Increase Factor**
Reinforced Concrete	
Concrete Compressive Strength	1.25
Reinforcing Steel (tensile and yield strength)	1.25
Concrete Unit Masonry	
Compressive Strength	1.0
Flexural Tensile Strength	1.0
Shear Strength	1.0
Wood and Light Metal Framing	
All Components	1.0

4.1.2.6 Modeling Guidance

General

The analytic model(s) used in assessing the potential for progressive collapse should be modeled as accurately as possible to the anticipated or existing conditions. This includes all material properties, design details, etc. In addition, the analyst shall realistically approximate the type of boundary conditions (e.g., fixed, simple, etc.), and should be aware of any limitations or anomalies of the software package(s) being used to perform the analysis.

Vertical Element Removal

The vertical element (i.e., the column, bearing wall, etc.) that is removed should be removed instantaneously. While the speed at which an element is removed has no impact on a static analysis, the speed at which an element is removed in a dynamic analysis may have a significant impact on the response of the structure. Because of this, it is recommended for the case where a dynamic analysis is performed, the vertical supporting element should be removed over a time period that is no more than 1/10 of the period associated with the structural response mode for the vertical element removal. Also the vertical element removal shall consist of the removal of the vertical element only. This removal should not impede into the connection/joint or horizontal elements that are attached to the vertical element at the floor levels. An example sketch illustrating the correct and incorrect way to remove a column is shown in Figure 4.6. It is critical that the user understand that the sketch is not representative of damage due to any specific threat (see Section 1.3 for discussion of member removal approach).

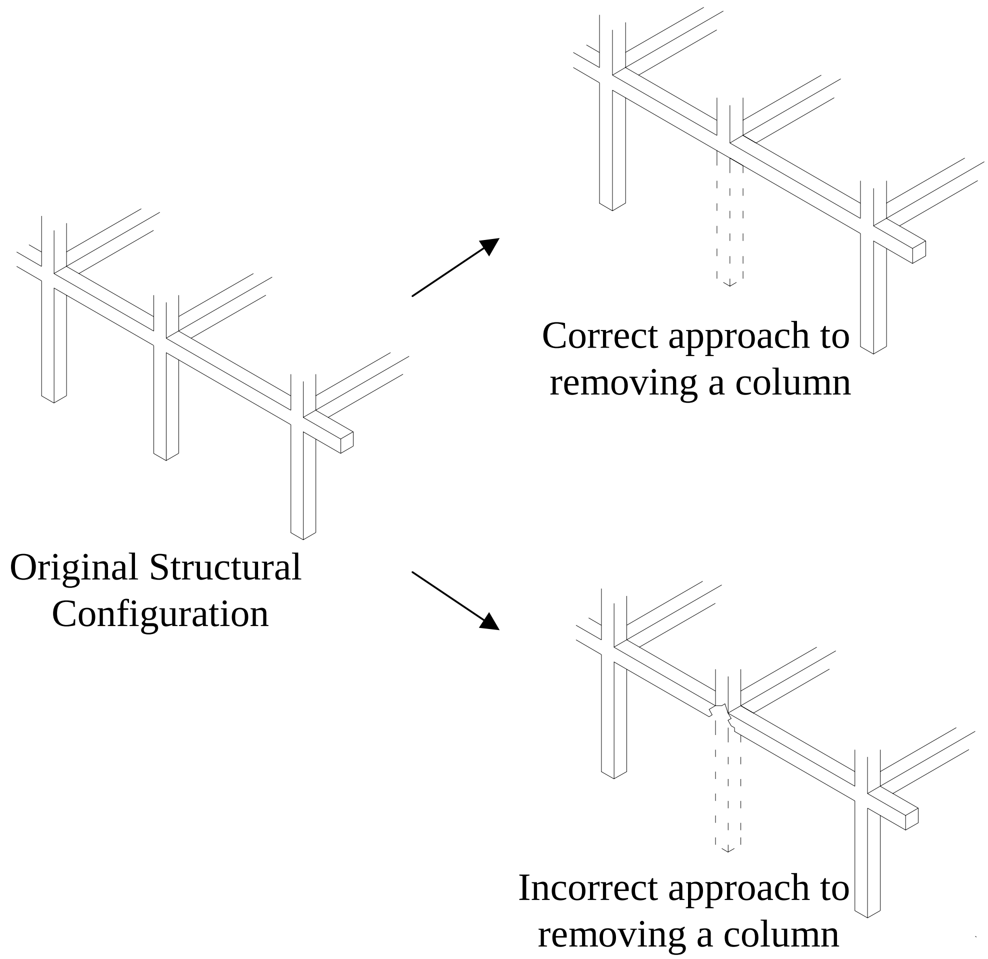


Figure 4.6. Sketch of the correct and incorrect approach for removing a column.

4.1.3 Redesign of Structural Elements

Structural configurations that are analyzed consistent with Section 4.1.2 and determined to have a high potential for progressive collapse shall be redesigned to a level that is consistent with a low potential for progressive collapse.

4.1.3.1 Procedure

The following steps shall be followed when redesigning the deficient structural elements identified in the analysis procedure (Section 4.1.2).

Step 1. As a minimum, the structural elements and/or connections identified as deficient in Section 4.1.2 should be redesigned consistent with the redistributed loading determined in this process in conjunction with the standard design requirements of the project specific building code(s) using well-established design techniques. The redesign criteria for typical and atypical structural configurations follow:

Typical Structural Configurations

Structural elements and beam-to-column connections must have a DCR value of 2.0 or less for primary and secondary structural members in the design of deficient components and connections. If an approved alternate analysis criteria is used, the deficient components should be designed to, as a minimum, achieve the allowable values associated with that criteria for the redistributed loading.

Atypical Structural Configurations

Structural elements and beam-to-column connections must have a DCR value of 1.5 or less for both primary and secondary elements in the design of deficient components and connections. If an approved alternate analysis criteria is used, the deficient components should be designed to, as a minimum, achieve the allowable values associated with that criteria for the redistributed loading.

Step 2. Upon the completion of Step 1, the redesigned structure shall be reanalyzed consistent with analysis procedure outlined in Section 4.1.2.

Note:

In order to achieve the necessary design requirements, significant structural changes may be required, such as greatly increasing member sizes, the addition of symmetric reinforcement (for reinforced concrete elements), moment resisting connections, etc. However, the design criteria for atypical structures may be limited to the 'atypical' region if this is localized. For example, consider a building that uses transfer girders along one face of the perimeter and a typical structural configuration for the remainder of the structure. The perimeter structural bays shall be designed to have a DCR value of 1.5 or less, but the remainder of the building may be designed to have maximum DCR values of 2.0 for primary and secondary structural elements.

It should be noted that to achieve a low potential for progressive collapse more than one iteration of the redesign/analysis process may be required. For example, a change in the size of structural members may alter the magnitude and distribution of the redistributed load.

The designer is not limited to a particular method for improving the original design with respect to the minimization of the potential for progressive collapse. For the example illustrated in Figure 4.7, assume the results of Section 4.1.2 indicates the spandrel beams from the 2nd floor level to the roof level for a given rigid frame structure are not adequate in regards to the analysis criteria presented in Section 4.1.2.4. The designer has the freedom to evenly distribute an improved redesign along the total height of the facility or concentrate them over a few floor levels (Figure 4.7), as long as the overall intent of minimizing the potential for progressive collapse, as defined in Section 4.1.2.4, is accomplished.

4.2 Existing Construction

Existing facilities undergoing modernization should be upgraded to new construction requirements when required by the project specific facility security risk assessment and where feasible. In addition, facilities undergoing modernization should, as a minimum, assess the potential for progressive collapse as the result of an abnormal loading event. The flowchart, shown in Figure 4.8, outlines the process for assessing the potential for progressive collapse in existing facilities. Findings of this analysis should be incorporated into the project-specific risk assessment, and shall be documented in accordance with the provisions in Section 1.5. The 'analysis' provisions contained in Section 4.1.2 concerning analysis techniques, procedure, analysis considerations and loading criteria, analysis criteria, material properties, and modeling guidance, shall also apply to existing construction.

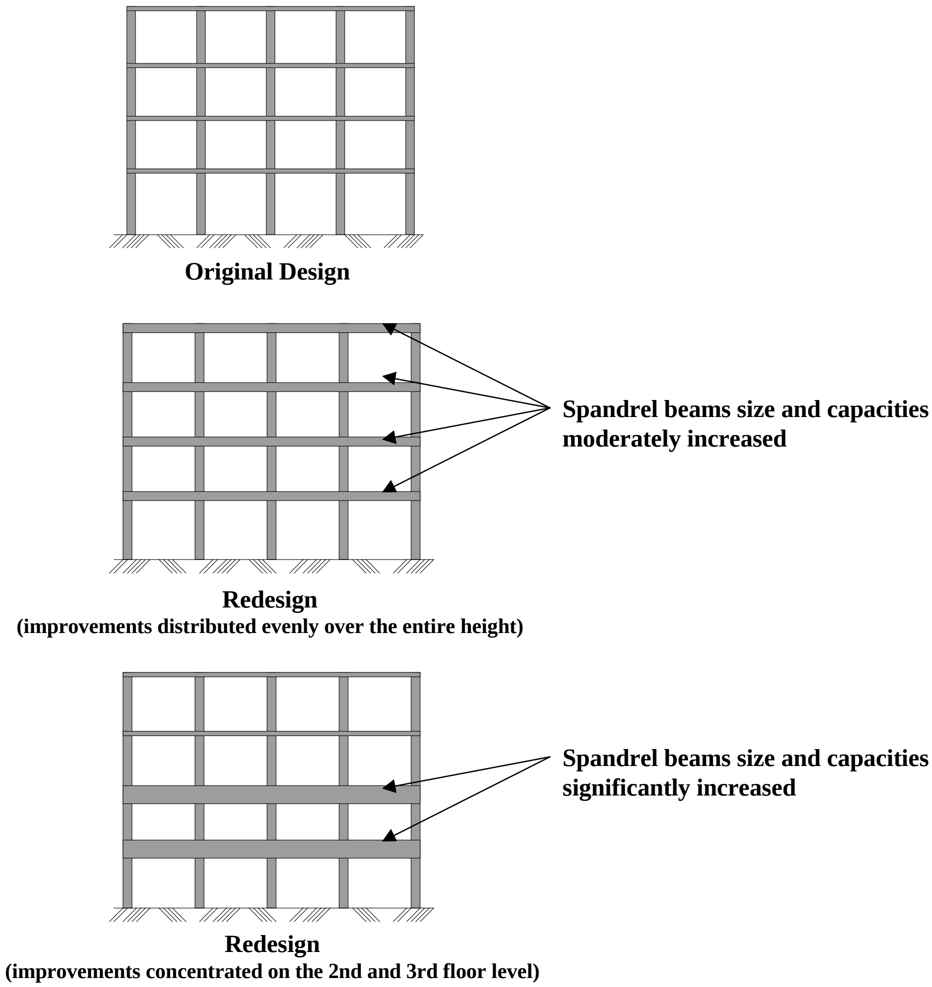


Figure 4.7. Possible approaches for the redesign of a structure that has been determined to have high potential for progressive collapse.

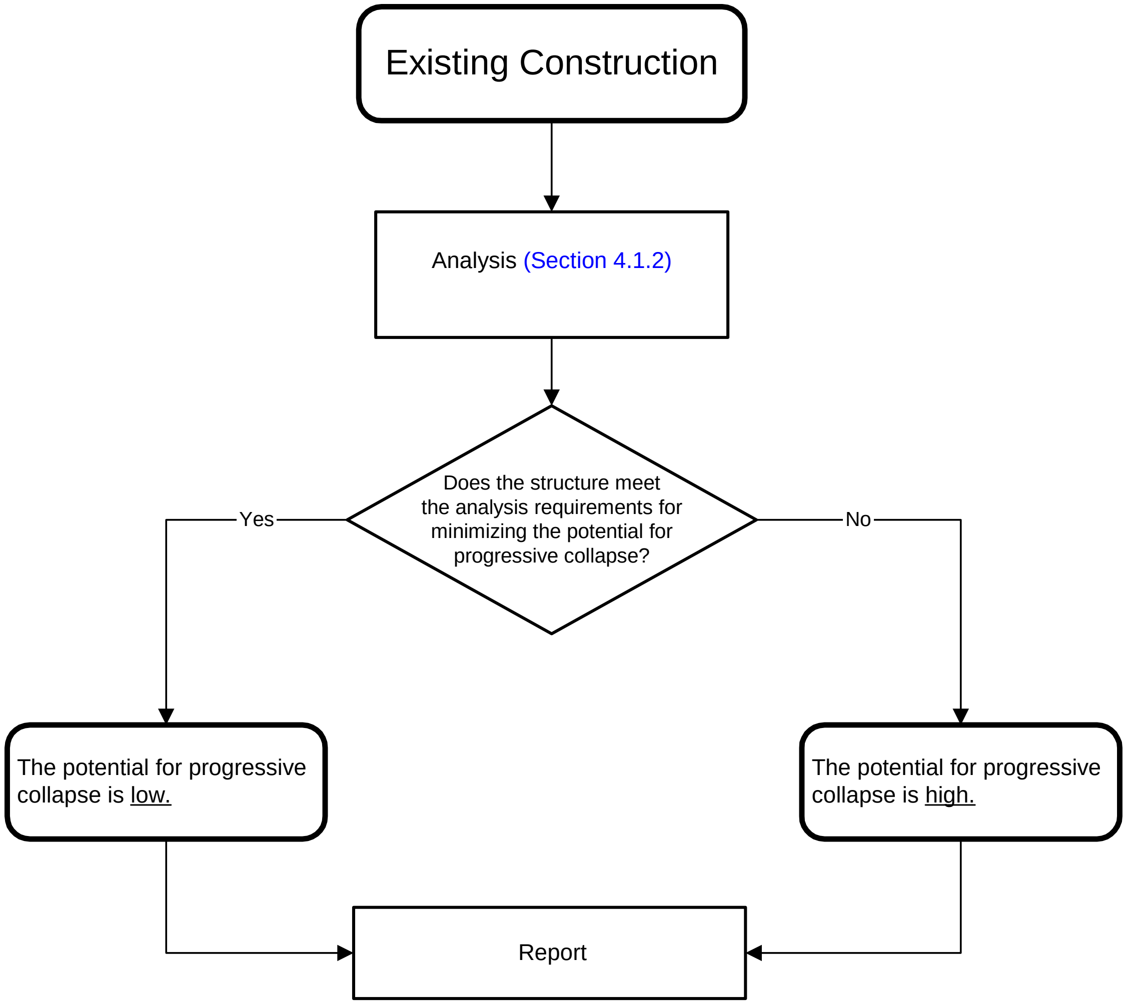


Figure 4.8. Process for assessing the potential for progressive collapse in existing Construction.

Section 5. Steel Frame Building Analysis and Design

5.1 New Construction

All newly constructed facilities shall be designed with the intent of reducing the potential for progressive collapse as a result of an abnormal loading event, regardless of the required level of protection. The process presented in these Guidelines consists of an analysis/redesign approach. This method is intended to enhance the probability that if localized damage occurs as the result of an abnormal loading event, the structure will not progressively collapse or be damaged to an extent disproportionate to the original cause of the damage. The flowchart, shown in Figure 5.1, outlines this process for reducing the potential for progressive collapse in newly constructed facilities.

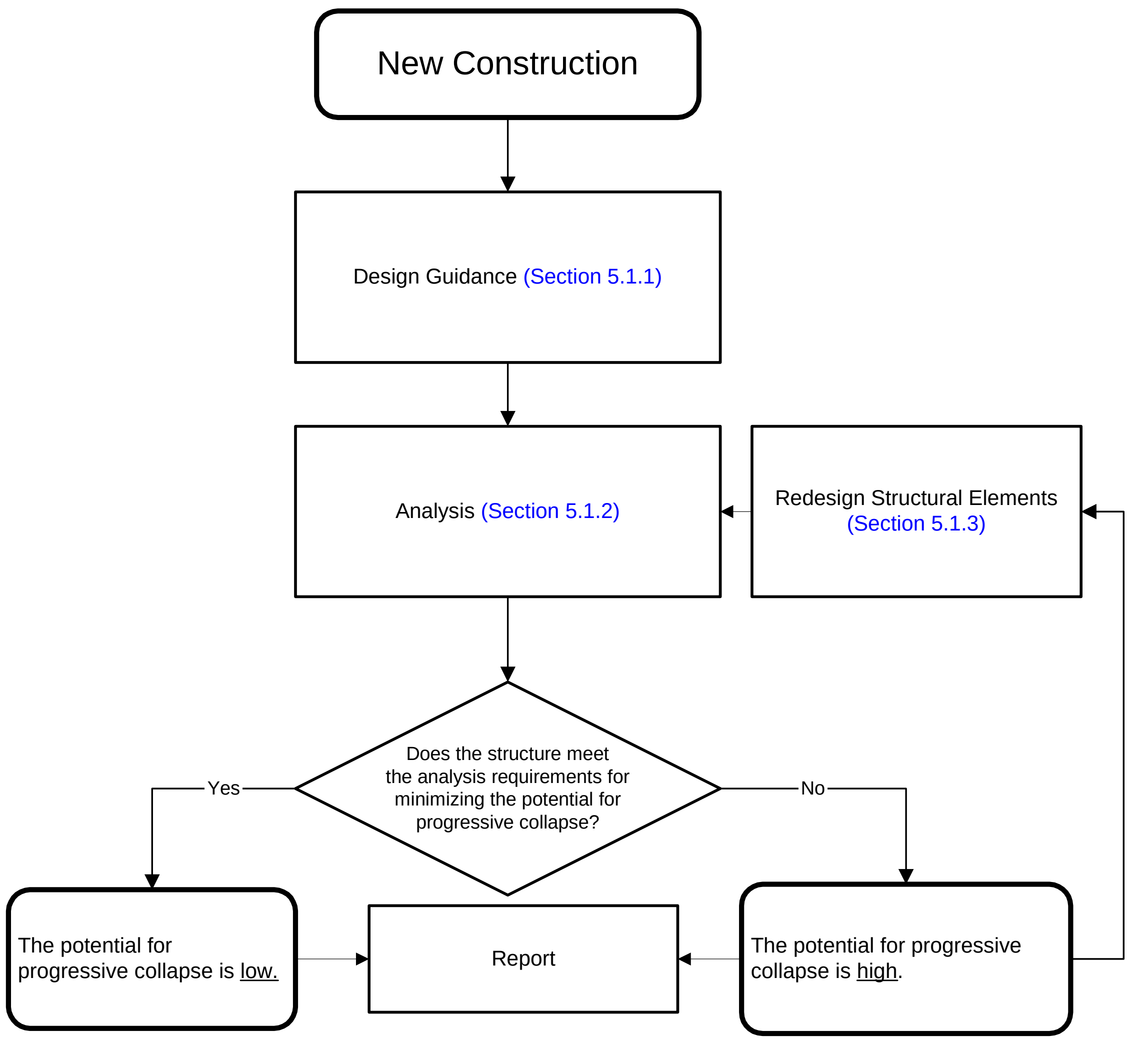


Figure 5.1. Process for reducing the potential for progressive collapse in new construction.

5.1.1 Design Guidance

Structural design guidance is provided in these Guidelines for addressing the mitigation of progressive collapse, as it may affect the *detailing* of local beam-to-column-to-beam connections, as well as the global configuration of primary and secondary structural steel girders, beams and columns. This structural design guidance, although not a requirement of these Guidelines, is provided for consideration during the initial structural design phase and prior to performing the progressive collapse analysis outlined in Section 5.1.2, to minimize the impact on the building's final design. These Guidelines should act as a supplement to the Interagency Security Committee (ISC) Security Design Criteria for New Federal Office Buildings and Major Modernization Projects, which states that mitigation of progressive collapse be addressed in the design of new structures.

It is critical that floor girders and beams be capable of spanning two full spans (i.e., a *double span* condition consisting of two full bays) as a minimum. This requires both beam-to-beam structural continuity across the removed column, as well as the ability of girders and beams to deform flexurally well beyond their elastic limit without experiencing structural collapse. It is therefore imperative that the following *local* beam-to-column connection characteristics be ascertained and/or implemented during the initial phases of structural design, as well as the *global* frame recommendations made herein. The incorporation of these features will provide for a much more robust steel frame structure and increase the probability of achieving a low potential for progressive collapse when performing the analysis procedure in Section 5.1.2.

5.1.1.1 Local Considerations

Discrete beam-to-beam continuity – Providing *discrete beam-to-beam continuity* (as defined in Section 2.1) in the design of steel frame connections is considered *fundamental* to mitigating progressive collapse in newly constructed steel frame structures. Accordingly, a structural engineer should be able to demonstrate that a proposed beam-to-column connection system for a given project provides a structurally-redundant clearly defined beam-to-beam continuity link across a column that is capable of independently redistributing gravity loads for a multiple-span condition.

Connection resilience – Providing *connection resilience* (as defined in Section 2.1) in the design of steel frame connections is considered *essential* to mitigating progressive collapse in newly constructed steel frame structures. Accordingly, a structural engineer should be able to demonstrate that a proposed beam-to-column connection system for a given project provides a connection geometry that exhibits the physical attributes needed to mitigate the effects of instantaneous column loss.

