

COURSE CONTENT

"Progressive Collapse of Structures: Analysis, Prevention, and Mitigation"

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COURSE OVERVIEW

This practical course provides a comprehensive examination of progressive collapse in buildings, from the initiating local failure to the propagation of disproportionate structural damage. It explains the principal causes, mechanisms, analysis procedures, design approaches, and mitigation strategies used to improve structural robustness and reduce the potential for widespread collapse.

The course begins with prominent progressive-collapse incidents, including Ronan Point, the Alfred P. Murrah Federal Building, the World Trade Center towers, and Champlain Towers South. These cases demonstrate how localized damage caused by explosion, impact, fire, deterioration, design deficiencies, or construction errors can develop into extensive structural failure.

Participants will examine the principal progressive-collapse mechanisms, including pancake, domino, zipper, section, instability, and mixed-type collapse. Particular attention is given to load redistribution, dynamic impact, loss of vertical load-bearing capacity, structural continuity, redundancy, ductility, and the development of alternative load paths.

The course reviews threat-dependent and threat-independent design approaches and compares the progressive-collapse provisions of Eurocode EN 1991-1-7, the GSA Guidelines, UFC 4-023-03, and ASCE 76-23. Key-element design, enhanced local resistance, alternative load-path methods, prescriptive ties, structural segmentation, and risk-based design strategies are discussed.

Progressive-collapse analysis procedures are presented, including linear static, linear dynamic, nonlinear static, and nonlinear dynamic methods, together with member-removal scenarios, load combinations, dynamic effects, modeling assumptions, and acceptance criteria. A detailed SAP2000 example demonstrates nonlinear dynamic analysis following the sudden removal of a critical structural element.

The final portion of the course examines practical prevention and mitigation measures for new and existing structures, including structural ties, enhanced continuity and ductility, strengthening, alternative load paths, segmentation, energy-absorption systems, structural fuses, supplemental cable systems, and other techniques intended to limit collapse propagation.

By completing this course, participants will be better prepared to recognize progressive-collapse vulnerabilities, select appropriate analysis procedures, interpret applicable design guidance, and develop practical strategies to improve structural robustness.

Upon completion of the course review, participants must complete a twenty-five-question multiple-choice quiz covering the entire course to earn 3 PDH credits.

LEARNING OBJECTIVES

Upon successful completion of this course, participants will be able to:

1. Explain the causes and fundamental characteristics of progressive and disproportionate structural collapse.
2. Discuss the engineering lessons obtained from prominent progressive-collapse incidents.
3. Distinguish among pancake, domino, zipper, section, instability, and mixed-type collapse mechanisms.
4. Explain how local member failure, impact, and load redistribution can initiate collapse propagation.
5. Evaluate the influence of strength, continuity, redundancy, and ductility on structural robustness.
6. Distinguish between threat-dependent and threat-independent progressive-collapse design approaches.

7. Explain the purpose and limitations of key-element design and enhanced local resistance.
8. Describe how alternative load paths enable a structure to redistribute loads following local damage.
9. Explain how horizontal, vertical, internal, and peripheral ties contribute to structural continuity.
10. Compare the principal progressive-collapse provisions of Eurocode EN 1991-1-7, GSA, UFC 4-023-03, and ASCE 76-23.
11. Explain the purpose and principal stages of a progressive-collapse risk assessment.
12. Identify appropriate member-removal and damage scenarios for progressive-collapse evaluation.
13. Distinguish among linear static, linear dynamic, nonlinear static, and nonlinear dynamic analysis procedures.
14. Explain the principal modeling assumptions, dynamic effects, load combinations, and acceptance criteria used in progressive-collapse analysis.
15. Describe the application of nonlinear dynamic progressive-collapse analysis using SAP2000.
16. Evaluate structural vulnerability following the loss of a critical load-bearing element.
17. Identify prevention and mitigation techniques applicable to new and existing structures.
18. Explain the potential applications of strengthening, segmentation, energy-absorption devices, structural fuses, and supplemental cable systems.
19. Select an appropriate combination of analysis and mitigation measures based on structural configuration, risk, practicality, and economy.

INTENDED AUDIENCE

This course is intended for structural engineers, civil engineers, forensic engineers, protective-design engineers, consulting engineers, licensed professional engineers, building officials, plan reviewers, researchers, educators, advanced engineering students, and other professionals involved in the analysis, design, assessment, rehabilitation, or strengthening of structures for progressive-collapse resistance.

CODE AND GUIDANCE BASIS

The concepts, analysis procedures, and design approaches presented in this course are based on progressive-collapse guidance from ASCE 76-23, GSA progressive-collapse guidelines, UFC 4-023-03, Eurocode EN 1991-1-7, ASCE/SEI 41 acceptance criteria, and the additional technical references identified in the course.

Participants should verify the editions and requirements adopted by the applicable authority, agency, owner, or jurisdiction and review all applicable supplements, revisions, project-specific security requirements, and professional standards before applying the course material to an actual structure.

COURSE CONTENT

Abstract of the Course

The world has seen a surge in rigorous study efforts on the progressive collapse of structures in the past few decades. These events have led to new standards and provisions in building codes of practice, many of which are still being developed and updated today. Although there have been some excellent reviews covering different aspects of progressive collapse, the sheer volume of research performed in this area in recent years means that highly relevant investigation methods and research findings are not covered by them. To fill this void, this review course aims to provide an up-to-date and comprehensive overview of progressive collapse research on building structures. The review is organized into 10 sections that cover:

(1) essential background information; (2) prominent collapse cases; (3) progressive collapse typology; (4) design standards; (5) investigation methods; (6) design guidance for progressive collapse analysis; (7) risk assessment ; (8) design and analysis method; (9) modelling example of progressive collapse analysis using sap2000; (10) design solutions against progressive collapse

Therefore, this course provides a crucial resource to acquire a global overview of current state-of-the-art progressive collapse research and future requirements, making it valuable to both novice and experienced practitioners and researchers.

1- Introduction

Various factors contribute to progressive collapse, with abnormal events being a primary cause. Abnormal events (resulting from fires, natural disasters, human error, wars, or terrorist attacks) have a low probability but significant consequences. These events introduce unanticipated dynamic loads, often overlooked in conventional design processes. Construction, material, and design flaws are other common causes of progressive collapse. For example, corrosion, a material flaw, can overload a member or joint, leading to failure and subsequent collapse of nearby structural components. Design and construction errors may cause misjudgments to a member's capacity, causing failure when subjected to design loads. Thus, preventing progressive collapse is, to some extent, based on the strength of the individual members. However, a comprehensive design considers the overall interaction among structural elements, ensuring a thorough understanding and predictability of structural behavior. The redundancy and ductility of the entire structural system significantly enhances its resistance to progressive collapse.

2. Historic Events

This section provides a brief overview of prominent progressive collapse incidents, elucidating their conceivable origins and preventative methodologies capable of mitigating such occurrences. These notable cases of progressive collapse have wielded considerable influence over both scholarly investigations and structural design standards.

The Ronan Point incident in 1968 involved the collapse of a residential tower after a gas explosion on the 18th floor caused a load-bearing corner panel to fail, which in turn triggered the progression of collapse to the entire corner of the building due to the impact loading of falling debris, as shown in Fig. 1(a). The subsequent collapse demonstrated the potential for a small event to trigger the failure of an entire section of a building. Researchers proposed that adequate ties between panels could have prevented the progression, leading to the development of progressive collapse Codes of Practice (CoPs) in the United Kingdom.