Examples of good detailing practice that ensures ductile behavior in steel frame connections include:

1. The configuration of weld line geometries such that a given line of weld metal is loaded primarily along its length in shear, not in tension across its throat, inherently results in robust performance, and minimizes the potential for highly

restrained condition due to the expected shrinkage of newly deposited welds during the cooling process.

2. The configuration of the connection's base metal elements (i.e., girder, beams, columns, and structural plates) such that the applied loading is resisted nearly uniformly across a beam flange, column flange, or a beam flange cover plate, in a direction parallel to its rolled direction, inherently results in ductile performance.

Connection redundancy – Providing *connection redundancy* (as defined in Section 2.1) in the design of steel frame connections is considered *essential* to mitigating progressive collapse in newly constructed steel frame structures. Accordingly, a structural engineer must select a beam-to-column-to-beam connection configuration that provides positive, multiple and clearly defined beam-to-beam load paths.

An example illustrating the physical characteristics of a typical steel frame beam-to-column-to-beam 'traditional' moment connection scheme is shown in Figure 5.2, including a depiction of its anticipated flexural deformation under the influence of gravity loads only (i.e., dead and live loads). As shown in Figure 5.3, when the beam-to-column-to-beam connection is subjected to column removal, a double span condition is created for the beam. It is questionable whether the structural configuration illustrated in Figure 5.2 will be capable of effectively redistributing loads when a primary support column is removed, as shown in Figure 5.3. Depending upon the type of beam-to-column-to-beam connections used, structural continuity may be compromised under such events. Accordingly, such configurations may be vulnerable to progressive collapse. Appendix D provides a listing of various connection types. Appendix D identifies both pre-Northridge and post-Northridge moment connection types, both public domain and proprietary, and provides descriptions and isometric sketches, which highlight the important attributes and differences in their connection geometries.

Connection Rotational Capacity - Only steel frame beam-to-column connection types that have been *qualified* by full-scale testing to verify that they provide the required level of *connection rotational capacity* should be used in the design of new buildings to mitigate progressive collapse, because of their proven ductility. The ability of a girder or beam in a steel frame system to structurally accommodate a double span condition, created by a "missing column" scenario, is considered *fundamental* in mitigating progressive collapse. Research has shown that in many cases, in order to successfully achieve a double span condition, the beam-to-column connection, while demonstrating the connection's ability to force the formation of plastic hinges in the girder or beam outside the beam-to-column connection, and while maintaining sufficient axial load carrying capacity in both the beam and the connection that joins beam to beam across the removed column.

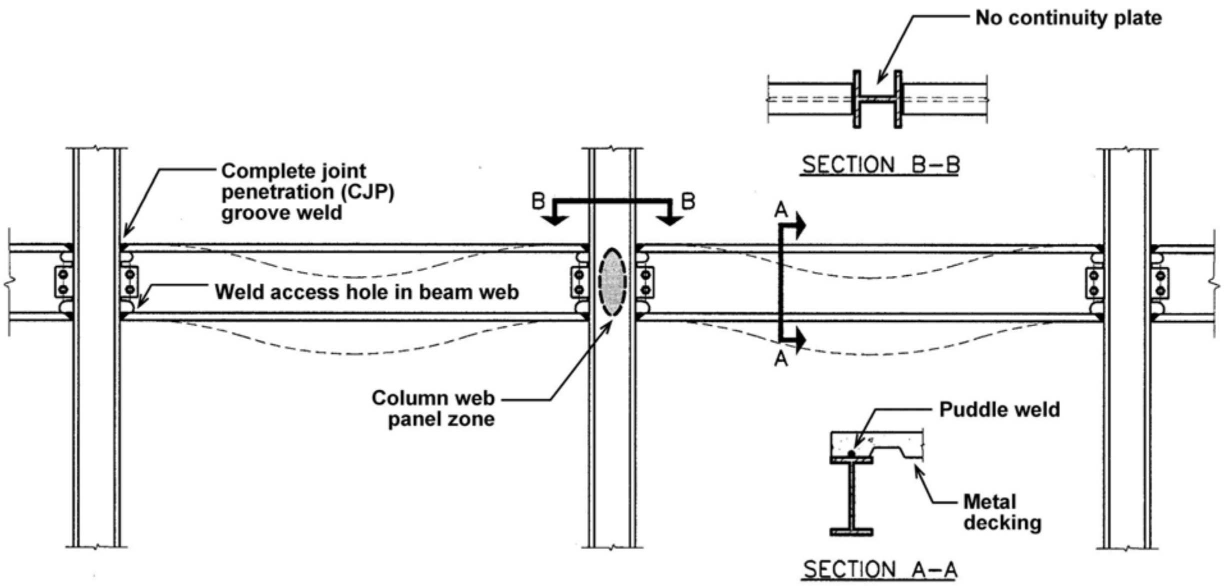


Figure 5.2. A sketch depicting a steel frame beam-to-column-to-beam 'traditional' moment connection scheme prior to removal of primary column support.

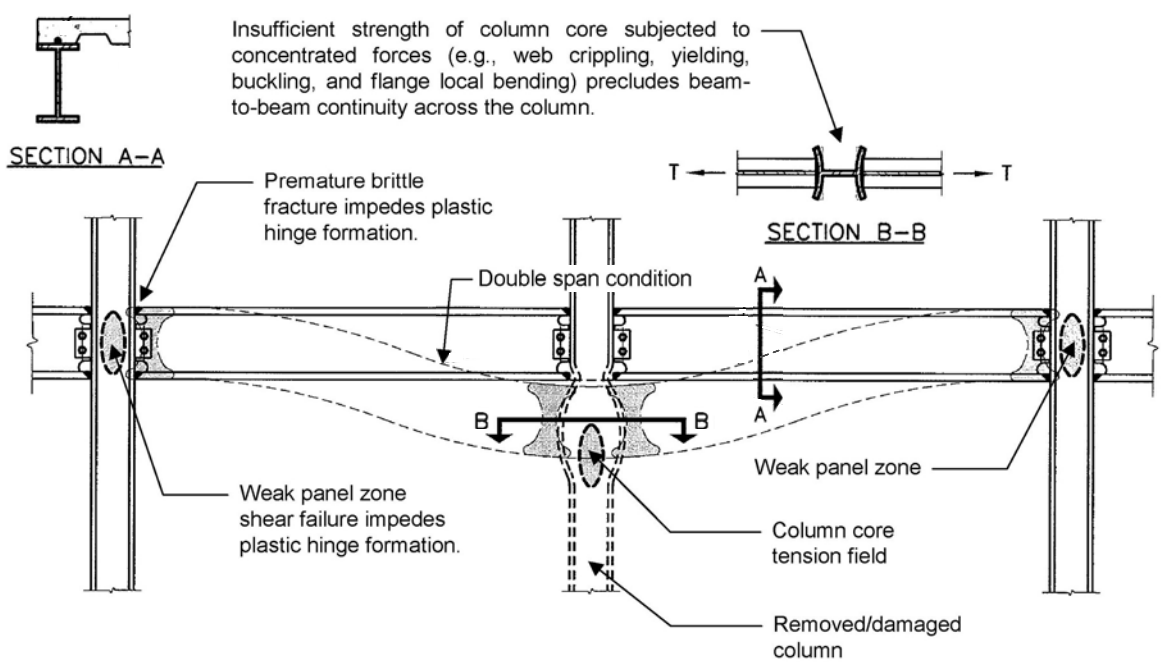


Figure 5.3. Response of the framing scheme shown in Figure 5.2, after the loss of primary column support, shows the inability to protect against progressive collapse.

Accordingly, the structural engineer should provide compliance documentation to the GSA, in accordance with the provisions of Section 1.5, for establishing a proposed connection's rotational capacity qualification by testing, which must comply with the most current provisions of Appendix S, Seismic Provisions for Structural Steel Buildings, dated May 21, 2002, published by the American Institute of Steel Construction (AISC). The proposed connection type should use similar beam spans, beam sizes, column sizes, column orientation (major axis versus weak axis), and level of anticipated column web panel zone participation, as those already tested.

By demonstrating the ability of a given beam-to-column moment connection to achieve the level of inelastic rotational capacity stipulated by the cited AISC standard, albeit conservative for certain connection types, the ability of a beam to accommodate the anticipated rotational demand on the connection created by a double span condition can be ascertained, using a nationally recognized fully-developed procedure. Until additional research has been conducted, including full-scale monotonic testing of double span conditions subjected to instantaneous column removal under sustained gravity loads, the use of the AISC standard is considered to be both practical and prudent.

Determination of Connection Strength Demands - In order to complete the design or investigation of a given beam-to-column connection type, for example, the sizing (or investigation of design adequacy) of the various plates, bolts, and joining welds that make up the connection, it is *essential* to determine the shears and flexural strength demands at *each critical section*. Each connection type may have different critical sections for which the *connection strength demand* must be calculated, depending on its particular connection geometry and the kinds of connection elements employed. The connection strength demand for each critical section may be calculated by first determining the *location* of the plastic hinge for a particular connection type being considered for the steel frame design. This location determines the magnitude of the moment at this point on the moment diagram, which is numerically equal to the plastic moment capacity of the beam (i.e., $M_p = F_{ye}Z_x$). Note that M_p includes the use of the expected yield strength F_{ye} which accounts for over strength in the nominal yield strength of the beam, plus strain hardening of the beam (see FEMA 356). The location of the plastic hinge will normally be identified in the full-scale cyclic test report being used to qualify the connection for the required rotational capacity. The next step requires the drawing of the beam's moment diagram for a double span condition, as illustrated in Figure 5.4. By superimposing the location of the known plastic hinge on the moment diagram, the ramping up effects, starting from the known location of M_p and continuing to the centerline of column, can be readily determined for addressing the increased connection strength demand on all critical sections to be designed or investigated.

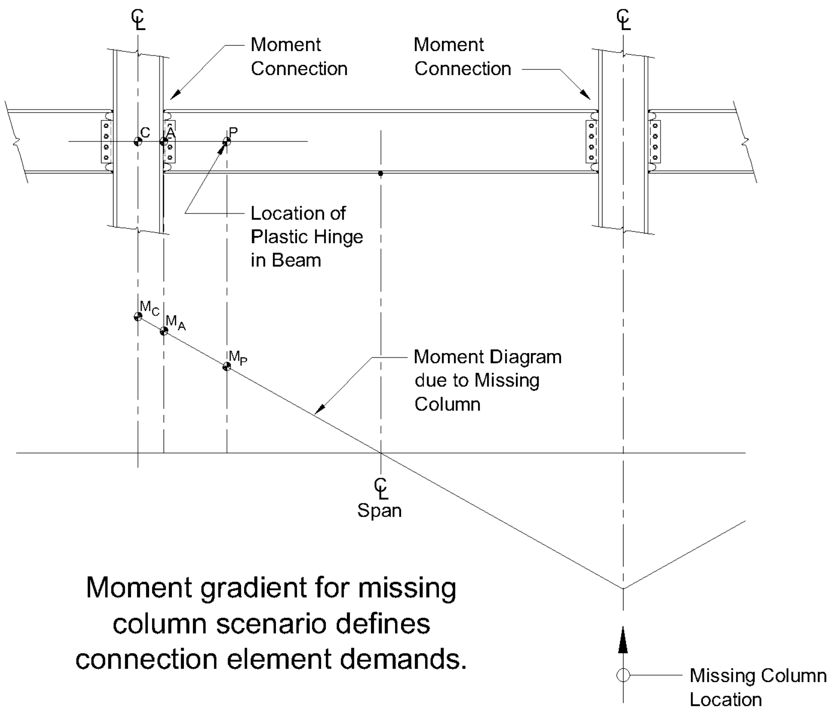


Figure 5.4. Moment gradient for 'missing column' scenario to determine connection strength demand at each critical connection element. Note: bending moment can increase significantly from beam's plastic hinge location to centerline of column.

5.1.1.2 Global Considerations

Global Frame Redundancy - The use of redundant lateral and vertical force resisting steel frame systems are highly encouraged when considering progressive collapse. Global frame redundancy tends to promote an overall more robust structure and helps to ensure that alternate load paths are available in the case of a structural element(s) failure. Additionally, global frame redundancy generally provides multiple locations for yielding to occur, which increases the probability that damage may be constrained.

Note:

A set of design procedures for preliminarily sizing structural components is included in Appendix B. These procedures are not required by these Guidelines. However, these procedures can be used to preliminarily size and detail elements prior to performing the progressive collapse analysis presented in Section 5.1.2.

5.1.2 Analysis

The following linear elastic, static analysis approach may be used to assess the potential for progressive collapse in all new and upgraded construction. Other analysis approaches may also be used, such as those discussed in Section 2.3, but the analysis considerations (Section 5.1.2.3) and allowable extents of collapse (Section 5.1.2.4), must be used in the assessment of the potential for progressive collapse.

The following procedure uses a linear elastic, static approach coupled with the following:

- Criteria for assessing the analysis results
- A suite of analysis cases
- Specific loading criteria to be used in the analysis

5.1.2.1 Analysis Techniques

The following analysis procedure shall be performed using well-established linear elastic, static analysis techniques. It is recommended that 3-dimensional analytic models be used to account for potential 3-dimensional effects and avoid overly conservative solutions. Nevertheless, 2-dimensional models may be used provided that the general response and 3-dimensional effects can be adequately idealized.

5.1.2.2 Procedure

The potential for progressive collapse can be determined by the following procedure.

Step 1. The components and connections of both the primary and secondary structural elements shall be analyzed for the case of an instantaneous loss in primary vertical support. The applied downward loading shall be consistent with that presented in Section 5.1.2.3.

Step 2. The results from the analyses performed in Step 1 shall be evaluated by utilizing the analysis criteria defined in Section 5.1.2.4.

Note:

If the analysis results show that the structural member(s) and/or connections/joints <u>are not in compliance</u> with the analysis criteria presented in Section 5.1.2.4 (i.e., the member and/or connection capacities are greatly exceeded and it is unlikely that the structure is capable of effectively redistributing loads), <u>the facility exhibits a high potential for progressive collapse</u> and the user shall redesign the members and/or connections/joints consistent with the procedure outlined in Section 5.1.3.1. However, if the analysis results show that the structural member(s) and/or connections/joints <u>are in compliance</u> with the analysis criteria presented in Section 5.1.2.4, <u>the facility exhibits a low potential for progressive collapse</u> and requires no further progressive collapse considerations.

5.1.2.3 Analysis Considerations and Loading Criteria

The following analysis considerations shall be used in the assessment for progressive collapse for typical structural configurations. Several atypical structural configurations are addressed in Section 5.1.2.3.2.

5.1.2.3.1 Typical Structural Configurations

The analysis scenarios selected for investigation shall be sufficient in number to include all unique structural differences that could affect the outcome of predicting either the low or high potential for progressive collapse. Such unique structural differences shall include, but are not limited to, differences in beam-to-beam connection type (simple vs. moment connection); significant changes in beam span and/or size; and significant changes in column orientation or strength (weak vs. major axis). Additional analysis scenarios may be required for such cases.

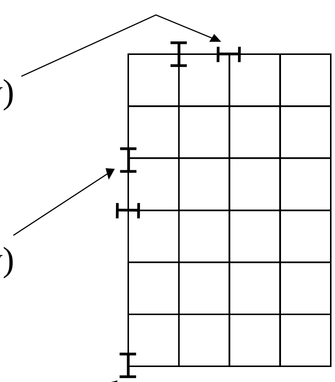
For facilities that have a relatively simple, uniform, and repetitive layout (for both global and local connection attributes), with <u>no atypical structural configurations</u>, the following analysis scenarios may be used:

<u>Framed Structures</u>

Exterior Considerations

The following exterior analysis cases shall be considered in the procedure outlined in Section 5.1.2.2.

1. Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at or near the middle of the short side of the building.
2. Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at or near the middle of the long side of the building.
3. Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at the corner of the building.

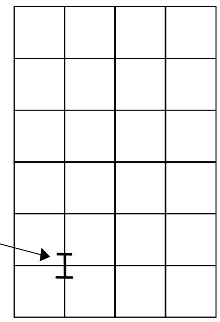


<u>Plan View</u>

Interior Considerations

Facilities that have underground parking and/or uncontrolled public ground floor areas shall use the following interior analysis case(s) in the procedure outlined in Section 5.1.2.2.

1 Analyze for the instantaneous loss of 1 column that extends from the floor of the underground parking area or uncontrolled public ground floor area to the next floor (1 story). The column considered should be interior to the perimeter column lines.



Plan View

Shear/Load Bearing Wall Structures

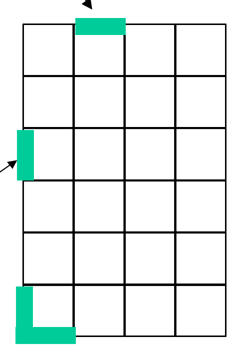
Exterior Considerations

There may be combination structures that use steel framing combined with load bearing wall sections. In this case, the following exterior analysis cases shall be considered in the procedure outlined in Section 5.1.2.2.

1 Analyze for the instantaneous loss of one structural bay or 30 linear feet of an exterior wall section (whichever is less) for one floor above grade, located at or near the middle of the short side of the building.

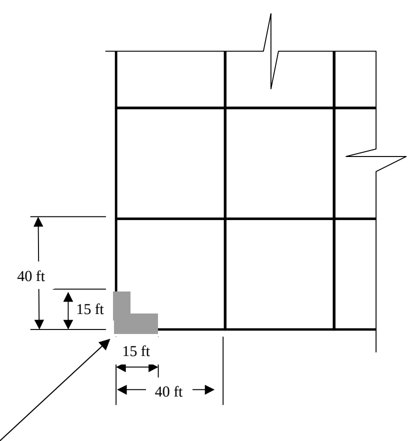
2 Analyze for the instantaneous loss of one structural bay or 30 linear feet of an exterior wall section (whichever is less) for one floor above grade, located at or near the middle of the long side of the building.

3 Analyze for the instantaneous loss of the entire bearing wall along the perimeter at the corner structural bay or for the loss of 30 linear feet of the wall (15 ft in each major direction) (whichever is less) for one floor above grade*.



Plan View

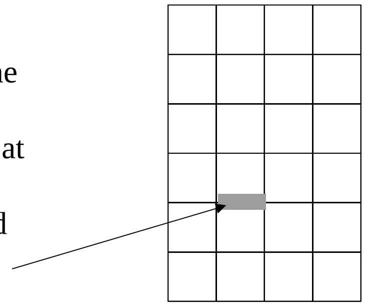
* The loss wall section for the corner consideration must be continuous and include the corner. For example, if the structural bay of a facility is 40 ft by 40 ft, the wall section that would require removal consists of 30 ft of the wall beginning at the corner and extending 15 ft in each major direction.



Interior Considerations

Facilities that have underground parking and/or uncontrolled public ground floor areas shall use the following interior analysis cases in the procedure outlined in Section 5.1.2.2.

1 Analyze for the instantaneous loss of one structural bay or 30 linear feet of an interior wall section (whichever is less) at the floor level of the underground parking area and/or uncontrolled ground floor area. The wall section considered should be interior to the perimeter bearing wall line.



Plan View

Analysis Loading

For static analysis purposes the following vertical load shall be applied downward to the structure under investigation:

$$\text{Load} = 2(DL + 0.25LL) \tag{5.1}$$

where,

DL = dead load
LL = live load

Note:

Depending on the facility characteristics and/or the outcome of the exemption process, the user may only be required to perform one of the analysis cases. For example, if the facility does not contain any uncontrolled parking areas and/or public areas, the user will not be required to perform the analyses for the interior considerations.

5.1.2.3.2 Atypical Structural Configurations.

All structures are generally unique and are often not *typical* (i.e., buildings often contain distinguishing structural features or details), hence, developing a set of analysis considerations that applies to every facility is impractical. Thus, the user of this guideline must use engineering judgment to determine critical analysis scenarios that should be assessed, in addition to the situations presented in Section 5.1.2.3.1. The intent of these provisions should be reflected in these analysis scenarios. Specifically, the scenarios should consider cases where loss of a vertical support (column or wall) could lead to disproportionate damage. Possible structural configurations that may result in an atypical structural arrangement include, but are not limited to, the following configurations:

- Combination Structures
- Vertical Discontinuities/Transfer Girders
- Variations in Bay Size/Extreme Bay Sizes
- Plan Irregularities
- Closely Spaced Columns

These atypical structural configurations are described in more detail in Appendix A.

5.1.2.4 Analysis Criteria

Structural collapse resulting from the instantaneous removal of a primary vertical support shall be limited. Typically, the allowable collapse area for a building will be based on the structural bay size. However, to account for structural configurations that have abnormally large structural bay sizes, the collapsed region will also be limited to a reasonably sized area. The allowable extent of collapse for the instantaneous removal of a primary vertical support member along the exterior and within the interior of a building is defined as follows.

Exterior Considerations

The maximum allowable extents of collapse resulting from the instantaneous removal of an exterior primary vertical support member one floor above grade shall be confined to:

1. the structural bays directly associated with the instantaneously removed vertical member in the floor level directly above the instantaneously removed vertical member

or

2. 1,800 ft^2 at the floor level directly above the instantaneously removed vertical member

whichever is the smaller area (Figure 5.5.a).

Interior Considerations

The allowable extents of collapse resulting from the instantaneous removal of an interior primary vertical support member in an uncontrolled ground floor area and/or an underground parking area for one floor level shall be confined to:

1. the structural bays directly associated with the instantaneously removed vertical member

or

2. 3,600 ft^2 at the floor level directly above the instantaneously removed vertical member

whichever is the smaller area (Figure 5.5.b). If there is no uncontrolled ground floor area and/or an underground parking area present in the facility under evaluation, the internal consideration is not required.

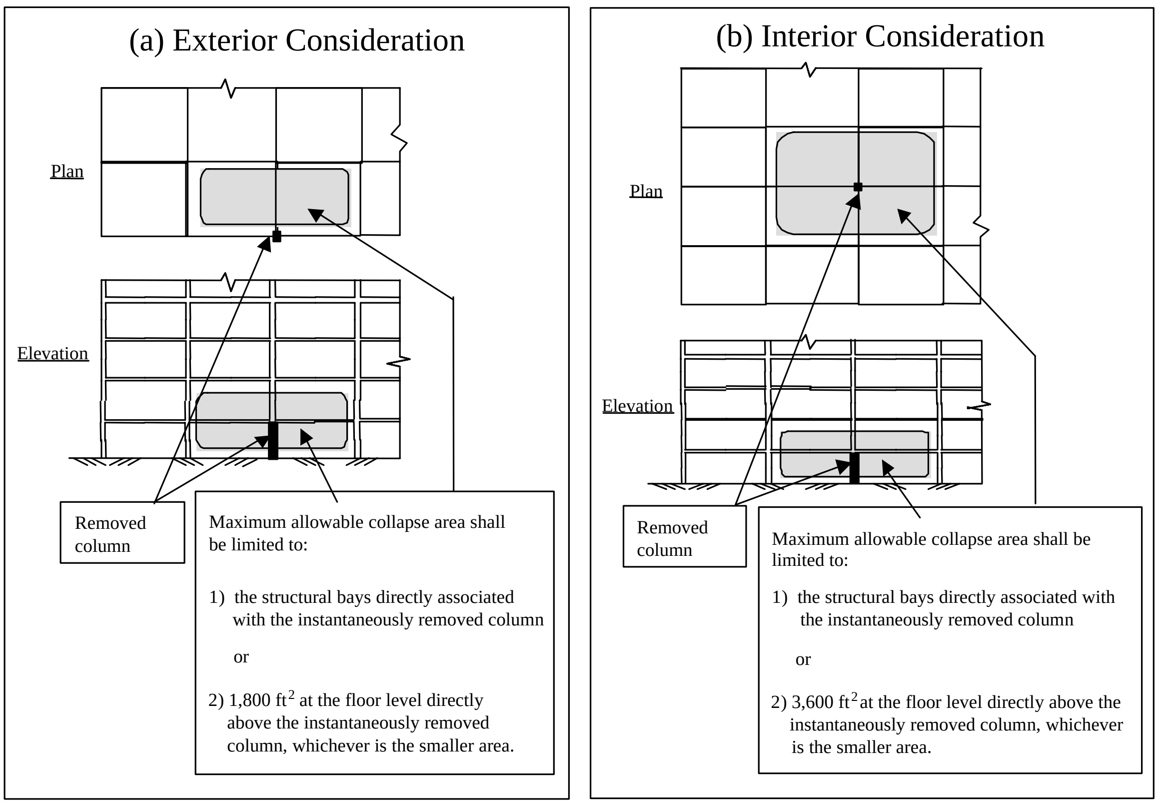


Figure 5.5. An example of maximum allowable collapse areas for a structure that uses columns for the primary vertical support system.

Acceptance Criteria

An examination of the linear elastic analysis results shall be performed to identify the magnitudes and distribution of potential demands on both the primary and secondary structural elements for quantifying potential collapse areas.

Upon removing the selected column from the structure, an assessment is made as to which beams, girders, columns, joints or connections, have exceeded their respective maximum allowable demands. The magnitude and distribution of demands will be indicated by **D**emand-**C**apacity **R**atios (*DCR*). Member ends exceeding their respective DCR values will then be released and their end moments re-distributed. These values and approaches are based, in part, on the methodology presented in the following references:

- NEHRP Commentary on the Guidelines for the Seismic Rehabilitation of Buildings (FEMA 274). Issued by Federal Emergency Management Agency, October 1997.
- Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356). Issued by Federal Emergency Management Agency, November 2000.
- Interim Antiterrorism/Force Protection Construction Standards, Guidance on Structural Requirements (Draft). Issued by Department of Defense, March 2001.
- Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects. U.S. General Services Administration and Applied Research Associates, Inc. November 2000.

Acceptance criteria for primary and secondary structural components shall be determined as:

$$DCR = \frac{Q_{UD}}{Q_{CE}} \tag{5.2}$$

where,

Q_{UD} = Acting force (demand) determined in component or connection/joint (moment, axial force, shear, and possible combined forces)

Q_{CE} = Expected ultimate, un-factored capacity of the component and/or connection/joint (moment, axial force, shear and possible combined forces)

Using the DCR criteria for the linear elastic approach, structural elements and connections with DCR values exceeding those given in Table 5.1 are considered to be severely damaged or collapsed. For atypical structural configurations, a value of (3/4)*DCR should be used (factor of 3/4 for uncertainties). Under no conditions is a DCR less than 1.0 required.

Note:

*The criteria for atypical structural configurations (i.e., DCR = (3/4)*DCR) may be limited to the 'atypical' region if this is localized. For example, consider a structure that uses transfer girders along one face of the perimeter and a typical structural configuration for the remainder of the structure. The perimeter structural bays along the side of the building that utilizes transfer girders shall use a DCR that is multiplied by a reduction factor of 3/4, but the remainder of the building shall use a DCR per Table 5.1 for the assessment of the potential for progressive collapse.*

The approach used in estimating the magnitude and distribution of the potential inelastic demands and displacements used in these Guidelines is similar to the 'm-factor' approaches currently employed in FEMA 273 and 356 for linear elastic analysis methods.

A variety of connection illustrations are provided in Appendix D. These illustrations should aid the engineer in selecting the appropriate connection DCR values from Table 5.1.

The step-by-step procedure for conducting the linear elastic, static analysis follows.

Step 1. Remove a vertical support from the location being considered and conduct a linear-static analysis of the structure as indicated in Section 5.1.2.2. Load the model with 2(DL + 0.25LL).

Step 2. Determine which members and connections have DCR values that exceed the acceptance criteria provided in Table 5.1. <u>If the DCR for any member end or connection is exceeded based upon shear force, the member is to be considered a failed member. In addition, if the flexural DCR values for both ends of a member or its connections, as well as the span itself, are exceeded (creating a three hinged failure mechanism – Figure 2.2), the member is to be considered a failed member.</u> Failed members should be removed from the model, and all dead and live loads associated with failed members should be redistributed to other members in adjacent bays.

Step 3. For a member or connection whose Q_{UD}/Q_{CE} ratio exceeds the applicable flexural DCR values, place a hinge at the member end or connection to release the moment. This hinge should be located at the center of flexural yielding for the member or connection. Use rigid offsets and/or stub members from the connecting member as needed to model the hinge in the proper location. For yielding at the end of a member the center of flexural yielding should not be taken to be more than ½ the depth of the member from the face of the intersecting member, which is usually a column (Figure 5.6).

Step 4. At each inserted hinge, apply equal-but-opposite moments to the stub/offset and member end to each side of the hinge. The magnitude of the moments should equal the expected flexural strength of the moment or connection, and the direction of the moments should be consistent with direction of the moments in the analysis performed in Step 1.

Step 5. Re-run the analysis and repeat Steps 1 through 4. Continue this process until no DCR values are exceeded. **<u>If moments have been re-distributed throughout the entire building and DCR values are still exceeded in areas outside of the allowable collapse region, the structure will be considered to have a high potential for progressive collapse.</u>**

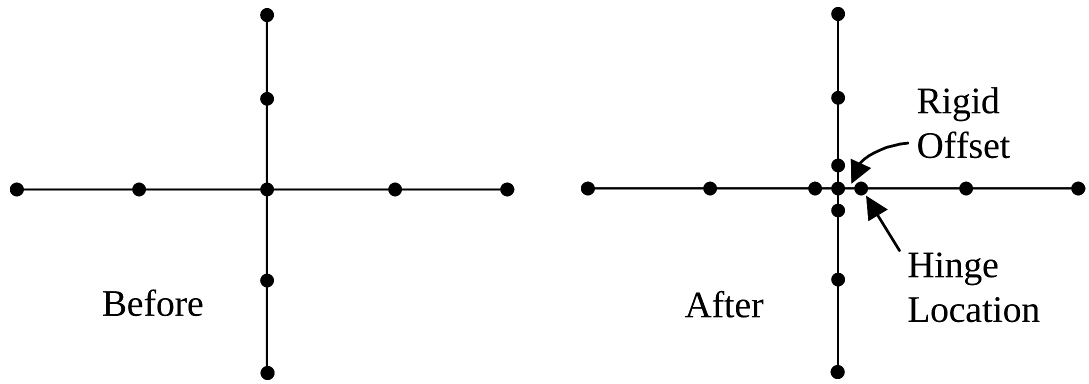


Figure 5.6. Rigid offset placement.

Table 5.1. Acceptance criteria for linear procedures— steel frame components.

Component/Action	Values for Linear Procedures: *DCR*
Beams – flexure	
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{418}{\sqrt{F_{ye}}}$	3
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{640}{\sqrt{F_{ye}}}$	2
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.
Columns – flexure	
For 0 < *P/P*$_{CL}$ < 0.5	
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{300}{\sqrt{F_{ye}}}$	2
b. $\frac{b_f}{2t_f} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{460}{\sqrt{F_{ye}}}$	1.25
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.

Table 5.1. Acceptance criteria for linear procedures— steel frame components (continued).

Component/Action	Values for Linear Procedures *DCR*
Columns – flexure	
For *P/P*$_{CL}$ > 0.5	
a. $\frac{b_f}{2t_f} \leq \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \leq \frac{260}{\sqrt{F_{ye}}}$	1
b. $\frac{b_f}{t_w} \geq \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \geq \frac{400}{\sqrt{F_{ye}}}$	1
Columns Panel Zone – Shear	2
Column Core – Concentrated Forces[2]	1.5
Fully Restrained Moment Connections	
Pre-Northridge (Pre 1995)	
Welded unreinforced flange (WUF)	2
Welded flange plate (WFP)	2
Welded cover plated flanges	2
Bolted flange plate (BFP)	2
Post-Northridge (FEMA 350) Public Domain	
Improved WUF-bolted web	2
Improved WUF-welded web	2
Free flange	2
Welded top and bottom haunches	2
Reduced beam section	2
Post-Northridge (FEMA 350) Proprietary[3]	
Proprietary System	≤3 (See Footnote 3)

Table 5.1. Acceptance criteria for linear procedures— steel frame components (continued).

Component/Action	Values for Linear Procedures DCR
Partially Restrained Moment Connection	
Top and bottom clip angle	
a. Shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Tension failure of horizontal leg of angle	1.5
c. Tension failure of rivets or bolts	1.5
d. Flexural Failure of angle	3
Double split tee	
a. Shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Tension failure of rivets or bolts	1.5
c. Tension failure of split tee stem	1.5
d. Flexural Failure of split tee	3
Bolted flange plate	
a. Failure in net section of flange plate or shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Weld failure or tension failure on gross section of plate	1.5
Bolted end plate	
a. Yield of end plate	3
b. Yield of rivets or bolts	2 (rivets); 1.5 (high strength bolts)
c. Failure of weld	1.5
Composite top and clip angle bottom	
a. Failure of deck reinforcement	2
b. Local flange yielding and web crippling of column	3
c. Yield of bottom flange angle	3
d. Tensile yield of rivets or bolts at column flange	1.5 (rivets); 1 (high strength bolts)
e. Shear yield of beam flange connections	2
Shear connection with or without slab	2

1. Notation for Table 5.1:

bf = Width of the compression flange
F_{ye} = Expected yield strength
h = Distance from inside of compression flange to inside of tension flange
t_w = Web thickness
P_{CL} = Lower bound compression strength of the column
P = Axial force in member taken as Q_{uf}
t_f = Flange thickness
d = Beam depth
d_{bg} = Depth of the bolt group

2. Column core concentrated force capacity shall be determined from AISC (1993) LRFD Specifications equations K1-1, K1-2, K1-4 and K1-8.
3. A DCR of 2 will be used for all untested proprietary fully restrained moment connections. A DCR of 1 will be used for all other untested proprietary connections. Tested proprietary connections must have documented test results that justify using DCR values greater than these. Under no circumstances should a DCR value exceeding 3 be used for any proprietary connection.
4. DCR values are for connection to strong axis of column. For connections to weak axis of column (Figure D 3 Appendix D) treat as atypical (DCR*0.75).
5. No DCR values less than 1.0 are required, even for atypical conditions.

5.1.2.5 Material Properties

For these Guidelines the design material strengths may be increased by a strength-increase factor to determine the expected material strength. These should be used only in cases where the designer or analyst is confident in the actual state of the facility's materials. These values are provided in Table 5.2 and Table 5.3.

Table 5.2. Default lower-bound material strengths[1] — steel frame components.

Properties based on ASTM and AISC Structural Steel Specification Stresses				
Date	**Specification**	**Remarks**	**Tensile Strength[2], ksi**	**Yield Strength[2], ksi**
1900	ASTM, A9	Rivet Steel	50	30
	Buildings	Medium Steel	60	20
1901-1908	ASTM, A9	Rivet Steel	50	25
	Buildings	Medium Steel	60	30
1909-1923	ASTM, A9	Structural Steel	55	28
	Buildings	Rivet Steel	46	23
1924-1931	ASTM, A7	Structural Steel	55	30
	Buildings	Rivet Steel	46	25
	ASTM, A9	Structural Steel	55	30
		Rivet Steel	46	25
1932	ASTM, A140-32T issued as a tentative revision to ASTM, A9 (Buildings)	Plates, Shapes, Bars	60	33
		Eyebar flats unannealed	67	36
1933	ASTM, A140-32T discontinued and ASTM, A9 (Buildings) revised Oct.30, 1933	Structural Steel	55	30
	ASTM, A9 tentatively revised to ASTM, A9-33T (Buildings) revised Oct.30, 1933	Structural Steel	52	28
	ASTM, A140-32T adopted as a standard	Rivet Steel	52	28
1934 on	ASTM, A9	Structural Steel	60	33
	ASTM, A141	Rivet Steel	52	28
1961 - 1990	ASTM, A36/A36M-00	Structural Steel		
	Group 1		62	44
	Group 2		59	41
	Group 3		60	39
	Group 4		62	37
	Group 5		70	41
1961 on	ASTM, A572, Grade 50	Structural Steel		
	Group 1		65	50
	Group 2		66	50
	Group 3		68	51
	Group 4		72	50
	Group 5		77	50
1990 on	A36/36M-00 & Dual Grade	Structural Steel		
	Group 1		66	49
	Group 2		67	50
	Group 3		70	52
	Group 4		70	49

1. Lower-bound values for material prior to 1960 are based on minimum specified values. Lower-bound values for material after 1960 are near minus one standard deviation values from statistical data.
2. The indicated values are representative of material extracted from the flanges of wide flange shapes.

Table 5.3. Factors to translate lower-bound properties to expected-strength steel properties.

Property	Year	Specification	Factor
Tensile Strength	Prior to 1961		1.10
Yield Strength	Prior to 1961		1.10
Tensile Strength	1961 - 1990	ASTM A36/A36M-00l	1.10
	1961 - present	ASTM A572/A572M-89, Group 1	1.10
		ASTM A572/A572M-89, Group 2	1.10
		ASTM A572/A572M-89, Group 3	1.05
		ASTM A572/A572M-89, Group 4	1.05
		ASTM A572/A572M-89, Group 5	1.05
	1990 - present	ASTM A36/A36M-00l & Dual Grade Group 1	1.05
		ASTM A36/A36M-00l & Dual Grade Group 2	1.05
		ASTM A36/A36M-00l & Dual Grade Group 3	1.05
		ASTM A36/A36M-00l & Dual Grade Group 4	1.05
Yield Strength	1961 - 1990	ASTM A36/A36M-00l	1.10
	1961 - present	ASTM A572/A572M-89, Group 1	1.10
		ASTM A572/A572M-89, Group 2	1.10
		ASTM A572/A572M-89, Group 3	1.05
		ASTM A572/A572M-89, Group 4	1.10
		ASTM A572/A572M-89, Group 5	1.05
	1990 - present	ASTM A36/A36M-00l Plates	1.10
		ASTM A36/A36M-00l Dual Grade, Group 1	1.05
		ASTM A36/A36M-00l Dual Grade, Group 2	1.10
		ASTM A36/A36M-00l Dual Grade, Group 3	1.05
		ASTM A36/A36M-00l Dual Grade, Group 4	1.05
Tensile Strength	All	Not Listed[1]	1.10
Yield Strength	All	Not Listed[1]	1.10

1. For materials not conforming to one of the listed specifications.

5.1.2.6 Modeling Guidance

General

The analytic model(s) used in assessing the potential for progressive collapse should be modeled as accurately as possible to the anticipated or existing conditions. This includes all material properties, design details, etc. In addition, the analyst shall realistically approximate the type of boundary conditions (e.g., fixed, simple, etc.), and should be aware of any limitations or anomalies of the software package(s) being used to perform the analysis.

Vertical Element Removal

The vertical element (i.e., the column, bearing wall, etc.) that is removed should be removed instantaneously. While the speed at which an element is removed has no impact on a static analysis, the speed at which an element is removed in a dynamic analysis may have a significant impact on the response of the structure. Because of this, it is recommended for the case where a dynamic analysis is performed, the vertical supporting element should be removed over a time period that is no more than 1/10 of the period associated with the structural response mode for the vertical element removal. Also the vertical element removal shall consist of the removal of the vertical element only. This removal should not impede into the connection/joint or horizontal elements that are attached to the vertical element at the floor levels. An example sketch illustrating the correct and incorrect way to remove a column is shown in Figure 5.7. It is critical that the user understand that the sketch is not representative of damage due to any specific threat (see Section 1.3 for discussion of member removal approach).

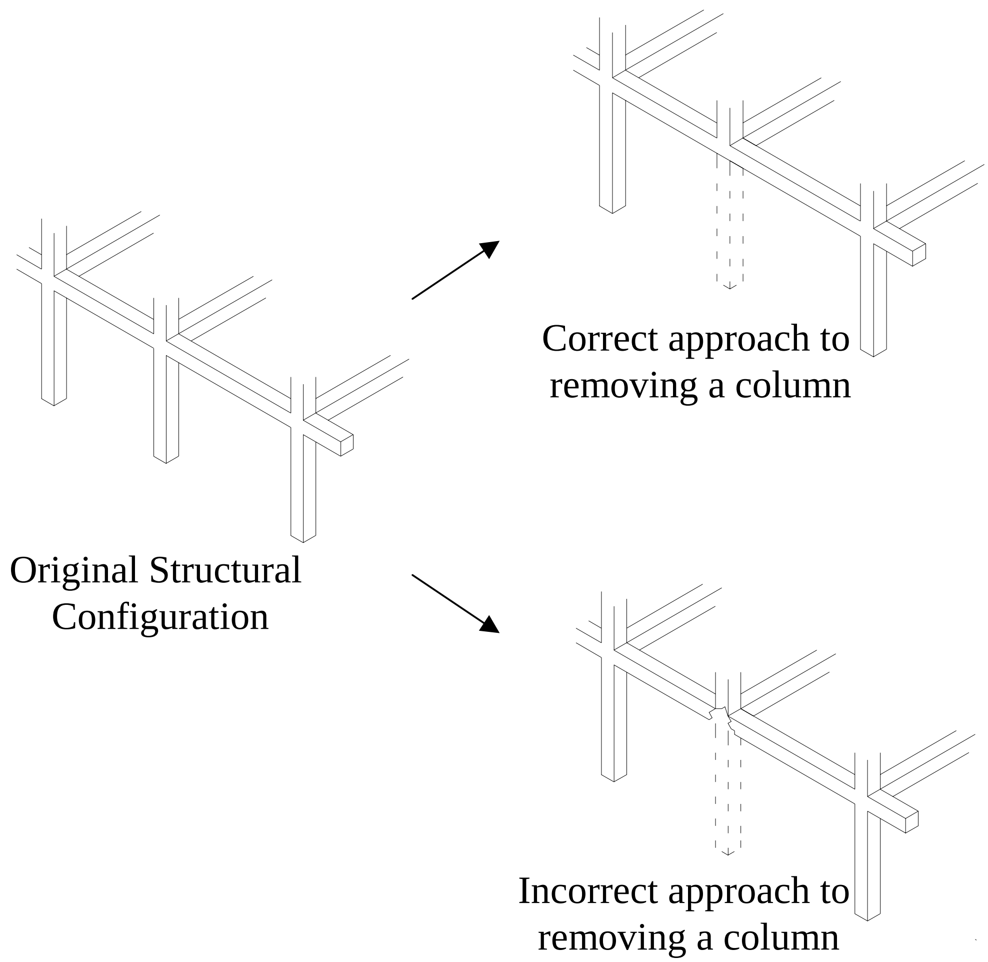


Figure 5.7. Sketch of the correct and incorrect approach for removing a column.