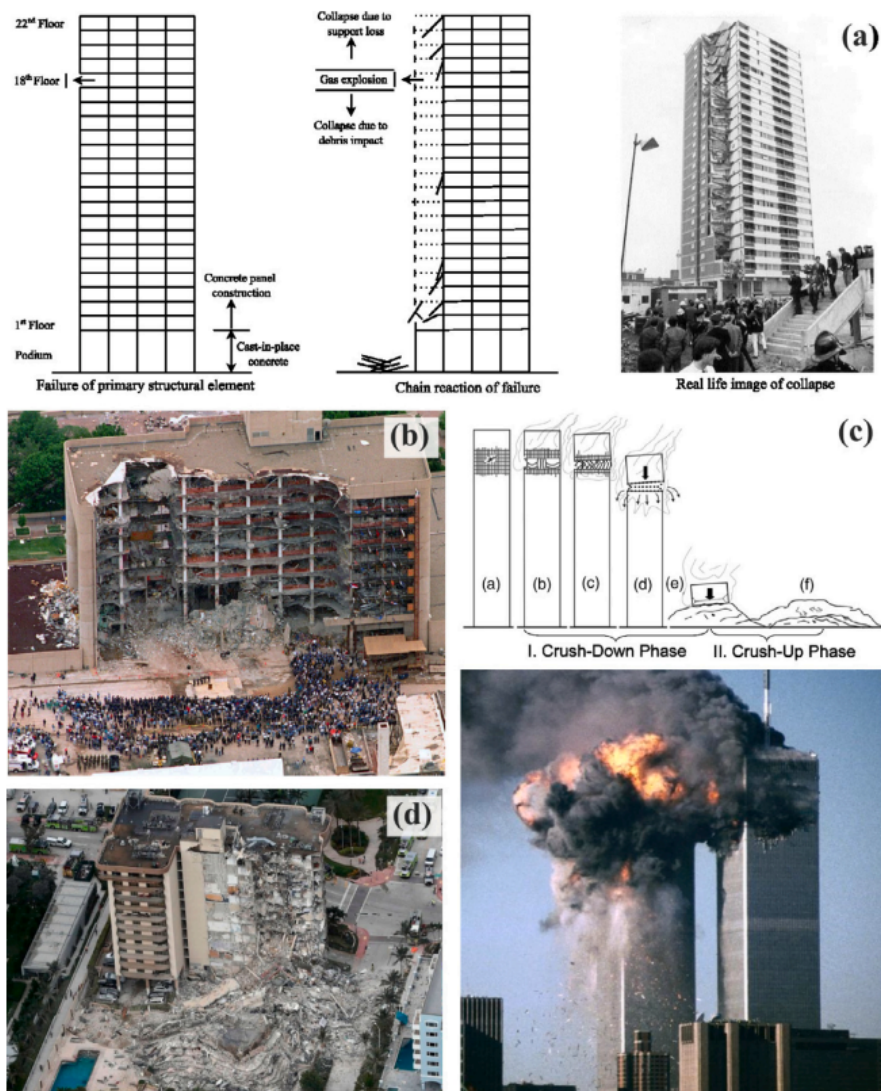


Fig. 1. Progressive collapse events: (a) Ronan point collapse sequence, adapted based on ; (b) Alfred P. Murrah Building after collapse ; (c) Predicted collapse scenario of WTC 1 and 2 and Initial damage endured by WTC Twin Towers ; and (d) Champlain Towers South after partial collapse

The Alfred P. Murrah Federal Building in Oklahoma City suffered a progressive collapse in 1995 due to a truck bomb, leading to the loss of key columns supporting a transfer girder, shown in Fig. 1(b). The disproportionate collapse was attributed to a significant part of the building relying solely on the girder, highlighting the need for mitigation methods, such as alternative load paths and enhanced structural reinforcement.

The collapse of the World Trade Centre (WTC) 1 and 2 towers in 2001, triggered by the impact of hijacked planes, showcased the challenge of halting the progression of collapse in the face of severe initial damage, as shown in Fig. 1(c). The steel structure's properties and potential irregularities in core stiffness could have possibly influenced the collapse, raising questions about the impact of stiffness irregularities on progressive collapse resistance.

The collapse of Champlain Towers South in 2021 involved a sudden partial collapse of a condominium in Florida, as shown in Fig. 1(d). Although the exact cause is still under investigation, deterioration in concrete and reinforcement near the pool deck area and drainage problems were noted in the re-certification report. Adequate waterproofing and retrofitting measures might have prevented the collapse, highlighting the importance of structural maintenance and safety measures in ageing buildings.

Additionally, Table 1 provides a concise overview of several instances of progressive collapse, exemplifying the severe consequences of this phenomenon. The table further states the possible factors contributing to such failures and highlights the disproportionate nature of their impact.

Table 1- Historic progressive collapse events

Incident	Year	Location	Structural system	No. floor	Triggering event	Initial damage	Final damage	Disproportionate
Ronan Point Skyline Plaza Towers	1968	London, UK	Large-panel	22	Gas Explosion	Minor	Partial	Yes
	1973	Fairfax, US	RC frame	26	Premature removal of shoring	Minor	Partial	Yes
Hotel New World	1986	Little India, Singapore	RC frame	6	Static Fatigue	Minor	Total	Yes
L'Ambiance Plaza	1987	Bridgeport, US	Steel frame/ Lift-slab	16	Failure of lifting system	Minor	Total	Yes
Alfred P. Murrah Federal Building	1995	Oklahoma City, US	RC frame with shear wall	9	Truck bomb	Moderate	Partial	Yes
Sampoong Dept Store	1995	Seoul, South Korea	RC frame	5	Overload	Minor	Partial	Yes
Khobar Towers	1996	Khobar, Saudi Arabia	Pre-cast concrete building	8	Bomb explosion	Moderate	Partial	No
Pipers Row Car Park	1997	Wolverhampton, UK	RC frame/ Lift-slab	5	Deterioration, poor maintenance	Minor	Partial	Yes
WTC Bldg 1	2001	New York, US	Steel frame	110	Aircraft impact and fire	Severe	Total	No
WTC Bldg 2	2001	New York, US	Steel frame	110	Aircraft impact and fire	Severe	Total	No
WTC Bldg 7	2001	New York, US	Steel frame	47	Debris impact and fire	Minor	Total	Yes
Windsor Tower	2005	Madrid, Spain	Steel frame-RC core	32	Fire	Moderate	Partial	No
I-35 W Bridge	2007	Minnesota, US	Steel truss-arched bridge	–	Deterioration, poor maintenance	Moderate	Total	Yes
Pyne Gould Corporation	2011	Christchurch, New Zealand	RC frame	5	Earthquake	Minor	Total	Yes
Rana Plaza	2013	Savar, Bangladesh	RC frame	8	Misuse, overload	Minor	Partial	Yes
Texas Railroad Bridge	2013	Texas, US	Wooden trestle bridge	–	Fire	Moderate	Total	Yes
Plasco Building	2017	Tehran, Iran	Steel frame	17	Fire	Moderate	Total	Yes
Surfside, Miami	2021	Florida, US	RC frame	12	Corrosion, poor maintenance	Minor	Partial	Yes

3. Types of progressive collapse

There are different types of progressive collapse. Each type can be characterized depending on the nature of the collapse progression through a structure. The main progressive collapse categories are the pancake, zipper, domino, section, instability, and mix-type. Fig. 2 helps to visualize the most common types of progressive collapse. These types are also grouped into broader categories depending on the mechanism behind the type of collapse. For example, pancake- and domino-type collapses can be grouped into the impact category, as they are caused by the sudden dissipation of the potential energy of the failed elements into kinetic energy. Furthermore, zipper and section collapse types can be attributed to the ‘redistribution’ group since they mainly occur due to the redistribution of forces from failed members to other parts of a structure. In this section, the different collapse types, their possible causes, and potential susceptible types of structures will be explored further.

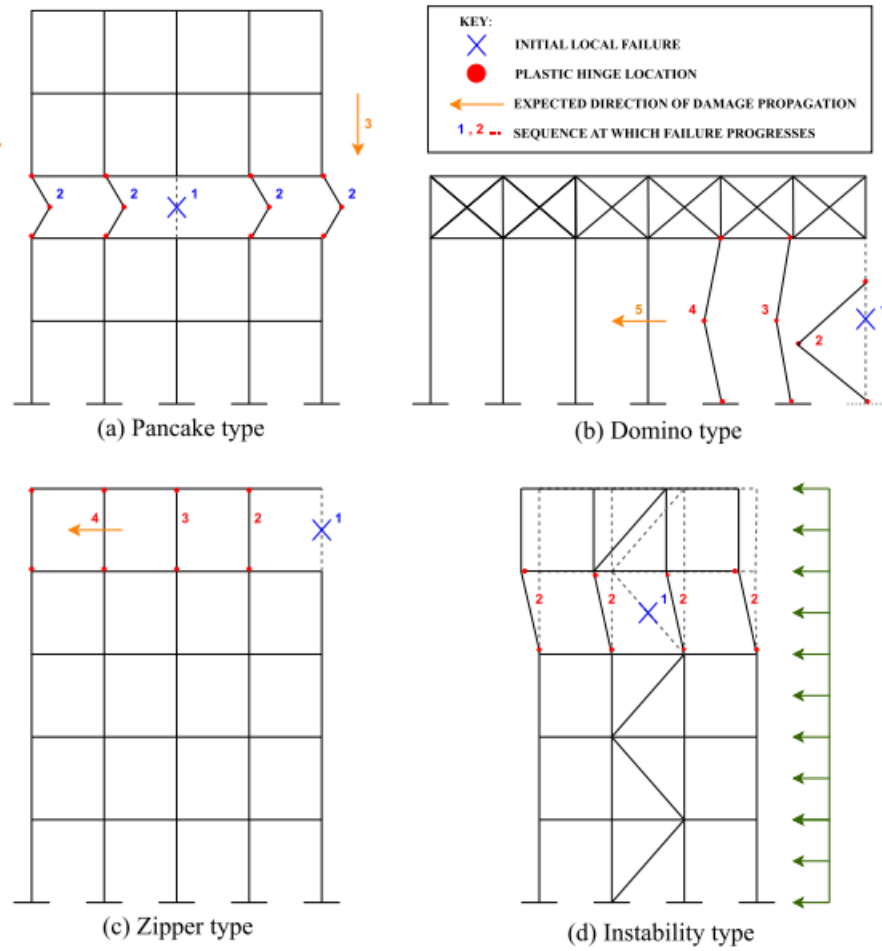


Fig. 2. Demonstration of different collapse mechanisms

3.1. Pancake collapse

The primary cause of the pancake-type collapse is the loss in vertical load-bearing capacity caused by an unusual event, such as a fire or a blast. This then causes the failure of members, which consequently starts falling as debris on members in lower storeys. This debris exerts a high dynamic impact load on the storeys below, in many cases, subjecting these storeys to loadings estimated to be up to four times higher than the static loadings they have been designed for, causing their collapse. This type of collapse is prevalent mainly in high-rise structures. Although many high-rise buildings can be highly redundant and have the ability to develop alternative load paths (ALPs) in case of column loss, they do not have the capability to stop this type of progressive collapse. This is likely attributed to the increase in debris and impact forces with the number of stories in a building.