5.1.3 Redesign of Structural Elements

Structural configurations that are analyzed consistent with Section 5.1.2 and determined to have a high potential for progressive collapse shall be redesigned to a level that is consistent with a low potential for progressive collapse.

5.1.3.1 Procedure

The following steps shall be followed when redesigning the deficient structural elements identified in the analysis procedure (Section 5.1.2).

Step 1. As a minimum, the structural elements and/or connections identified as deficient in Section 5.1.2 should be redesigned consistent with the redistributed loading determined in this process in conjunction with the standard design requirements of the project specific building code(s) using well-established design techniques. The redesign criteria for typical and atypical structural configurations follow:

Typical Structural Configurations

Structural elements and beam-to-column connections must meet the DCR acceptance criteria in the design of deficient components and connections. If an approved alternate analysis criteria is used, the deficient components should be designed to, as a minimum, achieve the allowable values associated with that criteria for the redistributed loading.

Atypical Structural Configurations

Structural elements and beam-to-column connections must meet the DCR acceptance criteria in the design of deficient components and connections. Note that a reduction factor of 3/4 must be multiplied to the DCR value for atypical structures. If an approved alternate analysis criteria is used the deficient components should be designed to, as a minimum, achieve the allowable values associated with that criteria for the redistributed loading.

Step 2. Upon the completion of Step 1, the redesigned structure shall be reanalyzed consistent with analysis procedure outlined in Section 5.1.2.

Note:

In order to achieve the necessary design requirements, significant structural changes may be required, such as increasing member sizes, providing beam to beam continuity across the column (for steel frame connections), strengthening of moment resisting connections, etc. However, the design criteria for atypical structures may be limited to the 'atypical' region if this is localized. For example, consider a building that uses transfer girders along one face of the perimeter and a typical structural configuration for *the remainder of the structure. The perimeter structural bays shall be designed to meet DCR acceptance criteria with a reduction factor of 3/4 applied to the DCR value, but the remainder of the building may be designed with no reduction factors applied to the DCR value.*

It should be noted that to achieve a low potential for progressive collapse more than one iteration of the redesign/analysis process may be required. For example, a change in the size of structural members may alter the magnitude and distribution of the redistributed load.

The designer/analyst is not limited to a particular method for improving the original design with respect to the minimization of the potential for progressive collapse. For the example, in Figure 5.8, assume the results of Section 5.1.2 indicate the perimeter girders from the 2nd floor level to the 13th floor level for a given moment frame structure are not adequate in regards to the analysis criteria presented in Section 5.1.2.4. The designer has the freedom to evenly distribute an improved redesign from the 2nd floor level to the 6th floor level by introducing a Vierendeel truss to support the remaining floors from the 7th floor level to the 13th floor level (Figure 5.9), as long as the overall intent of minimizing the potential for progressive collapse, as defined in Section 5.1.2.4, is accomplished.

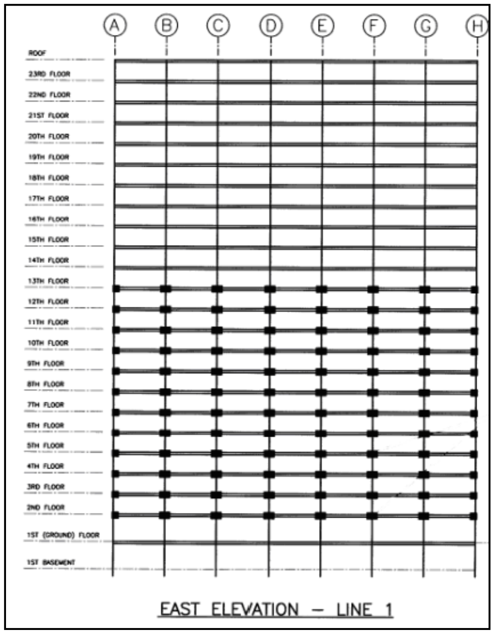


Figure 5.8. Extent of upgrade application using moment connections on exterior frames.

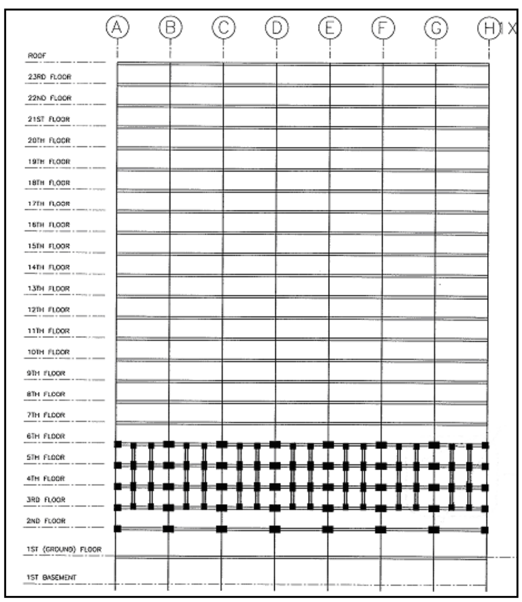


Figure 5.9. Extent of Vierendeel truss upgrade application using moment connections on exterior frames.

5.2 Existing Construction

Existing facilities undergoing modernization should be upgraded to new construction requirements when required by the project specific facility security risk assessment and when feasible. In addition, facilities undergoing modernization should, as a minimum, assess the potential for progressive collapse as the result of an abnormal loading event. The flowchart, shown in Figure 5.10, outlines the process for assessing the potential for progressive collapse in existing facilities. Findings of this analysis should be incorporated into the project-specific risk assessment, and shall be documented in accordance with the provisions in Section 1.5. The 'analysis' provisions contained in Section 5.1.2 concerning analysis techniques, procedure, analysis considerations and loading criteria, analysis criteria, material properties, and modeling guidance, shall also apply to existing construction.

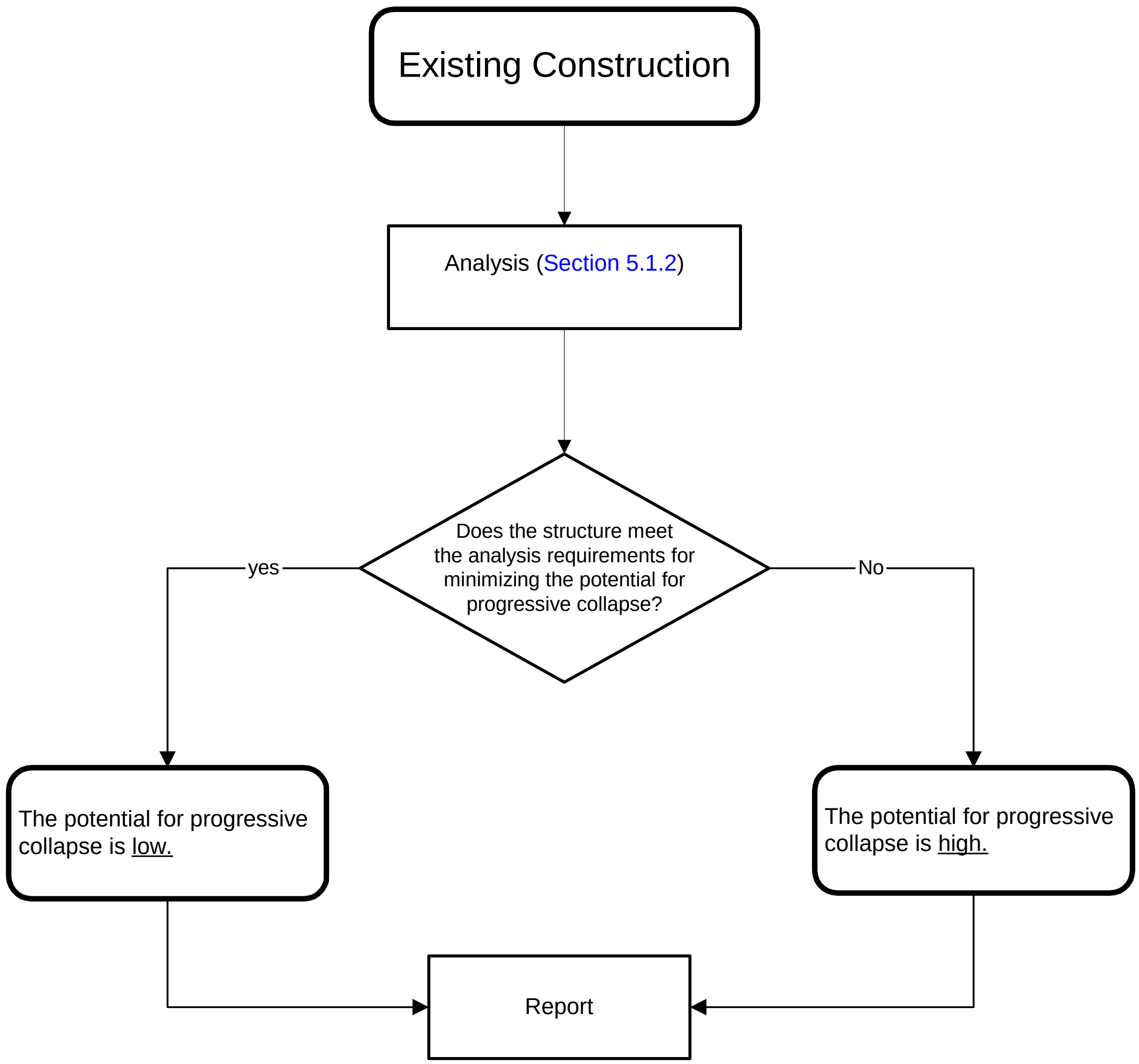


Figure 5.10. Process for assessing the potential for progressive collapse in existing construction.

Section 6. Resources

1. Allen, D.E. and Schriever, W.R. “Progressive Collapse, Abnormal Loads, and Building Codes.” Division of Building Research, National Research Council, 1972.

2. Architectural and Engineering Design Guidelines and Criteria for New Embassy Buildings. Office of Foreign Building Operations, Department of State, Washington, D.C. March 1990.

3. Army TM5-855-1, Air Force AFPAM 32-1147 (I), Navy NAVFAC P-1080, and DSWA DAHSCWEMAN-97, *Design and Analysis of Hardened Structures for Conventional Weapons Effects.* August 1998.

4. ASCE 7-95. American Society of Civil Engineers Minimum Design Loads for Buildings and Other Structures. Published by the American Society of Civil Engineers, New York.

5. Breen, J.E. and Siess, C.P. “Progressive Collapse-Symposium Summary.” ACI Journal, September 1979, pp. 997-1004.

6. Breen, J. “Research Workshop on Progressive Collapse of Building Structures, Held at the University of Texas at Austin, November 18-20, 1975.” National Bureau of Standards, Washington, D.C., November 1975.

7. The Building Regulations. Published by the Stationery Office, England, 1992.

8. Burnett, E. F. P. “The Avoidance of Progressive Collapse: Regulatory Approaches to the Problem.” National Bureau of Standards, Washington, D.C., October 1975.

9. Chapman, R.E. and Colwell, P.F. “Economics of Protection Against Progressive Collapse.” NBSIR 74-542, National Bureau of Standards, Washington, D.C, September 1974.

10. Corley, W.G. “Reducing Collateral Damage From Malevolent Explosions: Things Learned from the Oklahoma City Bombing.” Paper from The Structural Engineers World Congress, July 1998.

11. Crawford, J., Houghton, D., Dunn, B., and Karns, J. “Design Studies to the Vulnerability of Office Buildings to Progressive Collapse due to Terrorist Attack,” Research Report TR-01-10.1, Karagozian & Case Structural Engineers, Burbank, CA, 2001.

12. Department of Defense Interim Antiterrorism/Force Protection Construction Standards – Guidance on Structural Requirements (Draft). Department of Defense, March 5, 2001.

13. Desai, S. “Provision in Building Against Disproportionate Collapse.” Paper from The Institution of Civil Engineers and the Institution of Structural Engineers Half Day Meeting, March 1997.

14. “Fortskridanda Ras (Design to Avoid Progressive Collapse).” The National Swedish Board of Urban Planning, Publication No. 63, July 1, 1973.

15. Dusenberry, D., Kelly, D., Aquino, W., Mudlock, M., "Progressive Collapse Analysis of the Moorhead Building Using GSA Guidelines," Simpson, Gumpertz & Heger, Inc., Waltham, Massachusetts, March 12, 2003.

16. Ellingwood, Bruce and Leyendecker, E.V. “Approaches for Design Against Progressive Collapse.” Journal of the Structural Division, ASCE, 104(3), 1978, pp. 413-423.

17. Ellingwood, B. R. and Leyendecker, E. V. “Design Methods for Reducing the Risk of Progressive Collapse in Buildings.” National Bureau of Standards, Washington, D.C., April 1977.

18. Ellingwood, B., Leyendecker, E. and Yao, T. “Probability of Failure from Abnormal Load.” The Journal of Structural Engineering, Volume 109, Number 4, April 1983.

19. Ellingwood, B. and Longinow, A. “The Impact of the Ronan Point Collapse 25 Years After.” Paper from The Structural Engineers World Congress, July 1998.

20. Ettouney, M. and DiMaggio, P. “Integrated Study of Progressive Collapse of Buildings." Paper from Structural Engineers World Congress, July 1998.

21. “FEMA 273 – NEHRP Guidelines for the Seismic Rehabilitation of Buildings.” Federal Emergency Management Agency, October 1997.

22. “FEMA 274 – NEHRP Commentary on Guidelines for the Seismic Rehabilitation of Buildings.” Federal Emergency Management Agency, 1997.

23. “FEMA 350 - Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings.” Federal Emergency Management Agency, 2000.

24. “FEMA 351 - Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Frame Buildings.” Federal Emergency Management Agency, 2000.

25. “FEMA 355D – State of the Art Report on Connection Performance.” Federal Emergency Management Agency, 2000.

26. “FEMA 356 – Prestandard and Commentary for the Seismic Rehabilitation of Buildings.” Federal Emergency Management Agency, November 2000.

27. Ferahian, R. "Buildings: Design for Prevention of Progressive Collapse." Civil Engineering, Volume 42, Number 2, February 1972.

28. Fuller, G.R. "Seismic Resistance vs Progressive Collapse of Precast Concrete Panel Buildings." Proceedings of the Workshop on Earthquake-Resistant Reinforced Concrete Building Construction, Volume 3: technical papers; Berkeley, July 11-15, 1977. University of California, Berkeley, 1978.

29. GSA Security Criteria (Draft Revision, October 8, 1997). Building Technologies Division Office of Property Development Public Building Service General Services Administration.

30. Hawkins, N. and Mitchell, D. "Progressive Collapse of Flat Plate Structures," ACI Journal, Proceedings V.76, No. 7, July 1979, pp. 775-808.

31. Hinman, E.E. and Hammond, D.J. "Lessons From The Oklahoma City Bombing, Defensive Design Techniques." ASCE Press, 1997.

32. Houghton, D., Karns, J., Bryson, M., Kim, J., "Progressive Collapse Analysis and Design Guidelines Case Study: William S. Moorhead Federal Building Progressive Collapse Investigation and Upgrade Design Report," Research Report MHP 02328-00-1.r01, Myers, Houghton & Partners, Inc., Long Beach, California, January 13, 2003.

33. "The Implications of the Report of the Inquiry into the Collapse of Flats at Ronan Point, Canning Town." The Structural Engineer, Volume 47, Number 7, July 1969.

34. ISC Security Criteria for New Federal Office Buildings and Major Modernization Projects, The Interagency Security Committee, May 28, 2001.

35. Leyendecker, E.V., Breen, J.E., Somes, N.F., and Swatta, M. "Abnormal Loading on Buildings and Progressive Collapse." National Bureau of Standards, Washington, D.C., January 1976.

36. Leyendecker, E. V. "Investigation of the Skyline Plaza Collapse in Fairfax County, Virginia." National Bureau of Standards, Washington, D.C., February 1977

37. Matthews, S. "Robustness – Appraisal of existing large panel systems (LPS) buildings." Paper from The Institution of Civil Engineers and the Institution of Structural Engineers Half Day Meeting, March 1997.

38. McGuire, W., and Leyendecker, E.V. "Analysis of Non-reinforced Masonry Building Response to Abnormal Loading and Resistance to Progressive Collapse." NBSIR 74-526, National Bureau of Standards, Washington, D.C., November 1974.

39. Menzies, J. "Robustness – the SCOSS Report." Paper from The Institution of Civil Engineers and the Institution of Structural Engineers Half Day Meeting, March 1997.

40. Menzies, J.B. and Nethercot, D.A. "Progressive Collapse-Preventative Measures in the United Kingdom." Paper from Structural Engineers World Congress, July 1998.

41. Murtha-Smith, E. "Progressive Collapse Loads for Flat-Roof Structures." Journal of Structural Engineering, June 1995.

42. Narayanan, R. "Robustness – trends in practice- dwellings and small buildings." Paper from The Institution of Civil Engineers and the Institution of Structural Engineers Half Day Meeting, March 1997.

43. National Building Code of Canada. Issued by the Canadian Commission of Building and Fire Codes, National Research Council of Canada 1995.

44. "The Oklahoma City Bombing-Improving Building Performance Through Multi-Hazard Mitigation." Federal Emergency Management Agency Mitigation Directorate, August 1996.

45. Progressive Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects. U.S. General Services Administration and Applied Research Associates, Inc. November 2000.

46. "Protecting Buildings from Bomb Damage." National Research Council. Executive Summary only.

47. Rasbash, D and Stretch, K. "Explosions in Domestic Structures." The Structural Engineer, Volume 47, Number 19, October 1969.

48. Smith, J., Brokaw, J., Herrle, K., "Progressive Collapse Analysis of the Las Vegas Federal Building and U.S. Courthouse." Applied Research Associates, Inc., Vicksburg, MS., May 2003.

49. Springfield, J. "Resistance to Progressive Collapse Requirements in Ontario, Canada." Paper from the Structural Engineers World Congress, July 1998.

50. Standards of Seismic Safety of Existing Federally Owned or Leased Buildings and Commentary (ICSSC RP4/NISTIR 5382). Building and Fire Research Laboratory, National Institute of Standards and Technology, Gaithersburg, MD. February 1994.

51. ***STANDGARD*** User's Guide. Applied Research Associates, Inc. March 2003.

52. Structural Engineers Association of California (2001). Commentary and Recommendations on FEMA 350, prepared by SEAOC Seismology Committee, FEMA 350 Task Group.

53. Unified Facilities Criteria (UFC) DoD Minimum Antiterrorism Standards for Buildings (UFC4-010.01). Department of Defense, 2002.

54. Yokel, F.Y., Pielert, J. H. and Schwab, A. R. "The Implementation of a Provision Against Progressive Collapse." NBSIR 75-715, National Bureau of Standards, Washington, D.C. 20234, August 1975.

Appendix A. Atypical Structural Configurations

Because all structures are unique and are often not *typical* (i.e., buildings often contain distinguishing features or details), developing a set of analysis considerations that works for every facility is impractical. Thus, the user of this guideline must use engineering judgment to determine critical analysis scenarios that should be assessed in order to meet the intent of this guideline. Possible structural configurations that may result in an atypical structural arrangement include but are not limited to the following items.

Combination Structures

For facilities that utilize a combination of frame and wall systems for the primary supporting structure the analyst shall apply considerations similar to that presented for typical building configurations. The user shall use engineering judgment to determine the critical situations that should be assessed for the potential for progressive collapse. The considerations may be similar to those utilized in typical building configurations, but additional configurations may be necessary depending on the structural makeup. The user may consider, but not be limited to the other atypical arrangements that follow, for determining the critical scenarios that should be assessed.

Vertical Discontinuities

Structures that have vertical discontinuities may warrant additional consideration for progressive collapse. Examples of vertical discontinuities include discontinuous shear walls or columns such as the use of transfer girders (Figure A.1). If vertical discontinuities are present in the primary structural configuration, analyses of the response of the building for a loss of primary vertical support in these areas shall be considered.

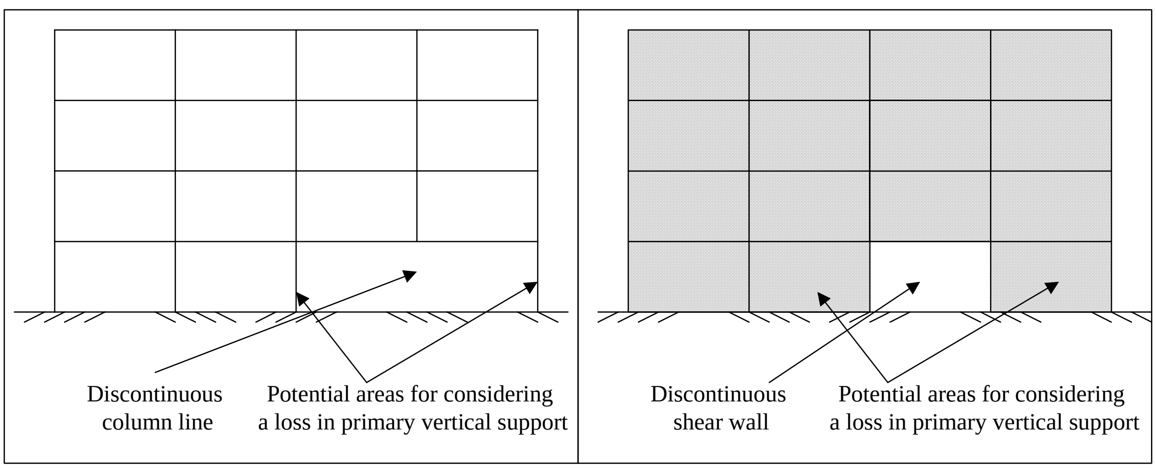


Figure A.1. Examples of vertical discontinuities.

Variations in Bay Size/Extreme Bay Sizes

A building configuration that contains structural bay(s) that have a large variance in size (compared to what may be considered a typical bay size of the facility) or extremely large bay sizes should be considered vulnerable and an assessment of the potential for progressive collapse shall be performed in these areas (Figure A.2). Structural bays that are greater than 30 ft in any direction are considered extreme.

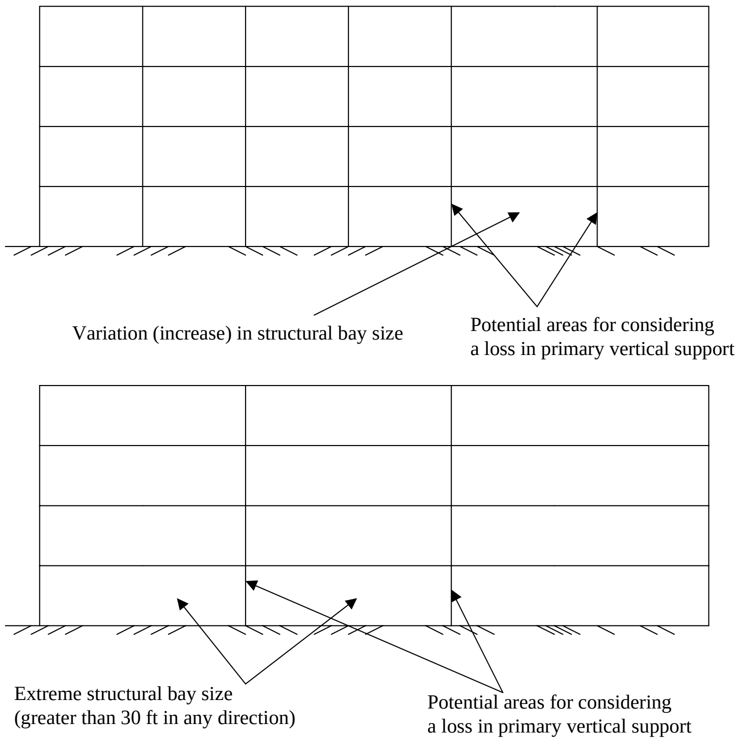


Figure A.2. Examples of buildings with substantial variation in bay size and extreme bay sizes.

Plan Irregularities

Plan irregularities such as re-entrant corners could present vulnerable areas in regards to the potential for progressive collapse. This type of structural configuration should be investigated regarding potential for progressive collapse. For example consider the hypothetical structure shown in Figure A.3. The removal of a primary support along the exterior of this structure could potentially collapse three structural bays from the ground floor level to the roof.

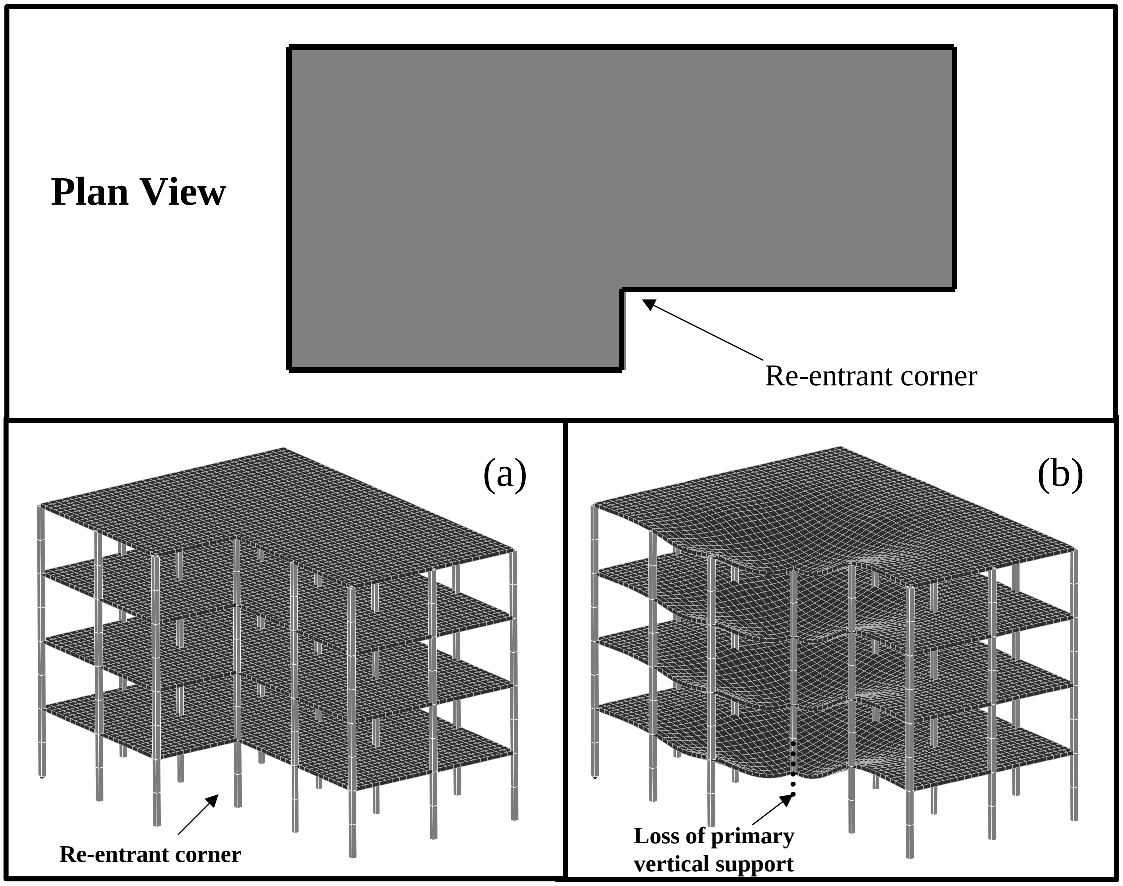


Figure A.3. (a) Example of a structure with a re-entrant corner. (b) The probable response of the structure for the case of a loss in primary vertical support in the re-entrant corner.

Closely Spaced Columns

Structures that have closely spaced columns (Figure A.4) may present uncertainty to the analyst when deciding on what primary vertical support to remove in the analysis process. Typically, some of the columns are likely to be architectural in nature as opposed to a true structural column. Structures that have this type of structural configuration shall be analyzed for a loss in support from both the architectural column as well as the structural column to assess the potential for progressive collapse. In the situation where structural columns are closely spaced, the structure should be analyzed for the loss of both columns if the distance between the columns is less than or equal to 30% of the longest dimension of the associated bay. Otherwise, only the loss of one column shall be required in the analysis.

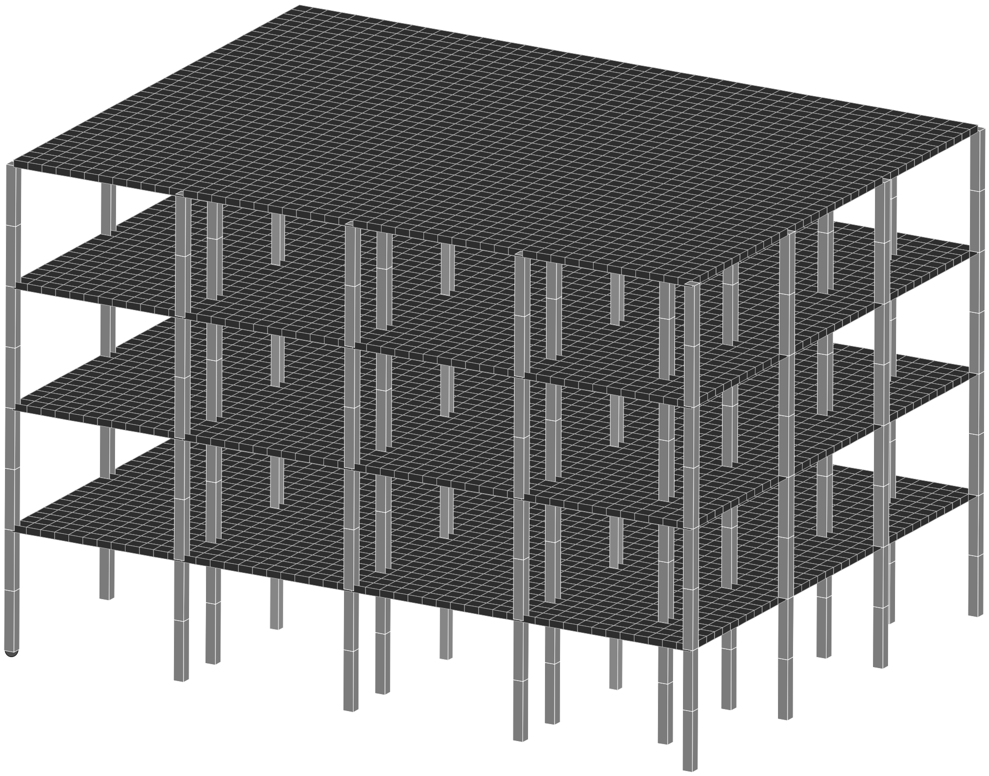


Figure A.4. Example of a building that has closely spaced columns.

Appendix B. Design Guidance

The design provisions outlined in this section *are not required* by this Guideline, but may be used to develop preliminary sizes and possibly enhance the initial structural design prior to assessing the potential for progressive collapse as outlined in Section 4.1.2 and Section 5.1.2. The procedures outlined in this Appendix shall be used to supplement the requirements of the project specific building code(s) in the design of both primary and secondary structural elements.

B.1 Foundation

The building foundation and foundation/structure connection should be designed such that for the case of an instantaneous removal of a primary vertical component (i.e., a column, wall section, etc.) these elements are capable of resisting the potential redistribution of forces. In order to enhance that possibility the following minimum design base shear procedure is presented for use in the design of the building foundation and foundation/structure connection.

Unfactored, ultimate capacities of the foundation elements may be used for this design provision.

Note:

If the base shear magnitude(s) determined in this section is less than the base shear value(s) determined for other load requirements (e.g., seismic, etc.) additional foundation design consideration based on the provisions outlined in this section are not necessary.

However, if the base shear load(s) determined in this section is larger than the base shear value(s) determined for other load requirements (e.g., seismic, specific blast design, etc.), additional foundation design considerations are recommended based on the magnitude(s) determined in this section in conjunction with using unfactored, ultimate load capacities in the design of the foundation elements. Hence, the design should be capable of meeting the greater of the requirements of this section as well as all other required building codes or specific blast design requirements.

Minimum Design Base Shear

The minimum design base shear values may be determined using the included program (an automated version of the Design Base Shear procedure) or by following the ensuing procedure. To begin the automated version of the Design Base Shear process, click on the 'Begin Minimum Design Base Shear Determination' button (at right).

Begin Minimum Design Base Shear Determination

Procedure

The following procedure shall be used to determine the specific minimum base shear requirements for each major direction of the building (Figure B.2).

Step 1. Using Figure B.3, determine L_n, where,

L_n* = the ranges from point X to points *n* (1 through 25 as depicted in Figure B.3) (ft)
SD = minimum defended standoff distance (ft)
H = total height of building (ft)
W = width of considered face (ft)

*L_n should be rounded down to the nearest foot

Step 2. Using Table B.1, select the Λ_n values consistent with the ranges (L_n) determined in Step 1.

Step 3. Calculate the average Λ value, Λ_{ave}, using equation B.1.

$$\Lambda_{ave} = \frac{\sum_{n=1}^{25} \Lambda_n}{25} \quad \text{(B.1)}$$

Step 4. Calculate γ, using equation B.2.

$$\gamma = 144 \Lambda_{ave} WH \quad \text{(B.2)}$$

Step 5. Determine the required base shear value for resisting abnormal loads using equation B.3.

$$V_b = \frac{\pi \gamma}{2{,}000T} \quad \text{(lb)} \qquad \text{(B.3)}$$

where,

$$T = C_t (H)^{0.75}$$

and

C_t = 0.035 for steel moment-resisting frames
C_t = 0.030 for reinforced concrete moment-resisting frames and eccentric frames
C_t = 0.020 for all other construction types

An example of this process is shown in Appendix C.

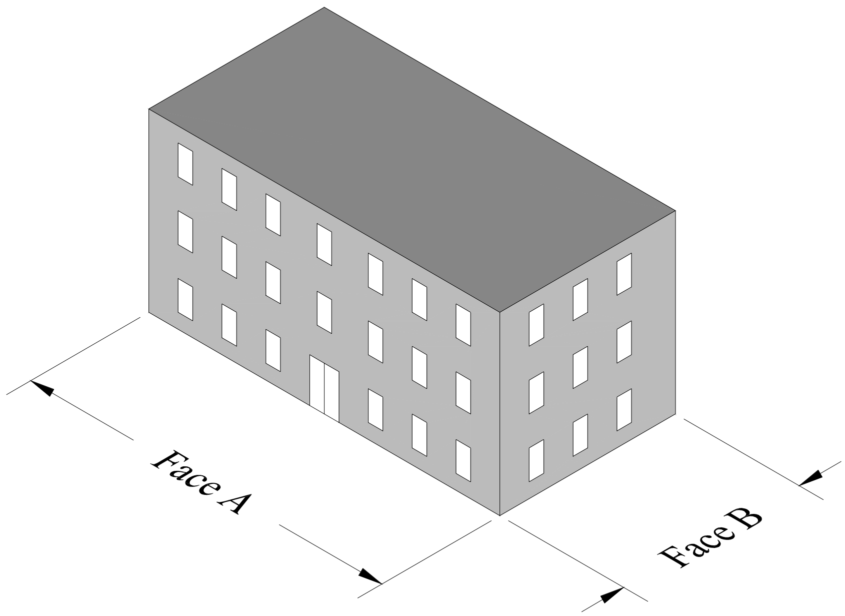


Figure B.2. Illustration depicting the two primary faces. Consideration of this process should be performed for both Face A and B.

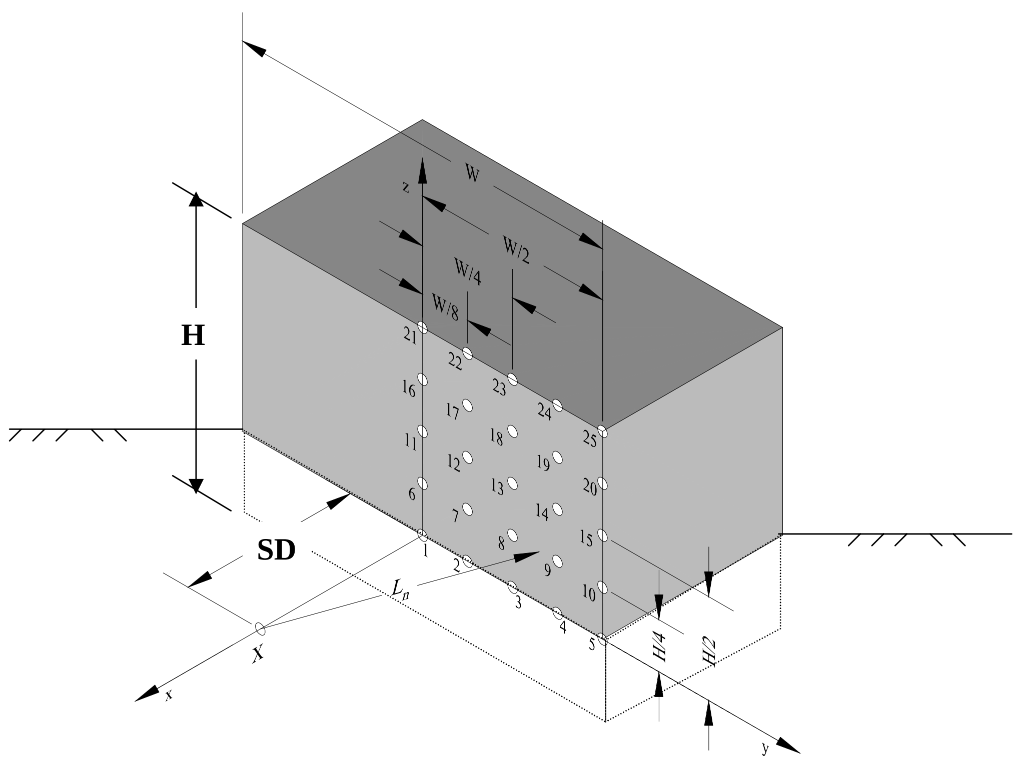


Figure B.3. Geometry parameters needed for calculating the design base shear value for Face A (similar for Face B).

Table B.1. Λ_n values for ranges 10 through 300 feet.

Range L_n **(feet)**	Λ
10	566.87
11	497.52
12	442.36
13	397.58
14	360.56
15	329.52
16	303.15
17	280.49
18	260.84
19	243.64
20	228.48
21	215.03
22	203.01
23	192.22
24	182.48
25	173.64
26	165.59
27	158.24
28	151.49
29	145.27
30	139.54
31	134.22
32	129.29
33	124.70
34	120.41
35	116.40
36	112.65
37	109.12
38	105.81
39	102.68
40	99.73
41	96.95
42	94.31
43	91.81
44	89.43
45	87.18
46	85.03
47	82.99
48	81.04
49	79.17
50	77.39
51	75.69
52	74.06
53	72.50
54	71.00
55	69.56
56	68.18
57	66.85
58	65.57
59	64.34
60	63.15
61	62.01
62	60.91

Range L_n **(feet)**	Λ
63	59.84
64	58.81
65	57.82
66	56.86
67	55.93
68	55.02
69	54.15
70	53.31
71	52.49
72	51.69
73	50.92
74	50.17
75	49.44
76	48.74
77	48.05
78	47.38
79	46.73
80	46.10
81	45.48
82	44.88
83	44.30
84	43.73
85	43.18
86	42.63
87	42.11
88	41.59
89	41.09
90	40.60
91	40.12
92	39.65
93	39.19
94	38.75
95	38.31
96	37.88
97	37.46
98	37.06
99	36.66
100	36.26
101	35.88
102	35.51
103	35.14
104	34.78
105	34.42
106	34.08
107	33.74
108	33.41
109	33.08
110	32.76
111	32.45
112	32.14
113	31.84
114	31.54
115	31.25

Table B.1. Λ_n values for ranges 10 through 300 feet (continued).

Range L_n (feet)	Λ
116	30.96
117	30.68
118	30.41
119	30.14
120	29.87
121	29.61
122	29.35
123	29.10
124	28.85
125	28.61
126	28.37
127	28.13
128	27.90
129	27.67
130	27.45
131	27.23
132	27.01
133	26.79
134	26.58
135	26.38
136	26.17
137	25.97
138	25.77
139	25.58
140	25.38
141	25.20
142	25.01
143	24.82
144	24.64
145	24.46
146	24.29
147	24.11
148	23.94
149	23.77
150	23.61
151	23.44
152	23.28
153	23.12
154	22.96
155	22.81
156	22.66
157	22.50
158	22.35
159	22.21
160	22.06
161	21.92
162	21.78
163	21.64
164	21.50
165	21.36

Range L_n (feet)	Λ
166	21.23
167	21.09
168	20.96
169	20.83
170	20.70
171	20.58
172	20.45
173	20.33
174	20.20
175	20.08
176	19.96
177	19.85
178	19.73
179	19.61
180	19.50
181	19.39
182	19.28
183	19.17
184	19.06
185	18.95
186	18.84
187	18.74
188	18.63
189	18.53
190	18.43
191	18.33
192	18.23
193	18.13
194	18.03
195	17.93
196	17.84
197	17.74
198	17.65
199	17.56
200	17.47
201	17.38
202	17.29
203	17.20
204	17.11
205	17.02
206	16.94
207	16.85
208	16.77
209	16.68
210	16.60
211	16.52
212	16.43
213	16.35
214	16.27
215	16.20

Table B.1. Λ_n values for ranges 10 through 300 feet (continued).

Range L_n (feet)	Λ	Range L_n (feet)	Λ
216	16.12	258	13.39
217	16.04	259	13.33
218	15.96	260	13.28
219	15.89	261	13.23
220	15.81	262	13.17
221	15.74	263	13.12
222	15.66	264	13.07
223	15.59	265	13.02
224	15.52	266	12.97
225	15.44	267	12.92
226	15.37	268	12.87
227	15.30	269	12.82
228	15.23	270	12.77
229	15.16	271	12.72
230	15.09	272	12.67
231	15.03	273	12.62
232	14.96	274	12.57
233	14.89	275	12.52
234	14.82	276	12.48
235	14.76	277	12.43
236	14.69	278	12.38
237	14.63	279	12.34
238	14.56	280	12.29
239	14.50	281	12.25
240	14.44	282	12.20
241	14.38	283	12.15
242	14.31	284	12.11
243	14.25	285	12.07
244	14.19	286	12.02
245	14.13	287	11.98
246	14.07	288	11.93
247	14.01	289	11.89
248	13.95	290	11.85
249	13.89	291	11.81
250	13.84	292	11.76
251	13.78	293	11.72
252	13.72	294	11.68
253	13.66	295	11.64
254	13.61	296	11.60
255	13.55	297	11.56
256	13.50	298	11.52
257	13.44	299	11.48
		300	11.44

B.2 Lateral Force Resisting System

The following procedures are design provisions for ensuring that the lateral force resisting system contains at least moderate resistance regarding laterally applied abnormal loads.