A prominent example of pancake-type collapse in tall buildings is the collapse of the WTC Twin Towers. Following jet collisions, fires, and initial failures, the resultant debris from the initial failure and subsequent failures possessed a significant amount of kinetic energy that could not have been dissipated without collapsing the storeys below. Pancake-type collapse can occur in low-rise buildings as well. Along with the loss of vertical load-bearing capacity, its primary features include vertical failure propagation and punching shear failures in slabs. Fig. 3(a) shows an example of this type of failure. In order to ensure the efficiency of a building while maintaining its robustness against pancake-type collapses, solutions such as energy absorption devices would potentially be implemented.

3.2. Domino collapse

Domino collapse is another type of impact collapse. In domino collapses, firstly, a member fails due to an initializing event. This failed member then hits a neighboring member laterally, causing the same overturning failure, which then propagates to neighboring members. The primary feature that distinguishes domino-type collapse from pancake-type collapse is that the forces that cause this form of collapse, such as gravity, are orthogonal to the direction of failure propagation. However, in pancake collapses, as can be interpreted, failure-inducing forces are parallel to the direction of collapse progression. Due to its mechanism, domino-type collapse occurs mainly in bridges or horizontal structures due to the failure of piers or other slender supporting members. An example of a domino-type collapse is the failure of a wooden trestle railroad bridge in Texas in 2013. The bridge completely collapsed due to a fire that started in one of the wooden trestles, which then collapsed and impacted nearby members, as shown in Fig. 3(b). This type of failure can be prevented by strengthening or retrofitting the members to withstand potentially induced loads from neighboring member failures.



Fig. 3. (a) Pancake collapse of a reinforced concrete structure following an Earthquake in Islamabad, Pakistan , (b) Domino collapse of a wooden trestle bridge in Texas, the USA , and (c) Pipers Row Car Park partial collapse, Wolverhampton, the UK .

3.3. Zipper collapse

Zipper collapse is one of the most common types of collapse since it can affect most structural arrangements. As mentioned above, zipper collapse is a type of redistribution collapse. This type of collapse occurs when the ALP, which was supposed to carry the load when load redistribution is required due to member failure, also fails. Failure of an ALP can be attributed to the sudden need for dynamic load redistribution. Unlike several other collapse types, impact loadings do not typically play a significant role in zipper type collapses. The failure of the top floor of Pipers Row Car Park, UK, in 1997 (shown in Fig. 3(c)) can be considered an example of this type of collapse. This failure was initiated as one column punched through the top floor slab. The load was then redistributed to other neighboring columns, which could not sustain the

additional loading and eventually punched through the slab. The most current guidelines, which focus on the use and enhancement of ALPs, aim to prevent this type of failure. Different approaches can be followed to enhance the performance of ALPs.

3.4. Section collapse

Section collapse is another type of redistribution collapse that is conceptually similar to zipper collapse. Section collapse, however, can be considered to occur in element sections. An example of this type of collapse can be the failure of a cross section in a tensioned bar. This failure causes further failure in the contiguous parts of the collapsing element due to the inability of the load to be redistributed adequately. Thus, it can be concluded that section collapse does not occur in objects containing structured, independent, but connected units. However, it occurs in single continuous units, such as cables and shells. Due to its abrupt and dependent nature, in many instances, the failure brought on by this kind of collapse can be described as a quick fracture rather than a progressive one .

3.5. Instability collapse

The failure of components, primarily intended to stabilize a structure, results in instability-type collapses. One of these components is bracing. For instance, bracing is necessary for pinned steel frames to ensure a structure's stability under lateral loading. If the bracing fails, the structure cannot withstand lateral loading. Instability failures can lead to immediate or progressive disproportionate collapses, depending on the function and location of the damaged element.

3.6. Mixed-type failure

It is uncommon for a structure to experience only one type of collapse in real-world failures. Therefore, mixed-type collapses predominate. Minimal in-depth research has been conducted to focus on the categorization and combination of different collapse types. However, according to Starossek , a famous example of a mixed type failure is the collapse of Sampoong Superstore in Seoul. In that structure, the failure began as columns were punched through the slabs, and when the load was redistributed, the failure spread horizontally to other columns, inducing zipper-type collapse. This caused a loss of vertical load-bearing capacity in the slabs, which caused pancake collapse.

Another example of this type of collapse is the Alfred P. Murrah Building, USA, in 1995 (refer to Section 2 for more details). In that building, the prevalent type of collapse was pancake collapse, which occurred due to the loss of vertical load-bearing capacity as the columns and, subsequently, the girders were damaged. Investigating the remains of the building also indicated that a domino-type collapse might have occurred. The columns may have been subjected to lateral forces from the initial detonation, which could have caused overturning forces and partially caused the columns to collide laterally. Finally, another example of a structure that underwent a mixed-type collapse is WTC 7. To date, the exact cause of the failure of WTC 7 is still being studied. However, based on the evidence gathered by the National Institute of Standards and Technology (NIST) , the most likely cause of failure is thermal expansion, which may have caused a girder to slide off the column it was resting on. This then led to the pancake failure of the floor area that was supported by that girder. Furthermore, it led

to the loss of lateral support and buckling in the column that the girder was restraining. This resulted in the redistribution of the loads to other members, which had extremely large spans, leading to their failure in a zipper-type collapse. A combination of the two collapses spread throughout the building, causing it to collapse completely in seconds.

3.7. Summary

From studying the various failure cases, it can be concluded that the progression of vertical failure is mainly attributed to pancaketype collapse. In contrast, horizontal progression is mainly caused by zipper-type and domino-type collapses. The most challenging issue in mixed-type failures is that a mitigation technique for one collapse can increase a building's susceptibility to other collapse types. This issue will be discussed further in later sections. Table 2 summarizes the most common collapse types and their possible mitigation techniques.

Table 2-Summary of most common progressive collapse types

Summary of most common progressive collapse types.

Collapse type	Example	Possible mitigation/prevention techniques
Pancake	-WTC Twin Towers (2001) -Sampoong Department Store (1995) -Alfred P. Murrah (1995) -WTC7 (2001)	Energy absorption devices to be applied to ensure impact from pancaking does not cause further vertical collapse propagation
Domino	-Wooden Trestle Railroad Bridge, Texas (2013) -Alfred P. Murrah (1995)	Retrofitting members to withstand loading along minor axes
Zipper	-Sampoong Department Store (1995) -WTC7 (2001)	-Ensuring ALPs are well designed -Proper detailing at column- slab connections to ensure the prevention of punching shear failure

4. Codes of practice and design guidelines

In building design, addressing progressive collapse is a relatively novel concept. Thus, only a few CoPs explicitly provide guidance on how to design against progressive collapse. Mainly, CoPs follow either a threat-dependent or a threat-independent approach. The choice of approach depends on various factors, including engineering judgment, economic considerations, and the nature of the proposed structure. Some of the most commonly adopted current codes within Europe and overseas, as well as their adopted design approaches, will be reviewed and discussed in this section. These codes are: Eurocode EN:1991-1-7, General Services Administration (GSA) Alternative Path Analysis & Design Guidelines for Progressive Collapse Resistance 2016, United Facilities Criteria (UFC) UFC 4-023-03 and the American Society of Civil Engineers (ASCE) ASCE 76-23. The readers are referred to Adam et al. reference for a summary of the progressive collapse prevention methods proposed in several other international design standards and guidance documents.

4.1. Types of approaches

Design standards employ threat-dependent and threat-independent approaches, as discussed in this section.

4.1.1. Threat-dependent approach

A threat-dependent approach mainly depends on designing a structure to be collapse-resistant to a specific threat. This technique is particularly useful in cases where the elements of a building are at high risk from certain known events. An example is a highway bridge or a building constructed close to a highway. In both types of structure, there is a very significant risk that the columns or piers, in the former case, may be struck by a fast-moving vehicle in the event of a highway accident. In such cases, it must be ensured that, for example, these incidents do not lead to a progressive collapse of the structure. Generally, most CoPs require consideration of events whose occurrence can be predicted and characterized, such as fires, earthquakes, and impacts. In the following sections, steps on how to achieve this will be described as per the directives from the various CoPs.

4.1.2. Threat-independent approach

Contrary to the threat-dependent approach, the threat-independent approach is not based on a specific event. The threat-independent method seeks to design a structure with improved strength, ductility, and redundancy levels to prevent progressive collapse under many undetermined risk scenarios. Moreover, IStructE's Manual for Systematic Risk Assessment (2013), for example, proposes adopting a threat-independent design approach as the main risk mitigation technique in a structure. This approach can be effective for several other hazards, and it can help decrease the sensitivity of the design to underlying assumptions usually made in an initial risk assessment. This decreased sensitivity comes from minimizing the presence of what it refers to as 'cliff edges' in the structural response. In other words, it no longer matters whether the loads are slightly higher than what was assumed in the design or if the strength is slightly lower. Thus, the 'cliff edge' defined by the ultimate capacity has been eliminated. Furthermore, several CoPs also guide following a threat-independent approach against progressive collapse design.

4.2. Design approaches

This section will discuss the design approaches most commonly incorporated within progressive collapse CoPs. The main techniques that will be examined include key element design, alternative load path methods, and prescriptive tie requirements. In further sections, the application of those approaches to building CoPs will be discussed.