Column Sizing

The following procedure may be used for developing preliminary column sizes in structures that utilize columns as the primary lateral force resisting system. **Column parameters determined in this procedure should be used only if they exceed the sizes required by other load requirements.**

The typical, required column size for each floor level may be determined using the included program (an automated version of the Column Sizing procedure) or by following the ensuing procedure. To begin the automated version of the Column Sizing process, click on the 'Begin Column Sizing Determination' button (at right).

Procedure

The following procedure shall be performed in the directions of both major axes as shown in Figure B.4.

Step 1. From Figure B.5, determine values of L_n, where,

n	=	*story level*
L_n^*	=	the ranges from point "X" to the mid-height of each story level Figure B.5 (ft)
SD	=	minimum defended standoff distance along the building face under consideration (ft)

*****L_n should be rounded down to the nearest foot

Step 2. Using Table B.1, select the Λ_n values consistent with the ranges, L_n, determined in Step 1.

The remaining steps should be performed independently for each story level (n).

Step 3. Calculate Λ_{sn} using equation B.4

$$\Lambda_{s_n} = \frac{w_b}{25}\Lambda_n \tag{B.4}$$

where,

w_b = typical bay width (ft)

Step 4. Determine the required bent story stiffness (K_{1n}) using equation B.5.

$$K_{1_n} = 15.952\Lambda_{s_n}^{2.0} \quad \text{(lb/in) for steel frames} \tag{B.5.a}$$

$$K_{1_n} = 16.5595\Lambda_{s_n}^{2.2} \quad \text{(lb/in) for r/c frames or flat slab structures} \tag{B.5.b}$$

Step 5. Calculate the adjusted bent story stiffness (K_{2n}) using equation B.6,

$$K_{2_n} = \frac{w_1}{w_{2n}} K_{1_n} \tag{B.6}$$

where,

w_1 = Unit weight assumed for equation B.5 derivation = 100 psf
w_{2n} = Adjusted unit weight for the bent and story under consideration (psf)

where,

$$w_{2_n} = w_{an} \frac{TA}{1{,}250} \quad \text{(psf)}$$

where,

w_{an} = actual unit weight for the bent and story under consideration (psf)
TA = Tributary plan area of bent (ft^2)

as shown in Figure B.4.

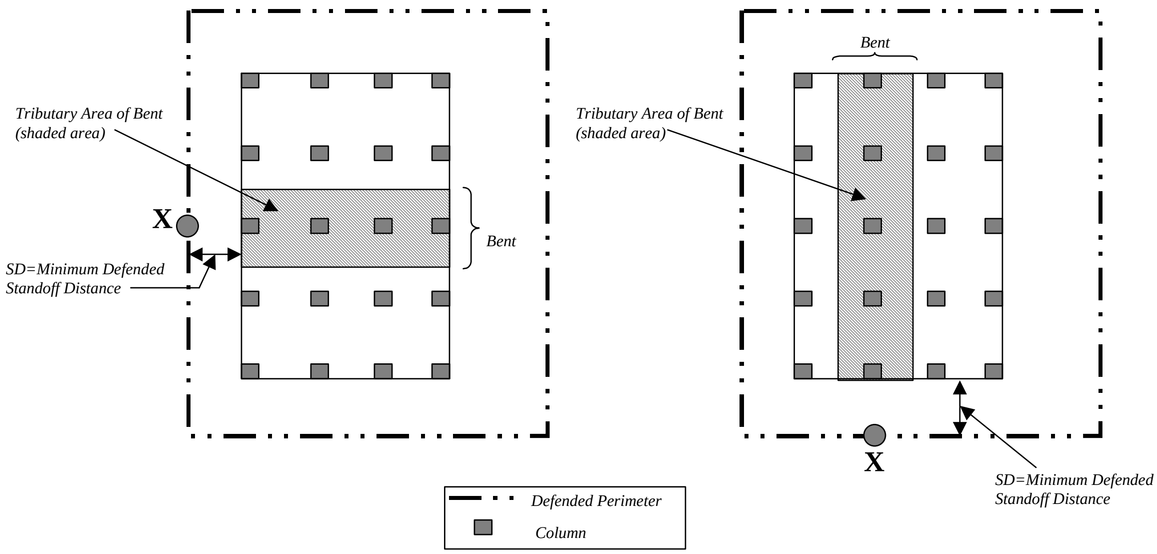


Figure B.4. Illustration of a facility being considered for column sizing. The bent story stiffness shall be evaluated for both major axes.

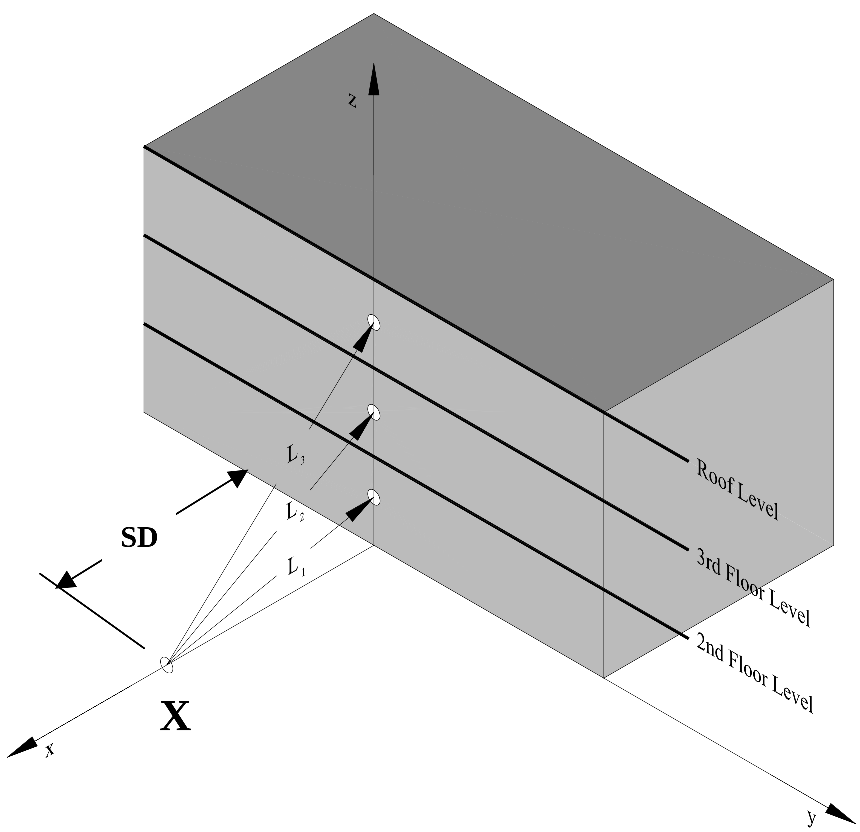


Figure B.5. Distance parameters needed for determining Λ_n.

Step 6. Once the adjusted bent story stiffness has been calculated, the required column stiffness can be determined by using equation B.7.

$$K_{col_n} = K_{2n} / N \tag{B.7}$$

where,

N = number of columns in bent

Step 7. The required moment of inertia for each column can then be calculated using equation B.8.

$$I_{col_n} = \frac{K_{col\,n} H_n^{\ 3}}{12E} \quad (\text{in}^4) \tag{B.8}$$

where,

H_n = Story height (inches)
E = modulus of elasticity (psi)

For reinforced concrete, E can be determined as:

$$E = w_c^{1.5} 33 \sqrt{f_c^{'}}$$

where,

w_c = concrete unit weight (pcf)
f'_c = concrete strength (psi)

Using the moment of inertia value calculated in equation B.8, a steel manual (e.g., AISC) can be referenced to select an appropriately sized steel column.

Using the moment of inertia value, I_{coln} calculated in equation B.8, the following equation can be used for sizing reinforced concrete columns.

$$I_{col\,n} = \frac{bd^3}{2}\left[5.5\rho + 0.083\right] \quad (\text{in}^4) \tag{B.9}$$

where,

b = column width (in)
d = column effective depth (in)
ρ = positive (and negative) reinforcing ratio

An example of this process is shown in Appendix C.

Equivalent Lateral Force Procedure

The base shear value(s) determined in B.1 shall be applied to the lateral force resisting system using a well-established 'Equivalent Lateral Force Procedure' such as the approaches outlined in the following references:

- NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures, 1997 Edition (FEMA 302). Issued by Federal Emergency Management Agency, 1997.
- 1997 Uniform Building Code, Volume 2 Structural Engineering Design Provisions.

The lateral force resisting system must be capable of resisting the applied transverse loading to allowable limits as defined by the utilized procedure.

Lateral force resisting system parameters determined in this procedure shall be used only if they exceed the parameters required by other load requirements.

Note:

Performance of this design provision is not necessary if all of the following items have been satisfied:

- *An Equivalent Lateral Force Procedure has been previously performed for the project specific seismic base shear value(s).*
- *The lateral force resisting system is capable of resisting the transverse loading associated with the seismic base shear value(s) to within allowable limits as defined by the utilized procedure.*
- *The seismic base shear value(s) are larger than the base shear value(s) determined in B.1.*

Design to Resist Column Buckling

The columns along the perimeter of the facility, between the 1st floor above grade and 3rd floor above grade should be designed using acceptable design techniques to resist buckling for an additional story of unsupported length (Figure B.6) when subjected to the vertical load requirement defined in equation B.10.

In addition, facilities having uncontrolled parking areas or public areas require column designs to resist potential buckling. Specifically, all columns in uncontrolled parking areas or public areas should be designed using acceptable design techniques to resist buckling for an additional story of unsupported length (Figure B.7) when subjected to the vertical load requirement defined in equation B.10.

The load requirement for this design provision consists of the vertical loading presented in equation 4.10 applied at each floor level.

$$\text{Load} = 2(\text{DL} + 0.25\text{LL}) \tag{B.10}$$

where,

DL = dead load
LL = live load

<u>Unfactored, ultimate load capacities of the columns may be used for this design consideration.</u>

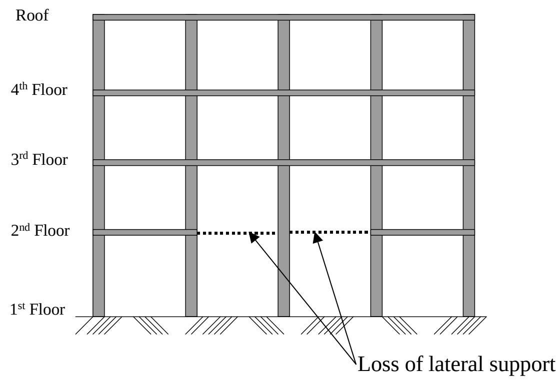


Figure B.6. Perimeter consideration for column buckling.

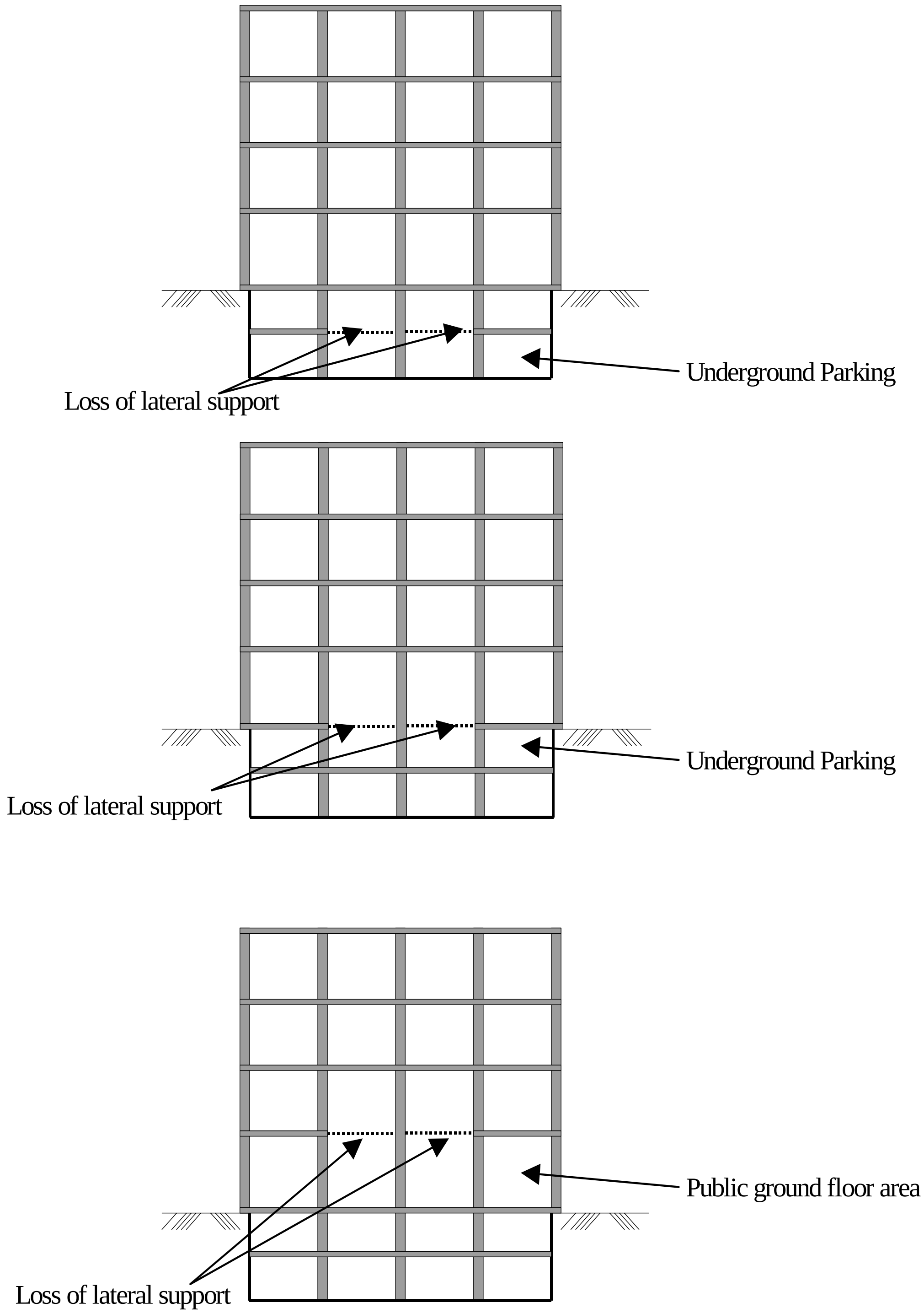


Figure B.7. Uncontrolled parking areas or public area considerations for column buckling.

B.3 Design to Resist Load Reversals

The primary and secondary structural elements should be designed with acceptable design techniques to effectively resist load reversals in the following locations:

<u>Facilities that have uncontrolled parking areas or public areas:</u>

> At least one structural bay deep around the perimeter of the structure from the ground floor to the roof level and for all interior structural bays for at least three floors above grade

or

<u>Facilities that do not have uncontrolled parking areas or public areas:</u>

> At least one structural bay deep around the perimeter of the structure from the ground floor to the roof level.

Horizontally oriented (i.e., roof beams, girders, etc.) primary structural elements shall be designed to resist the vertical load requirement given in equation B.10 <u>applied in the (1) downward direction and (2) upward direction</u>.

Vertically oriented, primary structural elements shall be designed to resist loading reversals associated with a reverse in the transverse loadings determined in B.1 – B.2.

<u>Unfactored, ultimate capacities of the structural elements may be used for this design provision</u>.

Note:

An example of designing horizontally oriented elements for load reversals would include the consideration of a primary girder. The load determination for designing this component includes the dead load associated with (1) the weight of the girder and (2) the load associated with the weight of the slab that the girder supports and the project specific live load requirements. These loads shall be utilized in conjunction with Equation B.10 for determining the applied load requirements that shall be used in the design of the girder to resist load reversals. The structural components should be designed to resist this loading when applied in a downward direction and also when applied in an upward direction. For example, in order to achieve this design requirement, symmetric reinforcement (which will increase the ultimate load capacity of the element) for reinforced concrete construction or moment resisting connections may be necessary.

B.4 Design to Resist Shear Failure

For the applied loading given in equation B.10, primary and secondary structural elements should be designed to resist shear failure in the following locations:

Facilities that have uncontrolled parking areas or public areas:

> At least one structural bay deep around the perimeter of the structure from the ground floor to the roof level and for all interior structural bays for at least three floors above grade

or

Facilities that do not have uncontrolled parking areas or public areas:

> At least one structural bay deep around the perimeter of the structure from the ground floor to the roof level.

Unfactored, ultimate load capacities of the primary structural elements may be used for this design provision.

Note:

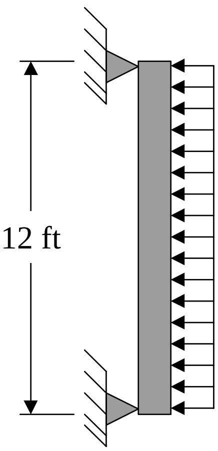
It is essential that the primary structural elements maintain sufficient strength and ductility under an abnormal loading event to preclude a shear failure. When the shear capacity is reached before the flexural capacity, the possibility of a sudden, non-ductile failure of the element exists which may lead to a progressive collapse of the structure.

Appendix C. Example Calculations

C.1 Nonfrangible/Frangible Façade Examples

Example 1

Consider a reinforced concrete (r/c) wall that spans 12 feet between floor levels and acts as a one-way slab system (i.e., the wall imparts transverse load to the floor levels). The r/c wall does not contain windows and has the following properties:



$$f_c^{'} = 4{,}000\,psi$$
$$f_y = 60{,}000\,psi$$
$$\rho = 0.0025$$

Treat as a simply supported, one-way slab subjected to a uniformly distributed load where:

d = 4.5 in (6 in slab with 1.5 in cover)
L = 12 ft or 144 in

The flexural capacity of a simply supported beam or one way slab can be determined as:

$$Capacity = \frac{8M_u}{L^2} \qquad \text{(psi)}$$

where,

$$M_u = \rho b d^2 f_y \left(1 - \frac{\rho f_y}{1.7 f_c^{'}}\right) = \text{(ultimate bending moment)}$$

and

$b = 1$ (considering a unit width of the wall)

Hence,

$$M_u = 2970 \frac{\text{lb - in}}{\text{in}}$$
$$Capacity = 1.15\,psi > 1.0 \text{ psi}$$

Hence, the façade system should be considered 'non-frangible'.

Example 2

Consider a combination façade system such as that shown in Figure C.1. This façade consists of a window system and a metal panel/CMU infill wall system.

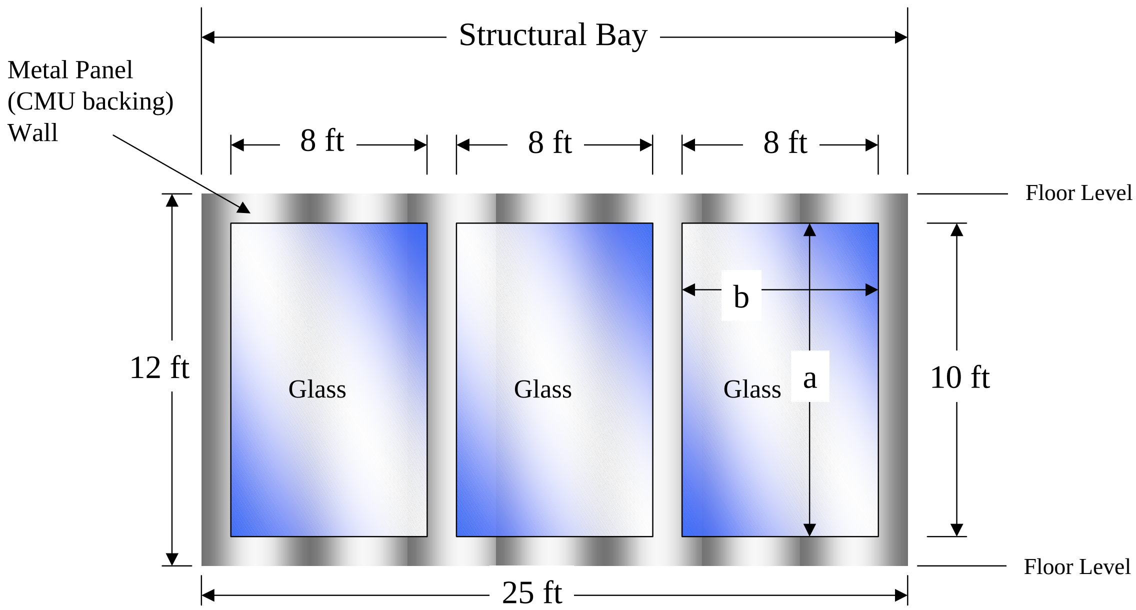


Figure C.1. Combination façade system.

First, determine the percent of wall occupied by each façade system.

Structural Bay Area = 12 ft x 25 ft)	= 300 ft^2	
Window = 3(8'x10')	= 240 ft^2	(80%)
Metal Panel/CMU Wall = 300 ft^2 - 240 ft^2	= 60 ft^2	(20%)

Only consideration of the window capacity for determining whether the façade system is frangible or non-frangible is required since the Metal Panel/CMU Wall consists of less than 25% of the wall area per structural bay.

The window openings are 8 ft (b) by 10 ft (a) and are capable of achieving two-way action. The glass consists of a 3/8 inch thick monolithic annealed pane with the following maximum yield strength:

$$f_y = 12{,}750\, psi$$

The glass will be analyzed as simply supported on all four edges and subjected to a uniformly distributed load.

The flexural capacity of a simply supported two-way slab can be determined as:

$$Capacity = \frac{1}{2}\left(\frac{22.3M_u}{a^2} + \frac{22.3M_u}{ab}\right) \qquad \text{(psi)}$$

where,

$$M_u = f_y S = \text{(ultimate bending moment)}$$

and

S = I/c = 0.0235 (in^3/in)

and

$I = bt^3/12$ = 0.0044 (in^4/in

and

b = 1 (assuming unit width)
t = glass thickness = 0.375

Hence,

$$M_u = 299.6\,\frac{\text{lb - in}}{\text{in}}$$

$$Capacity = 0.65\,psi < 1.0 \text{ psi}$$

Hence, the façade system should be considered 'frangible'.

Example 3

Consider a combination façade system such as that shown in Figure C.2. This façade consists of a window system and a metal panel/CMU infill wall system.

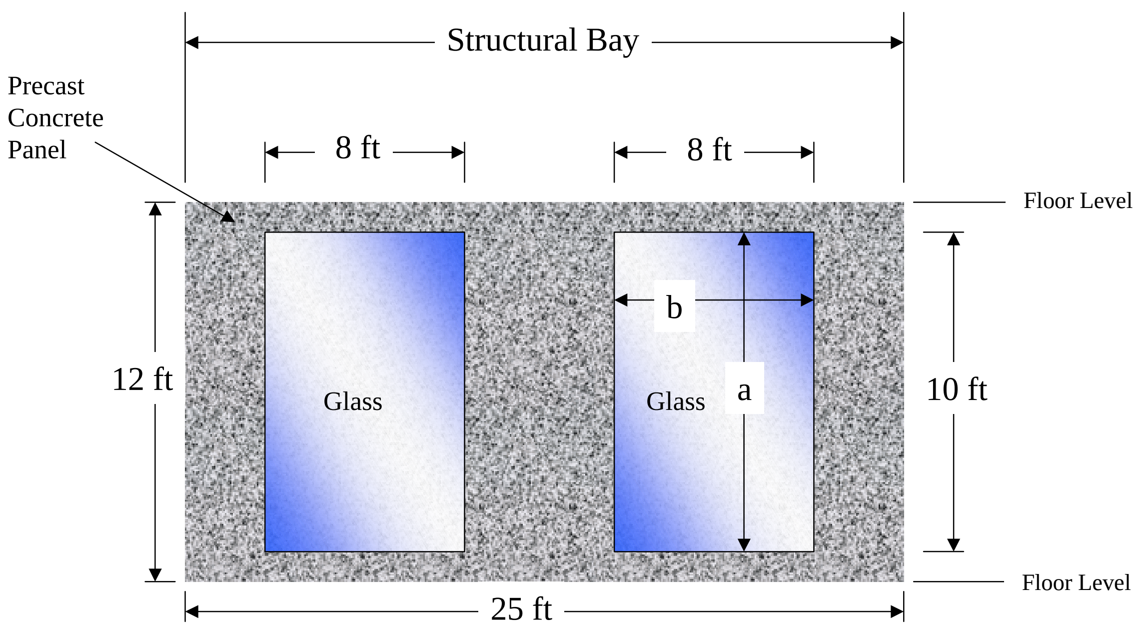


Figure C.2. Combination façade system.

First, determine the percent of wall occupied by each façade system.

Structural Bay Area = 12 ft x 25 ft	= 300 ft^2	
Window = 2(8'x10')	= 160 ft^2	(53.3%)
Metal Panel/CMU Wall = 300 ft^2 - 240 ft^2	= 140 ft^2	(46.7%)

Consideration of both the window capacity and wall capacity is required for determining whether the façade system is frangible or non-frangible.

The window openings are 8 ft (b) by 10 ft (a) and are capable of achieving two-way action. The glass consists of a 3/8 inch thick monolithic annealed pane with the following maximum yield strength:

$$f_y = 12{,}750\,psi$$

Thus the capacity of the windows are:

$Capacity = 0.65\,psi$ < 1.0 psi (See Example 2 for window capacity determination)

This part of the façade system should be considered 'frangible'.

Now consider the precast concrete panel portion of the façade. Assume this façade system has properties similar to the properties given in Example 1. Thus, the capacity of the precast concrete wall is:

$Capacity = 1.15\,psi$ > 1.0 psi (See Example 1 for precast concrete wall capacity determination)

This part of the façade system should be considered 'non-frangible'.

Recall that the largest of the capacities determined dictate whether the façade system is considered frangible or non-frangible. Hence, the façade system assessed in this example should be considered 'non-frangible'.

C.2 Design Base Shear

The procedure to determine base shear requirements as described in Appendix B is illustrated by the following example:

Building parameters:

Construction Type		=	reinforced concrete or steel frame
Total building height,	H	=	50 ft.
Face A width (see Figure C.3),	W_a	=	150 ft.
Face B width (see Figure C.3),	W_b	=	75 ft.
Face A minimum standoff distance,	SD_a	=	80 ft.
Face B minimum standoff distance,	SD_b	=	100 ft.

<u>The base shear for Face A is calculated as</u>:

Step 1. Using Figure C.4, determine L_n, where,

L_n = the ranges from point X to points *n* (1 through 25 as depicted in Figure C.4) (rounded to the nearest foot)
SD = minimum defended standoff distance (ft)
H = total height of building (ft)
W = width of considered face (ft)

Step 2. Using Table B.1, select the Λ_n values consistent with the ranges (L_n) determined in the Step 1.

The calculated L_n and Λ_n values are summarized in Table C.1, resulting in a total Λ value

$$\Sigma\Lambda = 956.67$$

Step 3. Calculate the average Λ value, Λ_{ave}, using equation B.1.

$$\Lambda_{ave} = \frac{\sum_{n=1}^{25} \Lambda_n}{25} = 38.27$$

Step 4. Calculate γ, using equation B.2.

$$\gamma = 144\Lambda_{ave} WH = 41{,}331{,}600$$

Table C.1. Calculated Λ values for Face A.

	x ft	y ft	z ft	Ln (ft)	Λ
1	0	80	0	80	46.1
2	37.5	80	0	88	41.59
3	75	80	0	110	32.76
4	0	80	25	84	43.73
5	37.5	80	25	92	39.65
6	75	80	25	112	32.14
7	0	80	50	94	38.75
8	37.5	80	50	102	35.51
9	75	80	50	121	29.61
10	18.75	80	0	82	44.88
11	56.25	80	0	98	37.06
12	0	80	12.5	81	45.48
13	18.75	80	12.5	83	44.3
14	37.5	80	12.5	89	41.09
15	56.25	80	12.5	99	36.66
16	75	80	12.5	110	32.76
17	18.75	80	25	86	42.63
18	56.25	80	25	101	35.88
19	0	80	37.5	88	41.59
20	18.75	80	37.5	90	40.6
21	37.5	80	37.5	96	37.88
22	56.25	80	37.5	105	34.42
23	75	80	37.5	116	30.96
24	18.75	80	50	96	37.88
25	56.25	80	50	110	32.76
				ΣΛ =	956.67

Step 5. Determine the required base shear value for resisting abnormal loads using equation B.3.

$$V_b = \frac{\pi\gamma}{2{,}000T} \qquad \text{(lb)}$$

where,

$$T = C_t(H)^{0.75}$$

and

C_t = 0.035 for steel moment-resisting frames
C_t = 0.030 for reinforced concrete moment-resisting frames
and eccentric frames
C_t = 0.020 for all other construction types

For a reinforced concrete frame, C_t = 0.03, thus

$$T = C_t(H)^{0.75} \quad = \quad 0.564$$

and

$$V_b = \frac{\pi\gamma}{2{,}000T} \;=\; 115{,}113 \text{ (lb)} \cong 115 \text{ kip on Face A.}$$

If the frame had been steel (C_t = 0.035), then

$$T = C_t(H)^{0.75} \quad = \quad 0.658$$

and

$$V_b = \frac{\pi\gamma}{2{,}000T} = 98{,}668 \text{ (lb)} \cong 99 \text{ kip on Face A.}$$

<u>The base shear for Face B is calculated as</u>:

Step 1. Using Figure C.4, determine L_n, where,

L_n	=	the ranges from point X to points *n* (1 through 25 as depicted in Figure C.4) (rounded to the nearest foot)
SD	=	minimum defended standoff distance (ft)
H	=	total height of building (ft)
W	=	width of considered face (ft)

Step 2. Using Table B.1, select the Λ_n values consistent with the ranges (L_n) determined in the Step 1.

The calculated L_n and Λ_n values are summarized in Table C.2, resulting in a total Λ value

$$\Sigma\Lambda = 845.39$$

Step 3. Calculate the average Λ value, Λ_{ave}, using equation B.1.

$$\Lambda_{ave} = \frac{\sum_{n=1}^{25}\Lambda_n}{25} \quad = \; 33.8$$

Step 4. Calculate γ, using equation B.2.

$$\gamma = 144\Lambda_{ave}WH \; = 18{,}252{,}000$$

Table C.2. Calculated Λ values for Face B.

	x ft	y ft	z ft	Ln (ft)	Λ
1	0	100	0	100	36.26
2	18.75	100	0	102	35.51
3	37.5	100	0	107	33.74
4	0	100	25	103	35.14
5	18.75	100	25	105	34.42
6	37.5	100	25	110	32.76
7	0	100	50	112	32.14
8	18.75	100	50	113	31.84
9	37.5	100	50	118	30.41
10	9.375	100	0	100	36.26
11	28.125	100	0	104	34.78
12	0	100	12.5	101	35.88
13	9.375	100	12.5	101	35.88
14	18.75	100	12.5	103	35.14
15	28.125	100	12.5	105	34.42
16	37.5	100	12.5	108	33.41
17	9.375	100	25	104	34.78
18	28.125	100	25	107	33.74
19	0	100	37.5	107	33.74
20	9.375	100	37.5	107	33.74
21	18.75	100	37.5	108	33.41
22	28.125	100	37.5	110	32.76
23	37.5	100	37.5	113	31.84
24	9.375	100	50	112	32.14
25	28.125	100	50	115	31.25
				ΣΛ =	845. 39

Step 5. Determine the required base shear value for resisting abnormal loads using equation B.3.

$$V_b = \frac{\pi\gamma}{2{,}000T} \qquad \text{(lb)}$$

$$T = C_t(H)^{0.75} = 0.564 \text{ (as previously calculated)}$$

and

$$V_b = \frac{\pi\gamma}{2{,}000T} = 50{,}834 \text{ (lb)} \cong 51 \text{ kip on Face B.}$$

For a steel frame (C_t = 0.035 and T = 0.658), then

$$V_b = \frac{\pi\gamma}{2{,}000T} = 43{,}572 \text{ (lb)} \cong 44 \text{ kip on Face A.}$$

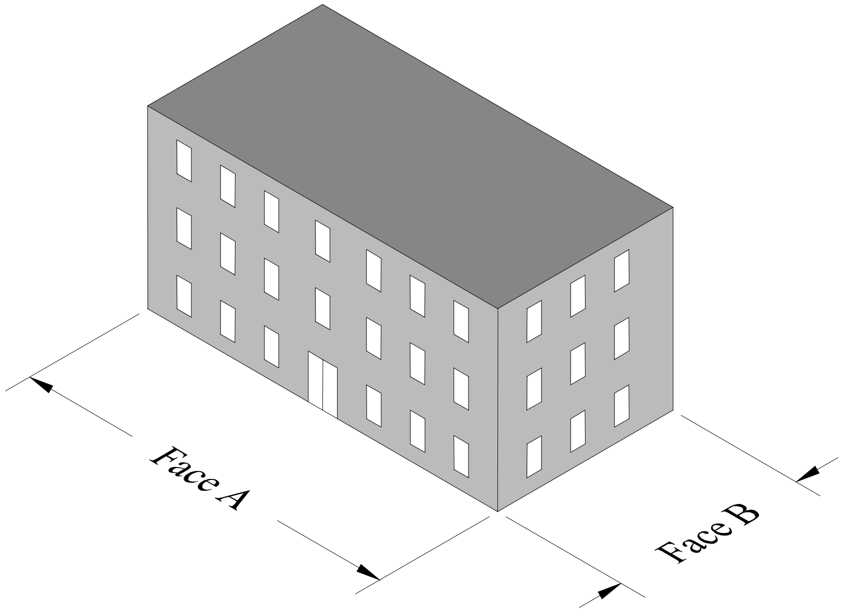


Figure C.3. Perform base shear calculation for both faces (A & B) of the building.

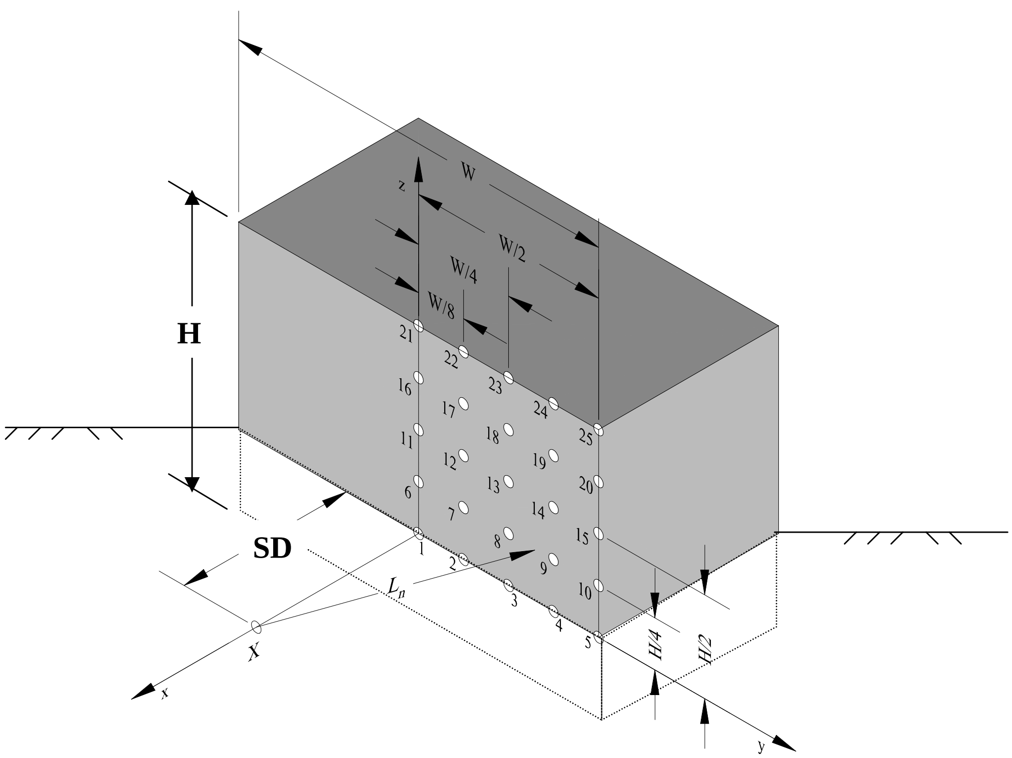


Figure C.4. Generalized geometry parameters for base shear calculation.

C.3 Lateral Force Resisting System - Column Sizing

The procedure to determine preliminary column sizes as described in Appendix B is illustrated by the following example:

Building parameters :

Construction Type		=	reinforced concrete, f'_c = 4000 psi
Total four-story building height,	H	=	50 ft.
	H_1	=	14 ft.
	H_2	=	12 ft.
	H_3	=	12 ft.
	H_4	=	12 ft.
Face A width,	W_a	=	150 ft.
Face B width,	W_b	=	75 ft.
Typical bay width, Face A	bw_a	=	30 ft.
Typical bay width, Face B	bw_b	=	15 ft.
Actual unit weight, all floors	w_{an}	=	70 psf
Face A minimum standoff distance,	SD_a	=	80 ft.
Face B minimum standoff distance,	SD_b	=	100 ft.

The procedure will be demonstrated for Load Level 2 in both major directions as shown in Figure C.5. For Face A:

Step 1. From Figure C.6, determine values of L_n, where,

n = *story level*

L_n = the ranges from point "X" to the mid-height of each story level (Figure C.6) (rounded to the nearest foot)

SD = minimum defended standoff distance along the building face under consideration (ft)

Step 2. Using Table B.1, select the Λ_n values consistent with the ranges, L_n, determined in Step 1.

The calculated L_n and Λ_n values for Face A are summarized in Table C.3.

Table C.3. Calculated L_n and Λ_n values for Face A.

Story Level	Story Height (ft)	Total Height (ft)	Range, L (ft)	Λ
1	14	14	80	46.1
2	12	26	82	44.88
3	12	38	86	42.63
4	12	50	91	40.12

The remaining steps should be performed independently for each story level (*n*). The example will demonstrate the steps for story level one.

Story Level 1:

Step 3. Calculate Λ_{sn} using equation B.4 (adjust for actual bay width)

$$\Lambda_{s_n} = \frac{w_b}{25}\Lambda_n \quad = \quad 30 * 46.1 / 25 \; = \; 55.32$$

where,

bw_a = typical bay width (ft) = 30 ft.

Step 4. Determine the required bent story stiffness (K_{1n}) using equation B.5b for reinforced concrete frames or flat slab structures:

$$K_{1_n} = 16.5595\Lambda_{s_n}^{2.2} \quad = \quad 109{,}157 \quad \text{(lb/in)}$$

Step 5. Calculate the *adjusted* bent story stiffness (K_{2n}) using equation B.6,

$$K_{2_n} = \frac{w_1}{w_{2n}} K_{1_n} \quad = \; 86{,}633 \text{ (lb/in)}$$

where

w_1 = Unit weight assumed for equation B.5 derivation = 100 psf
w_{2n} = Adjusted unit weight for story 1 = 126 psf

and

$$w_{2_n} = w_{an}\frac{TA}{1{,}250} \quad = \; 126 \text{ psf}$$

where,

w_{an} = actual unit weight = 70 psf
TA = Tributary plan area of bent = 30 ft x 75 ft = 2250 ft^2

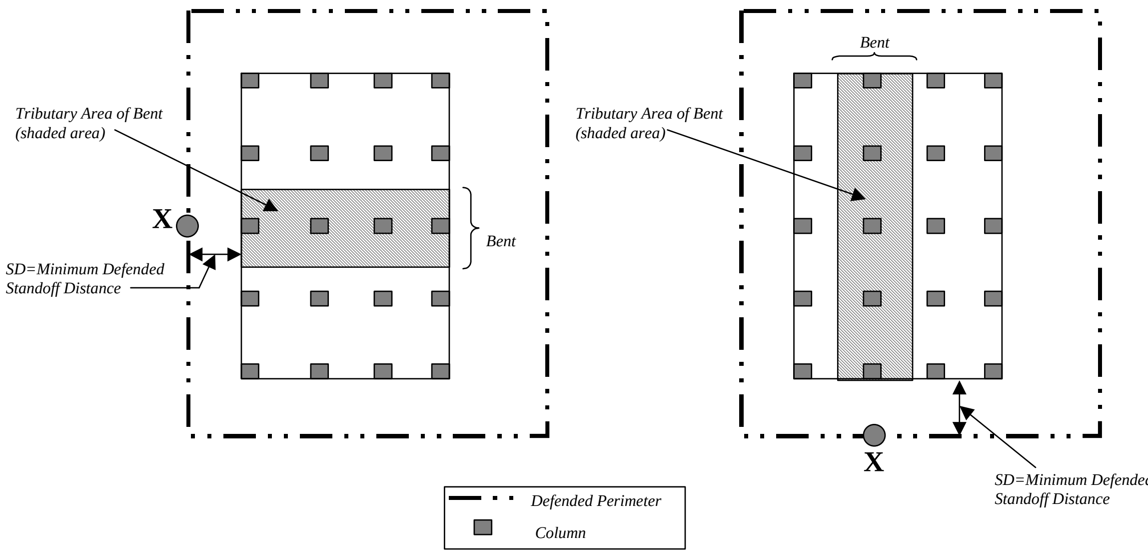


Figure C.5. Illustration of a facility being considered for column sizing. The bent story stiffness shall be evaluated for both major axes.