4.2.1. Key element design

Key element design is a threat-dependent approach applied through locally strengthening elements. This method aims to reduce the probability of initial local failure rather than mitigating collapse propagation. In this method, key elements in a structure and their supporting members are designed to withstand the general minimum prescribed loadings or loadings from certain identified events, such as the impact of a vehicle or an explosion. A key element can be defined as an element whose failure leads to the collapse of a 'significant area' of a structure. That 'significant area' and the loading that should be considered are defined differently in various CoPs. In a structure where several elements are considered key elements, ensuring their collapse resistant design can be very uneconomic. Additionally, disregarding strengthening other elements makes them more vulnerable to potential attacks, even though their structural significance might be less. Thus, to ensure that the benefits of key element design are optimized, this method should be used in conjunction with other global methods,

such as incorporating ties and other redundancy measures. This will ensure the robustness of a structure under various threat scenarios.

4.2.2. Alternative load path method

ALPs can be described as paths in a structure through which loads can be redistributed after loss of an element, enabling the structure to bridge local failure, as illustrated in Fig. 4. Moreover, according to Starossek and Wolff, the ability of a structure to develop ALPs can be used as a measure of its redundancy. Several CoPs highly depend on developing ALPs as the main progressive collapse mitigation technique. To ensure the effectiveness of this method, the adequacy of ALPs under additional, potentially redistributed loads should be considered. To investigate this, detailed analyses should be performed to help understand the behaviour of a structure following the loss of various load-bearing elements. In structural design, the development of ALPs can be enhanced by means of structural ties, strength, and ductility. The incorporation of ties will be further discussed in the following section. Due to the fact that the ALP method depends on enhancing a structure's overall robustness and collapse resistance, it can be considered a threat-independent approach. Other means of enhancing alternative load paths can be considered in the original structural layout design process. An effective structural form or arrangement, in the form of a regular floor layout, for example, can help in the efficient and inherent incorporation of ALPs into a structure.

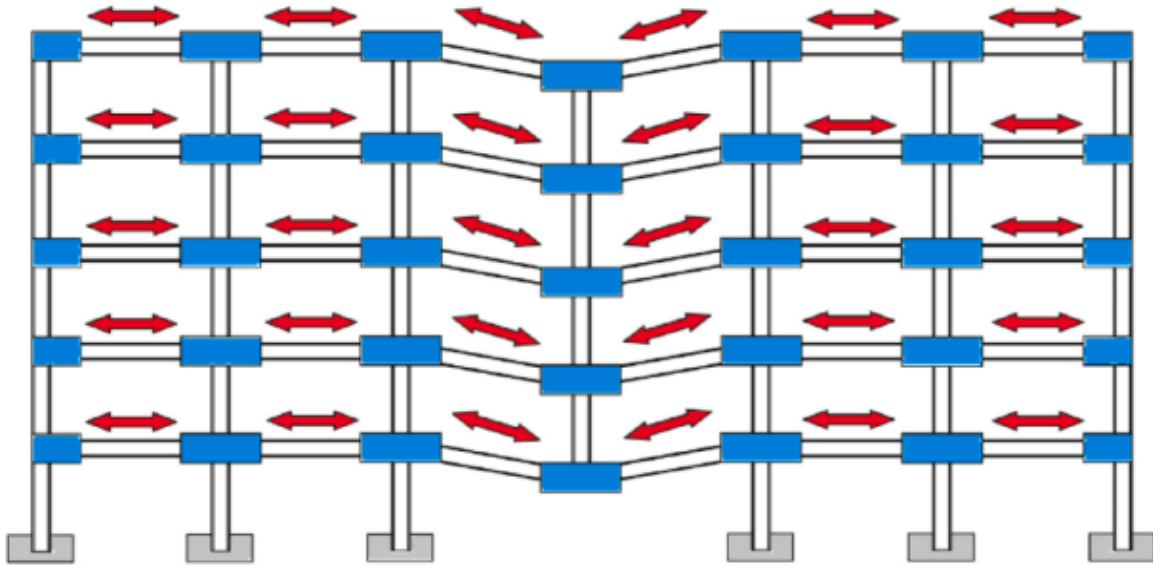


Fig. 4. Load redistribution by alternative load paths (ALPs) under column loss scenario at the catenary stage

4.2.3. Prescriptive tie requirements

For ALPs to develop, continuity must be ensured in a structure. The incorporation of ties is one of the main methods through which continuity can be achieved. In the partial collapse of Ronan Point, the structural panels adjacent to the explosion location were not strong enough to withstand the resulting pressure. However, the main issue is that the building was not redundant enough, i.e., it could not develop ALPs. This was because appropriate tying did not exist between the precast concrete panels. In addition to having enough tying (continuity) between elements in a structure, the structural members should also be able to develop tie forces for an ALP to fully develop. Ties are link members embedded within a structure. One of the main functions of ties is to ensure that the elements of a structural system do not undergo excessive displacements in extreme events, thus preventing the elements from reaching their rotation or strain limits and failing. This helps to ensure that load redistribution can still occur throughout a structure. Several design guidelines propose prescriptive tie-force requirements. Therefore, the tie elements designed using these guidelines will be based on uniform predetermined requirements rather than those determined based on the demand of a system identified following detailed structural analysis procedures.

According to Mann et al. , the types of ties include peripheral, internal, horizontal, and vertical. Peripheral ties are located on the exterior of a structure since they are arguably the most vulnerable part of it in terms of external threats. All peripheral ties should be connected to internal ones for anchoring purposes. Moreover, internal ties are expected to form straight lines across the structure in two orthogonal directions. Internal ties should be designed with high ductility levels to ensure maximum benefit utilization. To address the possibility of walls or columns being pushed outwards, following an internal blast, for example, walls and columns should be tied back to the main structure using horizontal ties. Finally, vertical ties should exist between vertical elements to help identify a clear line of load transfer. Fig. 5 shows the different types of ties recommended for an in-situ concrete structure. Different CoPs have unique guidelines for tie requirements for different types of buildings. However, continuity might not be considered a positive aspect in all cases. This is because it can lead to further collapse as loads from members that fail get redistributed to others that cannot withstand all the additional loading on them. Thus, the concept of continuity can be implemented with segmentation to help prevent collapse from progressing to further sections of a structure.

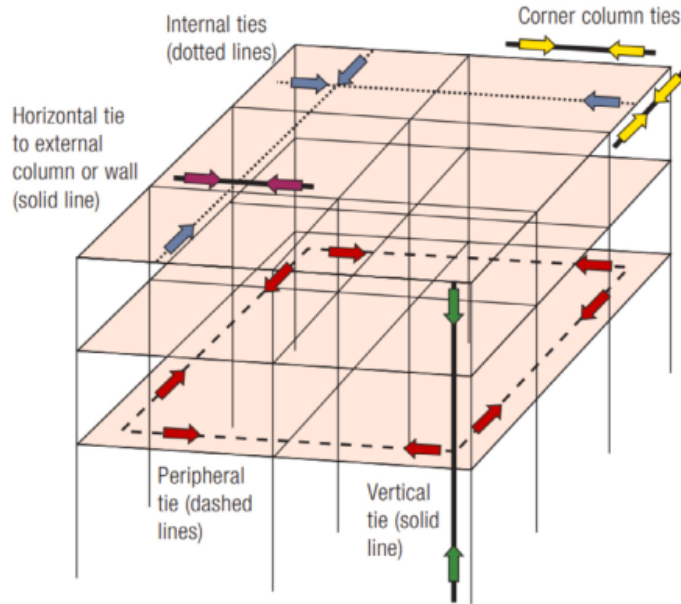


Fig. 5. Types of ties in reinforced concrete structures

4.3. Eurocodes (EN:1991-1-7)

After the Ronan Point incident in 1968, the UK started incorporating design guidance against progressive collapse in the British Code of Practice 110 issued in 1972 (CP 110: Part 1: 1972). This code was one of the earliest national CoPs to provide guidance for progressive collapse resistance design. This document was then followed by the Building Regulations Approved Document A, which was first published in 1992. Similarly, the Eurocodes also started incorporating progressive collapse design in various versions, of which the latest, Eurocode 1-Actions on structures-Part 1-7: General actions-Accidental actions (EN:1991-1-7) last amended in 2014, incorporating guidance from the British codes as well. This section will discuss the guidance in EN:1991-1-7 regarding progressive collapse.

In its design guidance, EN:1991-1-7 implements both threat dependent and independent approaches. Some of the methods adopted by the Eurocode include key element design and the incorporation of ties and redundancy to ensure the ability to develop ALPs. The mitigation and prevention methods that need to be adopted in the design of a structure depend on its consequence class. Consequence classes are risk categories that help determine the criticality of a building based on its size and purpose. Four main consequence classes are defined in EN:1991-1-7: Consequence Class (CC) 1, 2A, 2B and 3. For example, smaller structures, such as residential buildings not exceeding four storeys, lie within CC 2A. Effective horizontal ties or anchorage of floors to walls should be provided for such buildings. Larger structures, such as buildings exceeding 15 storeys, are classified under CC 3. For this category, a systemic risk assessment should be undertaken to provide an understanding of the foreseeable and unforeseeable hazards and, therefore, design the structure accordingly. It is important to note that EN:1991-1-7 provides prescriptive guidance for tie incorporation in different types of

structures. Finally, although this Eurocode does not specify acceptable analysis methods for progressive collapse investigations, it guides the loadings that should be considered for several identified threats. For example, recommended design loads are provided for scenarios such as vehicular impact from a highway and ship impact from a waterway.