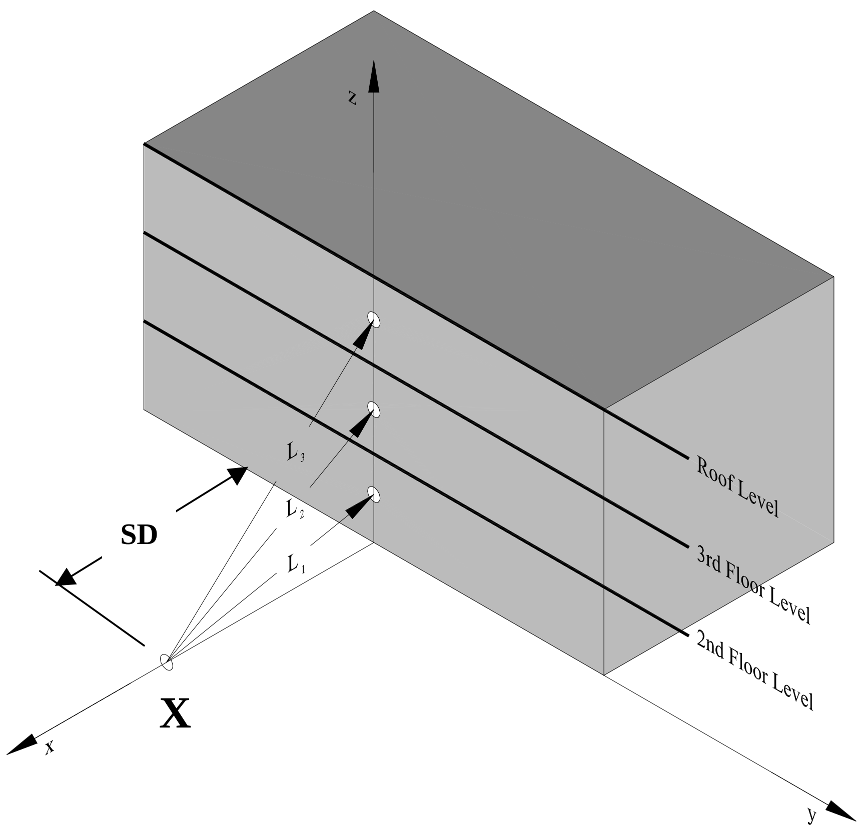


Figure C.6. Distance parameters needed for determining Λ_n.

Step 6. Once the adjusted bent story stiffness has been calculated, the required column stiffness can be determined by using equation B.7.

$$K_{col_n} = K_{2n} / N = 14{,}439$$

where,

N = number of columns in bent = 6 (75/15 + 1)

Step 7. The required moment of inertia for each column can then be calculated using equation B.8.

$$I_{col_n} = \frac{K_{col\,n} H_n^{\;3}}{12E} \quad (\text{in}^4)$$

where,

H_n = Story height = 168 inches
E = modulus of elasticity (psi)

For reinforced concrete, E can be determined as:

$$E = w_c^{1.5} 33 \sqrt{f_c^{'}} = 3{,}834{,}254 \text{ psi}$$

where,

w_c = concrete unit weight = 150 pcf
f'_c = concrete compressive strength = 4000 psi

Therefore,

$$I_{col_n} = \frac{K_{col\,n} H_n^{\;3}}{12E} = 1488 \text{ in}^4$$

Using this moment of inertia value, the following equation can be used for sizing reinforced concrete columns:

$$I_{col\,n} = \frac{bd^3}{2}\left[5.5\rho + 0.083\right] \quad (\text{in}^4)$$

where,

b = column width (in)
d = column effective depth (in)
ρ = positive (and negative) reinforcing ratio

For one percent steel (ρ = 0.01) and a square column, b = d = 12 inches.

For Face B:

Step 1. From Figure C.5, determine values of L_n, where,

n = *story level*
L_n = the ranges from point "X" to the mid-height of each story level (Figure C.6) (rounded to the nearest foot)
SD = minimum defended standoff distance along the building face under consideration (ft)

Step 2. Using Table B.1, select the Λ_n values consistent with the ranges, L_n, determined in Step 1.

The calculated L_n and Λ_n values are summarized in Table C.3.

Table C.3. Calculated L_n and Λ values for Face A.

Story Level	Story Height (ft)	Total Height (ft)	range, L (ft)	Λ
1	14	14	100	36.26
2	12	26	102	35.51
3	12	38	104	34.78
4	12	50	109	33.08

The remaining steps should be performed independently for each story level (n). The example will demonstrate the steps for story level four.

Step 3. Calculate Λ_{sn} using equation B.4 (adjust for actual bay width)

$$\Lambda_{s_n} = \frac{w_b}{25}\Lambda_n \quad = \ 15 * 33.08 / 25 \ = \ 19.85$$

where,

bw_b = typical bay width (ft) = 15 ft.

Step 4. Determine the required bent story stiffness (K_{1n}) using equation B.5b for reinforced concrete frames or flat slab structures:

$$K_{1_n} = 16.5595\Lambda_{s_n}^{2.2} \quad = \ 11{,}551 \ \ \text{(lb/in)}$$

Step 5. Calculate the *adjusted* bent story stiffness (K_{2n}) using equation B.6,

$$K_{2_n} = \frac{w_1}{w_{2n}}K_{1_n} \quad = \ 9{,}167 \ \text{(lb/in)}$$

where,

w_1 = Unit weight assumed for equation B.5 derivation = 100 psf
w_{2n} = Adjusted unit weight for story 4 = 126 psf

where,

$$w_{2_n} = w_{a\,n} \frac{TA}{1{,}250} \quad = 126 \text{ psf}$$

where,

w_{an}= actual unit weight = 70 psf (psf)
TA = Tributary plan area of bent = 15 ft x 150 ft = 2250 ft^2

Step 6. Once the adjusted bent story stiffness has been calculated, the required column stiffness can be determined by using equation B.7.

$$K_{col_n} = K_{2\,n} / N = 1{,}528$$

where,

N = number of columns in bent = 6 (75/15 + 1)

Step 7. The required moment of inertia for each column can then be calculated using equation 4.8.

$$I_{col_n} = \frac{K_{col\,n} H_n^{\ 3}}{12E} \quad (\text{in}^4)$$

where,

H_n =Story height = 144 inches
E = modulus of elasticity = 3,834,254 psi

Therefore,

$$I_{col_n} = \frac{K_{col\,n} H_n^{\ 3}}{12E} \quad = 99.2 \text{ in}^4$$

Using this moment of inertia value, the dimensions can be determined from:

$$I_{col\,n} = \frac{bd^3}{2}\left[5.5\rho + 0.083\right] \quad (\text{in}^4)$$

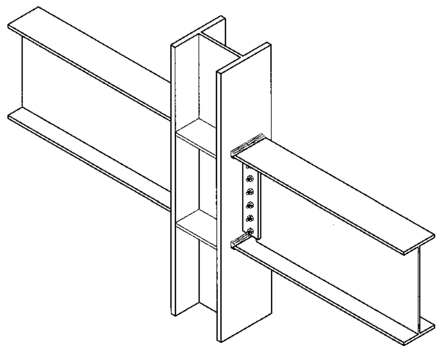
For one percent steel (ρ = 0.01) and a square column, b = d = 7 inches.

Appendix D. Structural Steel Connections

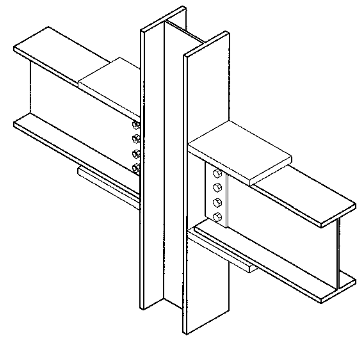
Table D.1. Steel moment frame connection types.

Connection	Description	Type	Figure
Public Sector (Public Domain)			
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	D-1(a)
Welded Flange Plates (WFP)	Flange plate with full-penetration weld at column and fillet welded to beam flange	FR	D-1(b)
Welded Cover-Plated Flanges	Beam flange and cover-plate are welded to column flange	FR	D-1(c)
Bolted Flange Plates (BFP)	Flange plate with full-penetration weld at column and field bolted to beam flange	FR or PR	D-1(d)
Improved WUF-Bolted Web	Full-penetration welds between beam and column flanges, bolted web, developed after Northridge Earthquake	FR	D-1(a)
Improved WUF-Welded Web	Full-penetration welds between beam and column flanges, welded web developed after Northridge Earthquake	FR	D-1(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	D-1(e)
Welded Top and Bottom Haunches	Haunched connection at top and bottom flanges developed after Northridge Earthquake	FR	D-1(f)
Reduced Beam Section	Connection in which net area of beam flange is reduced to force plastic hinging away from column face developed after Northridge Earthquake	FR	D-1(g)
Top and Bottom Clip Angles	Clip angle bolted or riveted to beam flange and column flange	PR	D-2(a)
Double Split Tee	Split tees bolted or riveted to beam flange and column flange	PR	D-2(b)
Composite Top and Clip Angle Bottom	Clip angle bolted or riveted to column flange and beam bottom flange with composite slab	PR	D-2(a) similar
Bolted Flange Plates	Flange plate with full-penetration weld at column and bolted to beam flange	PR	D-1(d)
Bolted End Plate	Stiffened or unstiffened end plate welded to beam and bolted to column flange	PR	D-2(c)
Shear Connection with or without Slab	Simple connection with shear tab, may have composite slab	PR	D-2(d)
Proprietary			
SidePlate™ System (US Patent Nos. 5,660,017, 6,138,427, 6,516,583 and 6,591,573)	Patented moment connection with full-depth side plates and fillet welds, developed following the Northridge earthquake.	FR	D-4
SlottedWeb™ (US Patent Nos. 5,680,738 and 6,237,303)	SImilar to WUF moment connections with extended slots at weld access holes to separate the beam flanges from the beam web in the region of the connection.	FR	D-5

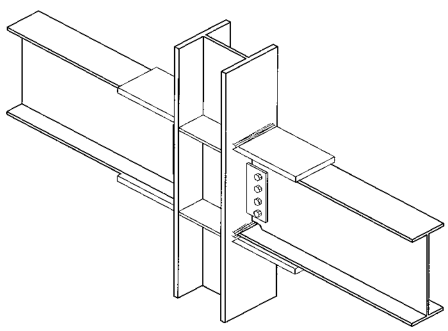
Note: PR = Partially Rigid Moment Connection
FR = Fully Rigid Moment Connection



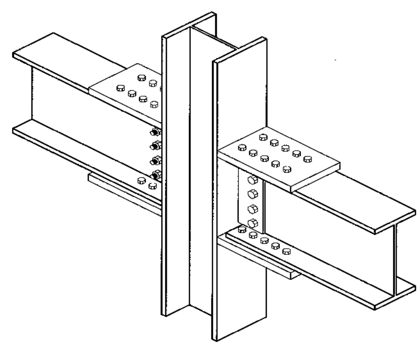
(a) WUF Fully Rigid Connection



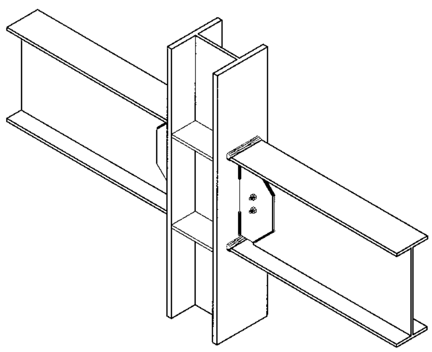
(b) Welded Flange Plate



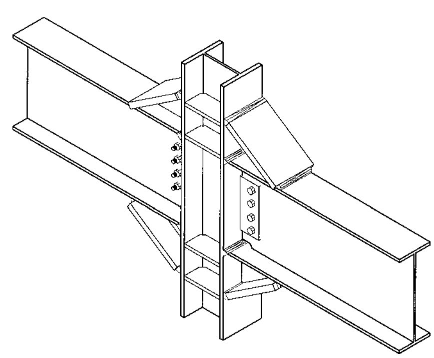
(c) Welded Cover Plated Flanges



(d) Bolted Flange Plate

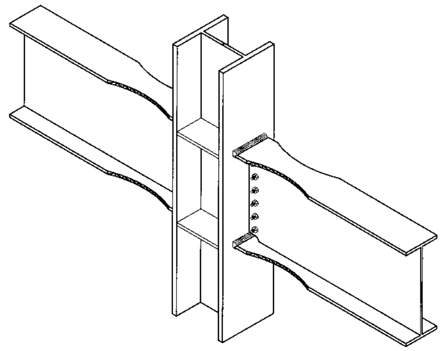


(e) Free Flange



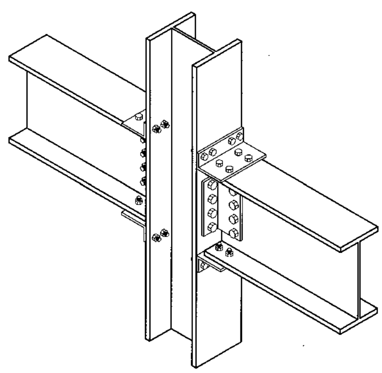
(f) Top and Bottom Haunch

Figure D.1. Fully rigid moment connections.

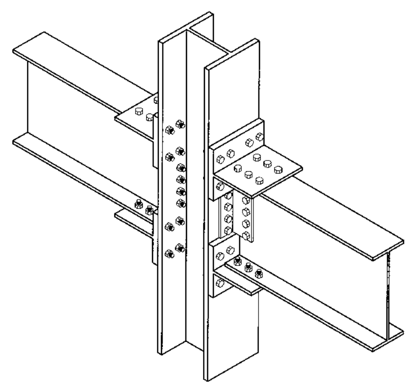


(g) Reduced Beam Section

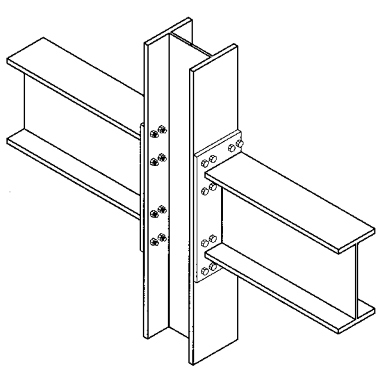
Figure D.1. Fully rigid moment connections (continued).



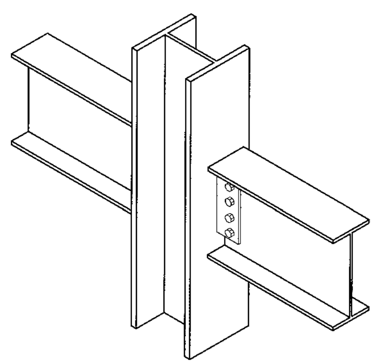
(a) Bolted or Riveted Angle



(b) Double Split Tee

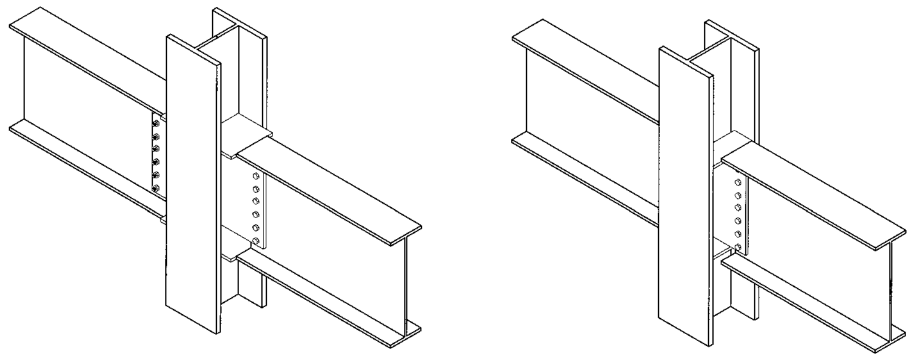


(c) End Plate (Unstiffened)



(d) Typical Shear Connection (without slab)

Figure D.2. Partially rigid moment connections.



(a) Fully Rigid Connection (b) Typical Shear Only Connection

Figure D.3 Weak axis connections.

Proprietary Moment Frame Connections

General

This section presents information on patented fully-restrained steel frame moment connections that have been privately developed. A discussion of several types of proprietary connections is included herein. These proprietary connections have been evaluated by recognized enforcement agencies and found to be acceptable for specific projects and/or for general application within the jurisdiction's authority. Inclusion of these proprietary systems herein does not constitute an endorsement by GSA on their fitness for any specific purpose. Other proprietary connections not included in this listing also exist. Designers wishing to consider specific proprietary connections for use in their structures should consult both the licensor of the connection and the applicable enforcement agency to determine the applicability and acceptability of the individual connection type for the specific design application. Use of these technologies without the express written permission of the licensor is in violation of intellectual property rights, under the patent laws of the United States and other countries.

SidePlate™ Connection System – SidePlate Systems, Inc.

The proprietary SidePlate™ connection system (US Patent Nos. 5,660,017, 6,138,427 6,516,583, and 6,591,573) is used in both new and retrofit construction, and is shown schematically in Figure D.4. Its connection geometry centers around a physical separation (commonly referred to as a 'gap') between the face of the column flange and the end of the beam, by means of parallel full-depth side plates, which inherently eliminates the highly-restrained condition and the high-order tri-axial strain concentrations that are intrinsic to the basic geometry of 'traditional' moment connection systems. Instead, all moment load transfer from the beam to the column reverts back to simple statics, using predictable equivalent force couples and basic engineering principles.

The parallel full-depth side plates act as robust continuity elements to sandwich and connect beam-to-beam, across the column, and are designed with adequate strength and stiffness to force all significant plastic behavior of the connection system into the beam, which, in a worst-case “missing column” scenario, insures the formation of plastic hinges at beam ends, outside the beam-to-column joint itself. SidePlate™ steel frame connection technology replicates the torsional and lateral bending stiffness and strength properties of reinforced concrete beams and girders, in the vicinity of the beam-to-column joint, by creating steel box sections with continuous, robust structural steel plates. This also improves the dynamic performance properties when subjected to blast loading. In addition, the continuous full-depth side plates replicate the continuous top and bottom main reinforcement steel through the column(s), typically provided in modern reinforced concrete structures to insure discrete beam-to-beam continuity across the column. Reliance on panel zone deformation of the column’s web is eliminated by providing three panel zones [i.e., the two side plates plus the column’s own web]. The top and bottom beam flange cover plates are used to bridge the difference between flange widths of the beam(s) and the column.

The construction of the SidePlate™ connection system uses all fillet-welded fabrication, configured with simple unrestrained fillet welds principally loaded longitudinally in shear for increased reliability and robustness.

The SidePlate™ connection’s tested cyclic rotational capacity exceeds all current Connection Qualification Criteria [AISC (2002) Seismic Provisions Structural Steel Buildings and FEMA 350] for large inter-story drift angle demands from earthquakes. The connection has been evaluated and accepted for use as a moment connection in Special Moment Frames (SMF) by the International Conference of Building Officials, ICBO ER-5366, as well as the City and County of Los Angeles (COLA RR 25393 and LACO-TAP Bulletin 3). All full-scale cyclic tests have been conducted at the Charles Lee Powell Structural Research Laboratories, University of California, San Diego.

Additional information on the SidePlate™ connection including use, modeling characteristics, full scale testing and performance can be obtained directly from SidePlate Systems, Inc., Cypress, California, (800) 475-2077 or www.sideplate.com.

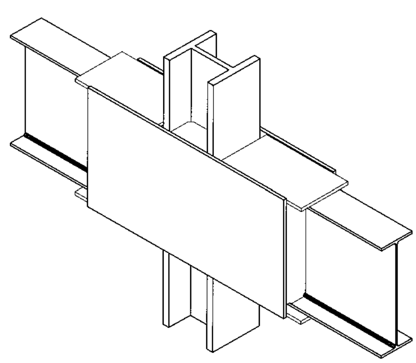


Figure D.4. SidePlate™ moment connection system.

SlottedWeb™ Connection – Seismic Structural Design Associates, Inc.

The proprietary SlottedWeb™ connection (US Patent Nos. 5,680,738 and 6,237,303) is shown schematically in Figure D.5. It is similar to the Welded Unreinforced Flange (WUF) moment connection with the addition of slots in the column and/or beam webs to separate the flanges from the web. Separating the beam web from the beam flanges reduces the large stress and strain gradients across and through the beam flanges by permitting the flanges to flex out of plane. Moreover, the slots in the beam web adjacent to the beam flanges allow the beam web and flange to buckle independently, thereby eliminating the degrading of the beam strength caused by lateral torsional buckling. The connection has been evaluated and accepted for use as a moment connection in Special Moment Frames (SMF) by the International Conference of Building Officials, ICBO ER-5861.

Additional information on the connection and its performance can be obtained directly from Seismic Structural Design Associates, Inc., Camdenton, Missouri, (866) 750-SSDA or www.ssda.net.

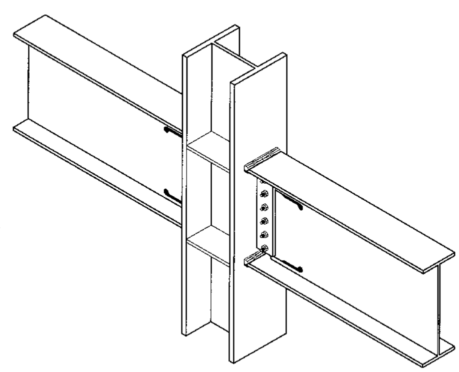


Figure D.5. SlottedWeb™ moment connection.