4.3.1. Discussion and recommendations

The Eurocodes provide a set of general requirements for progressive collapse design. Following these guidelines alone might, however, be considered insufficient for erecting structures that can be considered adequately ‘collapse resistant’. This can be attributed to the fact that rigorous design and analysis procedures are not proposed by the code. For example, the code does not clearly state the requirement of certain types of analysis procedures for higher-risk structures, such as Consequence Class 3 buildings. Moreover, although guidance is provided for estimating dynamic impact loads or an equivalent static load for various scenarios, a comprehensive method for ensuring that all relevant dynamic effects are accounted for is not included.

Additionally, the code emphasizes the importance of having adequate tying within all the structure, sufficient levels of ductility and continuity between members to ensure the activation of ALPs as a measure of robustness for the structure. However, this does not consider modern research claiming that having high levels of continuity in a structure can lead to further collapse progression. Furthermore, since different types of buildings are more susceptible to certain types of collapse (e.g. tall buildings can be more susceptible to pancake rather than domino-type collapse), the mitigation technique utilized in a structure should address its expected collapse type.

Some initial recommendations for the enhancement of this current code include:

- Dynamic amplification factors can be implemented when following static analyses to generally consider the dynamic effect of loading typically associated with progressive collapse.
- The concept of segmentation can be applied by either having stiffer or weaker elements in the structure to isolate collapse within segment boundaries, ensuring that damage does not further propagate to other areas in a structure.
- Comprehensive design recommendations for higher-risk structures should be outlined.
- Acceptable analysis methods and their applications should be identified.
- The notional accidental load of 34 kPa recommended for use in key element design is not appropriate for most accidental design situations. More specific guidance should be provided in this regard.
- Prescriptive rules for designing continuity reinforcement should be updated to account for research findings of the past decade.

It is important to note that the next generation of the Eurocodes aims to address some of the acknowledged gaps of the current code. Examples include potentially updating the current prescriptive tie methods and incorporating segmentation as a robustness measure.

4.4. GSA (Alternative path analysis & design guidelines for progressive collapse resistance 2016)

The GSA 2016 progressive collapse guidelines could be considered a combination of the different CoPs historically used in the USA, including the Department of Defence (DoD), Unified Facilities Criteria (UFC) and the Interagency Security Committee (ISC) guidelines. The main aim of this document is to bring alignment within the industry by reducing discrepancies between previous guidelines. The GSA guidelines follow a threat-independent approach, which focuses on limiting the progression of initial damage in a structure mainly through ensuring the development of ALPs and redundancy but does not explicitly consider the cause of initial failure. This is assessed by analyzing the effect of various load bearing elements' removal scenarios. This document mainly applies to all new GSA construction and Federal buildings undergoing major structural renovation.

The GSA categorizes structures into different facility security levels (FSLs). The design procedures and analysis methods to be adopted in the progressive collapse design of a structure depend on its FSLs. FSLs are determined based on security/ risk related factors such as target attractiveness, value and criticality. Given that, FSLs are usually determined by specialist bodies. Unlike the Eurocodes, the GSA provides detailed guidance on the acceptable analysis methods that can be adopted by design engineers in progressive collapse investigations. The applicability of an analysis method depends on a structure's regularity, Demand Capacity Ratio (DCR) and number of storeys. The analysis methods proposed in this code are linear static, nonlinear static, and nonlinear dynamic analyses. Linear static analyses are more applicable to regular structures not exceeding 10 storeys. For irregular structures above 10 storeys, nonlinear dynamic analyses could be adopted. Following the analysis process, various column removal scenarios are considered. The performance of a structure is assessed based on certain acceptance criteria. These acceptance criteria are mainly adapted from the Life Safety and Collapse Prevention limits defined by ASCE 41- 06 for seismic design. Adopting these criteria ensures a structure's collapse resistance rather than direct habitability to provide safety while maintaining an economical design.

4.4.1. Discussion and recommendations

The GSA guidelines provide detailed procedures for designing against progressive collapse. The main aim of the guideline is to ensure the development of ALPs under various member removal scenarios. Interestingly, prescriptive tie force requirements were included in previous versions of the GSA guidelines. However, these prescriptive rules have been completely removed in the latest version. In the current guidance, each structure is analysed in detail, and the performance is assessed based on a set of criteria to ensure the adequacy of the design. Although these guidelines might be considered one of the most rigorous, they still have some drawbacks.

Some drawbacks include that not all the initial damage caused by the original cause of element failure is considered. For example, if a bomb exploded near a structure, which led to a column loss, it might also damage other areas of the structure, which can significantly reduce its capacity. However, the GSA guidelines only consider the impact of column loss on structural integrity.

Another issue that can be considered in the GSA guidelines is that it depends only on one technique, which is the development of ALPs. In certain structures (e.g. tall structures with large spans), developing ALPs without having any element failure can lead to designing overly conservative, uneconomic structures. Thus, implementing additional collapse prevention methods with ALPs, including segmentation and energy absorption devices, can provide more economical and practical solutions.

4.5. UFC (4-023-03)

The UFC progressive collapse guidelines are mainly aimed towards the design of structures that the DoD of the USA personnel will occupy. In these guidelines, both direct and indirect, as well as threat-dependent and independent design approaches, are adopted. The alternative path method and the enhanced local resistance (ELR) methods are considered for direct design approaches. As with the GSA, the main aim of the alternative path method is to ensure that a structure is capable of bridging over local failure. Moreover, ELR refers to the local strengthening of elements to ensure sufficient strength for a structure to resist a specific threat. In terms of indirect design approaches, general minimum levels of strength, continuity and ductility are to be adopted. In the UFC, this can be achieved by the prescribed tie recommendations.

Like the Eurocode and GSA, the UFC groups buildings into different risk categories based on a structure's occupancy level and function or criticality. A structure's risk category determines the acceptable mitigation techniques that can be applied in its design process. Where adopting the alternative path method is allowable, a detailed analysis assessing the performance of the structure following a vertical loadbearing element loss should be undergone. Moreover, similar to the GSA, three main acceptable analysis methods exist: linear static, nonlinear static and nonlinear dynamic methods. The performance of the structure is then assessed based on the acceptance criteria adopted from ASCE 41.

4.5.1. Discussion and recommendations

The UFC adopts various design approaches with their applicability dependent on a structure's risk category and the designer's judgment. For example, for a lower risk category, such as RC II, designers can adopt ties and ELR or alternative path design. Such options help ensure that lower risk structures are designed safely and efficiently since only the methods more suitable to the considered structural arrangement can be adopted. As with the GSA, in terms of the alternative path assessment method adopted in this code, the loss of a single vertical load-bearing element should be considered at a time. As discussed previously, this excludes various initial local damage scenarios that could potentially affect the resulting behaviour of a structure.

4.6. ASCE (76-23)

ASCE's primary code for design against disproportionate collapse is ASCE 76-23. This code adopts guidance from various existing CoPs, including GSA 2016, UFC 4-023-03 and EN:1991-1-7. Moreover, this standard addresses the design of new and existing buildings. In ASCE 76-23, threat-independent and threat-dependent methodologies are considered, in

addition to direct and indirect design approaches. Similar to the GSA 2016 code, this standard adopts the alternative load path method to determine the robustness of a structure. Despite the similarities, there are several differences, which are outlined in this section.

Similar to the GSA and the Eurocode guidance, ASCE 76-23 proposes classifying buildings into different Collapse-Resistant Design Categories (CRDCs), CRDCs A, B, C and D. These categories are assigned following a risk assessment procedure which considers the likelihood of a hazard, vulnerability of the structure, the consequences associated with the risk and the building risk category (determined according to ASCE 7-16). The acceptable approach to be followed in the design process, whether hazard-independent or hazard-dependent, depends on the CRDC of the structure. If a threat-independent design procedure is followed, different Hazard-Independent Damage Scenarios (HIDS) should be applied to a structure to assess its performance. In the analysis process, a different suite of HIDSs should be considered for each CRDC, as defined by the code. The main aim of the analysis process in this code is to ensure the ability of a structure to develop ALPs. Like the GSA, the approved analysis methods are the linear static, nonlinear static and nonlinear dynamic procedures. Linear static procedures can be adopted for structures that meet the regularity requirements. Irregular structures that do not meet certain DCR requirements should adopt the nonlinear static or dynamic procedure.

The acceptable damage to a structure is then determined based on a structure's CRDC and the considered HIDS. In this code, this is assessed based on acceptance and performance criteria. The acceptance criteria adopted in this code are similar to those adopted in the GSA code. In terms of the performance criteria, the overall performance of a structure is assessed rather than focusing on individual elements. It is interesting to note that, in this code, partial collapse is acceptable. However, when determined, the impact of debris loading on the structure should be considered when evaluating the extent of failure.

4.6.1. Discussion and recommendations

ASCE 76-23 addresses various shortcomings of previous CoPs. For example, instead of having specified element removal scenarios, this code provides damage volumes to be applied to structures. Additionally, for CRDC D, multi-column removal scenarios or an equivalent damage volume should be considered following the defined HIDS. These recommendations provide a better representation of the initial damage that a structure may have sustained from a potential triggering event in real-life situations. Moreover, this code follows a more robust, systematic way in terms of risk categorization, considering various aspects of a structural system considering both factors relevant to a potential hazard and a building's properties. In terms of recommendations, similar to the GSA, one issue with this code is the high dependence on the development of ALPs. As mentioned previously, this could have negative implications on taller buildings or buildings of larger spans. Although this code briefly discussed segmentation, detailed recommendations for its potential applications have not yet been covered. Similarly, this code recommends undergoing an analysis for debris impact in cases where partial collapse is permitted. However, guidance on how to analyze debris impact has not been provided.

4.7. Codes comparison and summary

Generally, three main design approaches are adopted in current international disproportionate

collapse codes. As discussed previously, these approaches are key element design (local strengthening), alternative load path method and prescriptive tie recommendations. Each of the different codes discussed adopts some or all of these approaches. As noted, a very high similarity is observed between the analysis and performance assessment methods adopted in GSA 2016, UFC 4-023-03 and ASCE 76-23. This is because these three codes adopt this guidance from ASCE 41's seismic performance recommendations. Moreover, as can be concluded from this section, there are still gaps in the guidelines provided by all the discussed codes in terms of disproportionate collapse. Table 3 provides a summary and comparison between the discussed CoPs' approaches to progressive collapse design. Furthermore, to address some of the CoPs' gaps and issues highlighted within this section, Section 6.3 proposes a framework for progressive collapse design that satisfies current code guidance while incorporating proposals from the literature, which will be explored in Section 6.

Table 3- Comparison between the Eurocode, GSA, UFC, and ASCE disproportionate collapse guidance.

Code	EN:1991-1-7	GSA 2016	UFC 4-023-03	ASCE 76-23
Type of Approach	Threat dependent and independent	Threat independent	Threat dependent and independent	Threat dependent and independent
Risk Categories	Consequence classes: 1, 2A, 2B and 3	Facility security level (FSL): I, II, III, IV and V	Risk Category (RC): I, II, III and IV	Collapse-Resistance Design Category (CRDC): A, B, C and D
Ties	Vertical, horizontal, internal and perimeter ties	No specific guidance provided	Vertical and horizontal ties	No specific guidance provided
Acceptable Damage Progression Area	100 m ² or 15% of floor area, whichever smaller, in any two adjacent storeys	~15% of the floor area for exterior column removal ~30% of the floor area for internal column removal	No damage to the floor is allowed	Acceptable damage area is determined based on a structure's CRDC and considered HIDS
Key Element Design	Key elements to be designed to sustain a load of 34 kN/m ² in any direction	NA	Enhanced Local Resistance can be used as a design approach for RC II, III and IV	Local Strengthening could be implemented to reduce the consequences of an identified hazard
ALP	Incorporated through general robustness and ductility measures	Considered the main collapse prevention method applied in the GSA code; their formation is ensured by analysing different column removal scenarios	Alternative Path method can be used as a design approach for RC II, III and IV	Considered the main collapse prevention method applied in the ASCE code; their formation is ensured by analysing different damage volume scenarios
Column Removal Scenario Requirement for Threat Independent Design	Notional removal of each column or each beam supporting a column one at a time at each storey of the building (columns within a plan diameter of 2.25H are to be removed simultaneously; where H is the inter-storey height of the columns [11]).	Different internal and external load-bearing elements removal scenarios should be considered. Generally, a single element removal should be considered at a time.	For the Alternative Path approach, Column/ load-bearing wall removal locations are determined based on a structure's RC. Generally, a single element removal should be considered at a time.	Initial damage is applied in the form of notional damage volume defined based on a considered HIDS. For each CRDC, a suite of HIDS should be applied to a structure.
Accidental Loading Calculation	~An equivalent static load can be acquired for several dynamic sources from tables in the code ~A dynamic load can be calculated for impact cases from Annex C of EN:1991-1-7.	In the static analyses, dynamic loading is accounted for using amplification factors applied to the proposed load combinations.	In the static analyses, dynamic loading is accounted for using amplification factors applied to the proposed load combinations.	In the static analyses, dynamic loading is accounted for using amplification factors applied to the proposed load combinations.
Acceptance Criteria	NA	Elements are classified into deformation-controlled and force-controlled. For each type of analysis (linear static, non-linear static or non-linear dynamic), different acceptance criteria are available for the different element types.	Elements are classified into deformation-controlled and force-controlled. For each type of analysis (linear static, non-linear static or non-linear dynamic), different acceptance criteria are available for the different element types.	Elements are classified into deformation-controlled and force-controlled. For each type of analysis (linear static, non-linear static or non-linear dynamic), different acceptance criteria are available for the different element types.

6. Design guidance for progressive collapse analysis

Progressive collapse first attracted the attention of engineers from the structural failure of a 22-storey apartment building at Ronan Point, London, UK, in 1968. Progressive collapse refers to the action resulting from the failure of one structural element that leads to the failure of further

similar elements, which may result in entire structural collapse in some cases (ASCE, 2005). Vehicular collision, aircraft impact and gas explosions are some examples of the potential hazards and abnormal loads that can produce such an event (NIST, 2007)

The United Kingdom was the first country in the world to publish design guidance for preventing the progressive collapse of the building. The UK Building Regulations (2004) has led with requirements for the avoidance of disproportionate collapse. These requirements are refined in material-specific design codes BS5950 (2001) for structural steelwork. It can be described as: (i) prescriptive 'tying force' provisions that are deemed sufficient for the avoidance of disproportionate collapse; (ii) 'notional member removal' provisions that need only be considered if the tying force requirements cannot be satisfied; and (iii) 'key element' provisions applied to members whose notional removal causes damage exceeding prescribed limits. According to this design guidance, during the design process, the engineers should make sure that all the structures to be designed comply with BS 5950-1:2000 clause 2.4.5 (2001) Structural Integrity. Adequate ties will be incorporated into the frame to reduce the possibility of progressive collapse as required by the building regulation. The 'Key Element' will be designed for sustaining an accidental design loading of 34 kN/m².

The United States was also one of the first countries in the world to publish detailed design guidance on preventing progressive collapse in building design. The Department of Defence (DoD) (2010) and the General Services Administration (GSA) (2003) provide detailed information and guidelines regarding methodologies to resist progressive collapse of building structures. Both employ the alternate path method (APM) to ensure that structural systems have adequate resistance to progressive collapse. APM is a threat-independent methodology, which means that it does not consider the type of triggering event (such as fire, blast etc.) but rather the response of the structure after the triggering event has destroyed critical structural members. If one component fails, alternate paths should be designed to be available for the load such that a general collapse does not occur. Similar to the 'notional member removal', the methodology is generally applied in the context of a 'missing column' scenario to assess the potential for progressive collapse, and is used to check if a building can successfully absorb loss of a critical member. The technique can be used for the design of new buildings or for checking the capacity of an existing structure.

7. Risk Assessment

There are various triggers to progressive collapse, such as vehicle bombs, fire or gas explosion. Due to the limited database of progressive collapse events, it is not reasonable to assess the probability of occurrence of hazards. Therefore, a more reasonable route is to carry out a risk assessment.

According to DOD (2009), the risk assessment can be carried out based on the category of the buildings. It reduces to a consideration of consequences. In general, consequences are measured in terms of human casualties and, therefore, the occupancy of a building or structure is often the most critical issue.

Table 2-1 of DOD (2009) gives detailed information of the occupancy categories. When designing a building to prevent progressive collapse, the engineers can use it to determine the category of the building. Table-2-1 is reproduced as Table 4.

Table 4- Occupancy category

Nature of Occupancy	Occupancy Category
- Buildings in Occupancy Category I in \1\ Table 2-2 of UFC 3-301-01./1/ - Low Occupancy Buildings^a	I
- Buildings in Occupancy Category II in \1\ Table 2-2 of UFC 3-301-01./1/ - Inhabited buildings with less than 50 personnel, primary gathering buildings, billeting, and high occupancy family housing^{a, b}	II
Buildings in Occupancy Category III in \1\ Table 2-2 of UFC 3-301-01./1/	III
Buildings in Occupancy Category IV in \1\ Table 2-2 of UFC 3-301-01./1/	IV
Buildings in Occupancy Category V in \1\ Table 2-2 of UFC 3-301-01./1/	

^aAs defined by UFC 4-010-01 Minimum Antiterrorism Standards for Buildings.

^bOccupancy Category II is the minimum occupancy category for these buildings, as their population of function may require designation as Occupancy Category III, IV, or V.

United Facilities Criteria-DoD, 4-023-03, Whole Building Design Guide, a program of the National Institute of Building Sciences, 2009, Table 2-1.

8. Design and analysis method

The main objective of a progressive collapse analysis is to determine the vulnerability of a structure to progressive collapse and design corresponding mitigation measures to prevent such an occurrence. The design guidance in the UK and USA provides three basic design approaches: an indirect method and two direct methods.

8.1 Indirect design method

The indirect method consists of prescriptive design guidance to improve the robustness of the building by stipulating the minimum tying forces for connections, identifying key elements and designing for nominal increased design loading. It designs through the provision of minimum levels of strength, continuity and ductility, and therefore is the basic requirement for preventing progressive collapse of the buildings. This method has been adopted by both BS5990 (2001) and ASCE (2005).

8.2 Direct design method

Direct design methods consist of design guidance to increase the strength of the main structural elements to avoid failure under accepted abnormal loads or making the structure so that it can

cover the local failure area. There are two direct design methods.

The first direct method is to assess the capacity of the individual members and also the global structure to withstand the loading applied to the structure under each design level hazard. Thus, check their ability to withstand the hazard within agreed acceptance criteria.

Another direct design method is the well-known alternate load path method. This method consists of a systematic assessment of the hazards that can be applied to each structural element and a judgment of whether it can withstand that hazard level. If it is deemed that the structural element would not survive the hazard level then a structural model of the elements and their surrounding structure is developed. This structure is then assessed with the sudden removal of critical structural members, such as columns, to study the structure's ability to redistribute the forces on the structure without triggering progressive collapse. This method has been widely used in current analysis, most researchers such as Fu (2009) and Kim and Kim (2009) use this method in their research. The best way to implement this method is either through Abaqus® or SAP2000; the detailed modelling method will be introduced in a later section. General Services Administration (GSA) (2003) provides the detailed Guidelines on the Alternate Load Path Method that enables structural engineers to analyze the ability of a structure to withstand progressive collapse.

8.3 Selection of design method

The selection of above design methods is mainly based on the occupancy category of the building by design guidance such as British Building Regulations (2000) and DOD (2009). Table 2-2 in DoD (2009) (as shown in Table 5) provides detailed requirements of the progressive collapse analysis method base on the occupancy category that the engineers can refer to. Similar requirements are also provided in British Building Regulations (2000).

Table 5- Occupancy category and design requirements

Occupancy Category	Design Requirement
I	No specific requirements
II	Option 1: Tie Forces for the entire structure and Enhanced Local Resistance for the corner and penultimate columns or walls at the first story. OR Option 2: Alternate Path for specified column and wall removal locations.
III	Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first story columns or walls.
IV	Tie Forces; Alternate Path for specified column and wall removal locations; Enhanced Local Resistance for all perimeter first and second story columns or walls.

United Facilities Criteria-DoD, 4-023-03, Whole Building Design Guide, a program of the National Institute of Building Sciences, 2009, Table 2-1.

8.4 Structural analysis procedures and acceptance criteria

Based on the above design approach, four main analysis procedures were included in GSA (2003). They are: linear static, linear dynamic, nonlinear static and nonlinear dynamic. In progressive collapse analysis, an engineer should choose the approach based on the size and importance of the facility considered and the degree of structural regularity. The correspondent loading and the acceptance criteria to be used are dependent on the type of analysis adopted.

Load combinations

It is recognized that under certain column removal scenarios the loading present will be less than the ultimate design loading for the structure; therefore, GSA (2003) recommended that the load present on the column removal should be the full dead load plus 0.25 times the live load. However, the structural performance under a possible column removal is influenced by the dynamic characteristics of the structure itself, such as the dynamic amplification effect, which are automatically captured for dynamic procedures but not for static analyses. Therefore it is recommended that for static procedures, the load applied to the structure is increased to two times the value recommended above to be on the safe side. The required load combinations are summarized in Table 6.

Table 6- Load combination and acceptance criteria

	Load combination	Acceptance criteria
Static Linear	2x(Dead+0.25Live)	Demand capacity Ratio
Static Nonlinear	2x(Dead+0.25Live)	Rotation/Ductility
Dynamic Linear	Dead+0.25Live	Demand capacity Ratio
Dynamic Nonlinear	Dead+0.25Live	Rotation/Ductility

Acceptance criteria and analysis steps for linear procedures

The acceptance criteria for the four analyses are also shown in Table 6. For linear elastic procedures, the acceptance criteria are based on the demand capacity ratio (DCR) for each structural member and connection in the design. The use of linear elastic procedures is limited to structures that meet the requirements for irregularities and DCRs. Please refer to DOD (2009) for further details. The DCR is simply the force in each member/connection under the considered scenario divided by the expected ultimate, un-factored capacity of the member/connection considered. It is calculated as follows:

$$DCR = Q_{UDLim}/Q_{CE}$$

Where:

Q_{UDLim} are resulting actions (internal forces and moments)

Q_{CE} is expected strength of the component or element, as specified in Chapters 4 to 8 in DOD (2009)

Figure 4.3 of the GS A (2003) contains the DCR limit calculation table for beams and columns, the reader can refer to it to choose the correct DCR ratio in the design.

Here, we are going to introduce the steps of linear elastic static progressive collapse analysis, based on the requirement of DOD (2009) and GSA (2003).

Analysis procedure for linear elastic static progressive collapse analysis

- **Step 1.** Remove a vertical support from the location being considered and conduct a linear static analysis of the structure of the model with the load combination of 2(DL +0.25LL) according to Table 6.
- **Step 2.** After initial analysis, determine which members and connections have DCR values that exceed the acceptance criteria provided GSA (2003).
If the DCR for any member end or connection is exceeded based upon shear force, the member is to be considered a failed member.

If the flexural DCR values for both ends of a member or its connections, as well as the span itself, are exceeded, the member is to be considered a failed member.

- **Step 3.** Remove failed members from the model, and all dead and live loads associated with failed members should be redistributed to other members in adjacent bays.
- **Step 4.** 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 can be achieved using analysis program such as SAP2000.)
- **Step 5.** Re-run the analysis and repeat steps until no DCR values are exceeded.
- **Step 6.** Judgment of the collapse of buildings is performed as follows.

If moments have been re-distributed throughout the entire building (this can be achieved through checking the distribution of plastic hinges in analysis packages such as SAP2000) 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.

From the above procedure, we can see that the linear elastic static analysis procedure is quite simple and straightforward. But an engineer needs to repeat the steps several times before reaching a conclusion. Readers can fulfil this analysis through software such as SAP2000 or ETABS.

Acceptance criteria and analysis steps for nonlinear procedures

As shown in Table 6, for nonlinear procedures, the acceptance criteria are based on less onerous rotation and ductility demands for the members/connections considered. The analysis can be achieved using such approaches.

Analysis procedure for nonlinear dynamic procedure procedures

- Development of a model (ideally this model could include materially nonlinear, which can be achieved using a general purpose program such as ANSYS or Abaqus® as they can directly simulate the nonlinear feature such as the plasticity without inserting plastic hinge, with a load combination of full dead load plus 0.25 times the live load according to Table 6).
- A static step to determine the forces under this load combination of full dead load plus 0.25 times the live load.
- The removal of a column in a dynamic time step which is less than 1/10th the period of the fundamental mode including geometric nonlinearity. (In the author's experience the removal duration is important to achieve reliable analysis result). The determination of the subsequent free dynamic response under the column removal scenario including geometric nonlinearity (if the analysis program allows; programs such as Abaqus® and ANSYS allow geometric nonlinearity) or inserting the plastic hinges (in a program such as SAP2000 etc.).
- The maximum forces, displacements and rotations for each of the members/connections involved in the scenario will be determined from the dynamic analysis for comparison with the acceptance criteria outlined in Table 2.1 of the GSA guidelines (2003).

Summary

It can be seen that the recommendations in Table 6 for each type of analysis result in a different degree of inherent conservatism. The actual degree of conservatism will be dependent on the structural form considered. The linear static procedure is easy to implement, the nonlinear

dynamic procedure requires more modelling techniques, and therefore, in the next section we are going to demonstrate how to implement it in SAP2000.

8.5 Modelling example of progressive collapse analysis using SAP2000 – nonlinear dynamic procedure

In this section, a modelling example of progressive collapse analysis using SAP2000 will be demonstrated. The nonlinear dynamic procedure will be used here. Therefore, the load combination used here is $DL+0.25LL$, as required by GSA. The basic procedure for performing a progressive analysis in SAP2000 is through a time history analysis. The general procedures are as follows:

1. Set up a detailed 3D model.
2. Run an initial static analysis with the load combination $DL+0.25LL$.
3. Delete the column that will be removed, remember to check the column load before deleting.
4. Replace the column with an upward joint load of the same magnitude as an axial force in the column that was deleted.
5. Run column removal analysis by using a time history function that will remove the load applied in Step 4 over a user defined time period to simulate the removal of the column.

A 5-storey steel composite moment frame building was set up in SAP2000 as shown in Figure 16.

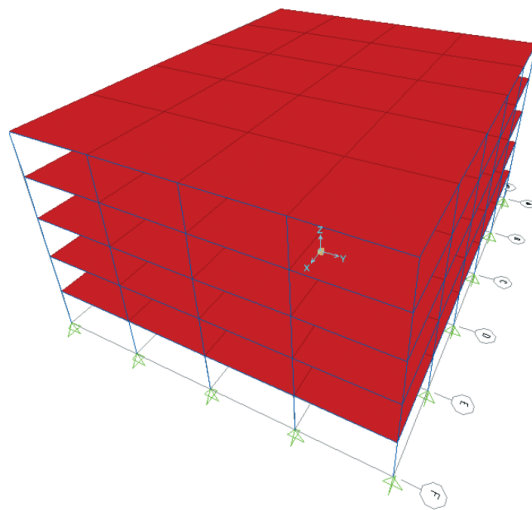


Fig. 16- A 5-storey steel composite moment frame building.

A step by step set of instructions is included below:

Step 1: Define removal time history

- Click *Define, Function, Time History*, and add *User Defined Function* as shown in Fig. 17

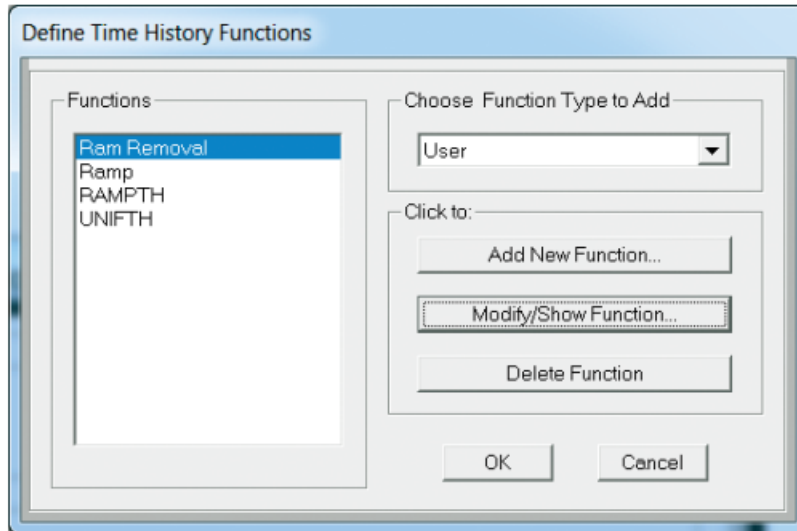


Fig. 17. Defining the time history function.

- Click *Add New Function*.
- A new window will pop up, as shown in Figure 18, define the time history function as shown.

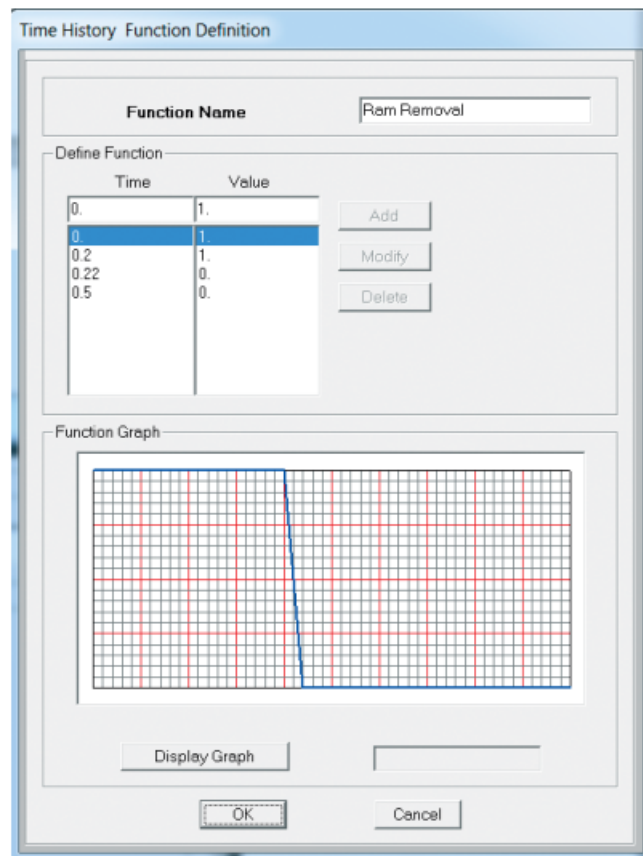


Fig. 18- Time history function for removal

Step 2: Define load cases

- Define load pattern.

Define the load pattern as shown in Fig. 19, add a reaction load pattern called *Reaction*.

Load Pattern Name	Type	Self Weight Multiplier	Auto Lateral Load Pattern
reaction	OTHER	0	
DEAD	DEAD	1	
Cladding	DEAD	0	
reaction	OTHER	0	
Live	LIVE	0	

Fig. 19- Load pattern

- Define load cases.

There are three extra load cases that need to be defined here, they are *Reaction Cases*, *Initial Static Cases*, and *Removal Cases* respectively, as shown in Figure 20

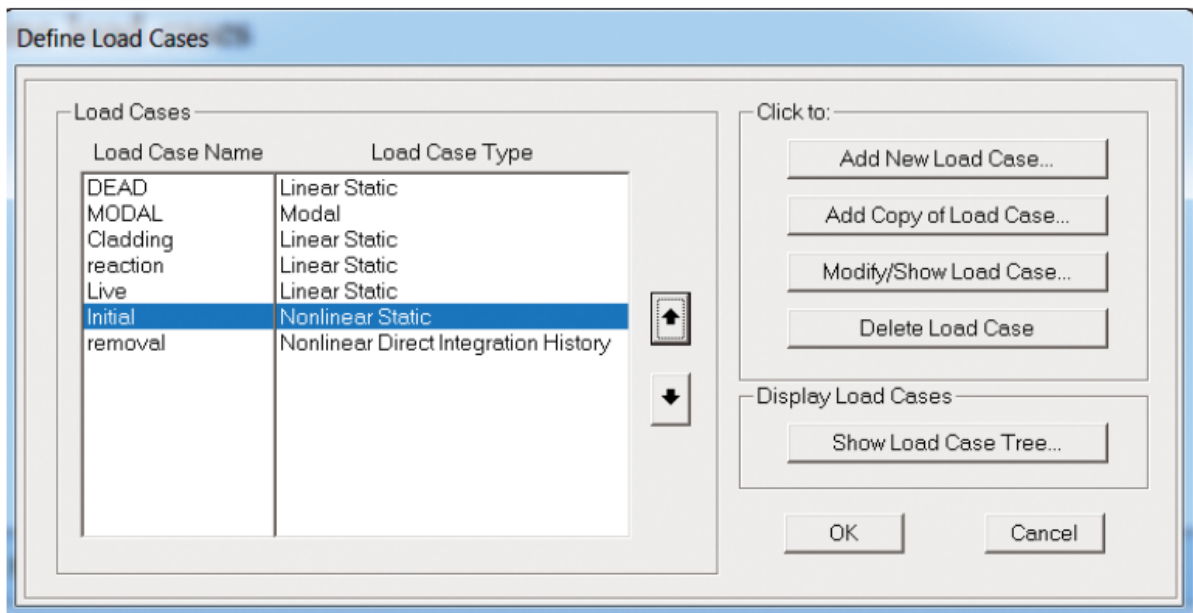


Fig. 20- Load cases.

- For the initial static load case, choose the parameters shown in Figure 21, the scale factor is chosen according to the required load combination by GSA.

Load Case Data - Nonlinear Static

Load Case Name: Notes:

Load Case Type:

Initial Conditions:

☒ Zero Initial Conditions - Start from Unstressed State

☐ Continue from State at End of Nonlinear Case

Important Note: Loads from this previous case are included in the current case

Modal Load Case:

All Modal Loads Applied Use Modes from Case:

Loads Applied:

Load Type	Load Name	Scale Factor
Load Pattern	Live	0.25
Load Pattern	Cladding	1
Load Pattern	DEAD	1
Load Pattern	Live	0.25

Analysis Type:

☐ Linear

☒ Nonlinear

☐ Nonlinear Staged Construction

Geometric Nonlinearity Parameters:

☒ None

☐ P-Delta

☐ P-Delta plus Large Displacements

Other Parameters:

Load Application:

Results Saved:

Nonlinear Parameters:

Fig. 21- Initial static analysis case

- For the column removal load case, choose the parameters as shown Figure 22; remember to set the analysis to continue from the initial static analysis. For the *Number of Output Steps* choose 100, for *Output Time Step Size* 0.01

Load Case Data - Nonlinear Direct Integration History

Load Case Name: Notes:

Load Case Type:

Initial Conditions:

☐ Zero Initial Conditions - Start from Unstressed State

☒ Continue from State at End of Nonlinear Case

Important Note: Loads from this previous case are included in the current case

Analysis Type:

☐ Linear

☒ Nonlinear

Time History Type:

☐ Modal

☒ Direct Integration

Geometric Nonlinearity Parameters:

☒ None

☐ P-Delta

☐ P-Delta plus Large Displacements

Modal Load Case:

Use Modes from Case:

Loads Applied:

Load Type	Load Name	Function	Scale Factor
Load Pattern	reaction	Ram Remov	1.
Load Pattern	reaction	Ram Removal	1.

☐ Show Advanced Load Parameters

Time Step Data:

Number of Output Time Steps:

Output Time Step Size:

Time History Motion Type:

☒ Transient

☐ Periodic

Other Parameters:

Damping:

Time Integration:

Nonlinear Parameters:

Fig. 22- Column removal cases

Step 3: Run initial static analysis and apply reversed column load

- Run the static analysis.
- Check the column load under the initial static load case for the targeted removal columns, and then delete the column manually.
- Apply a reversed equivalent value reaction force, make sure to select the load case *Reaction* in Figure 23.

The screenshot shows the 'Joint Forces' dialog box. The 'Load Pattern Name' is set to 'reaction'. The 'Units' are set to 'KN, m, C'. The 'Coordinate System' is set to 'GLOBAL'. The 'Loads' section has the following values: Force Global X: 0., Force Global Y: 0., Force Global Z: 728.676, Moment about Global X: 0., Moment about Global Y: 0., and Moment about Global Z: 0. The 'Options' section has 'Replace Existing Loads' selected. The 'OK' and 'Cancel' buttons are at the bottom right.

Fig. 23- Defining the reversed column load

Now the reaction force is applied to the point where the column has been removed, as shown in Figure 24.

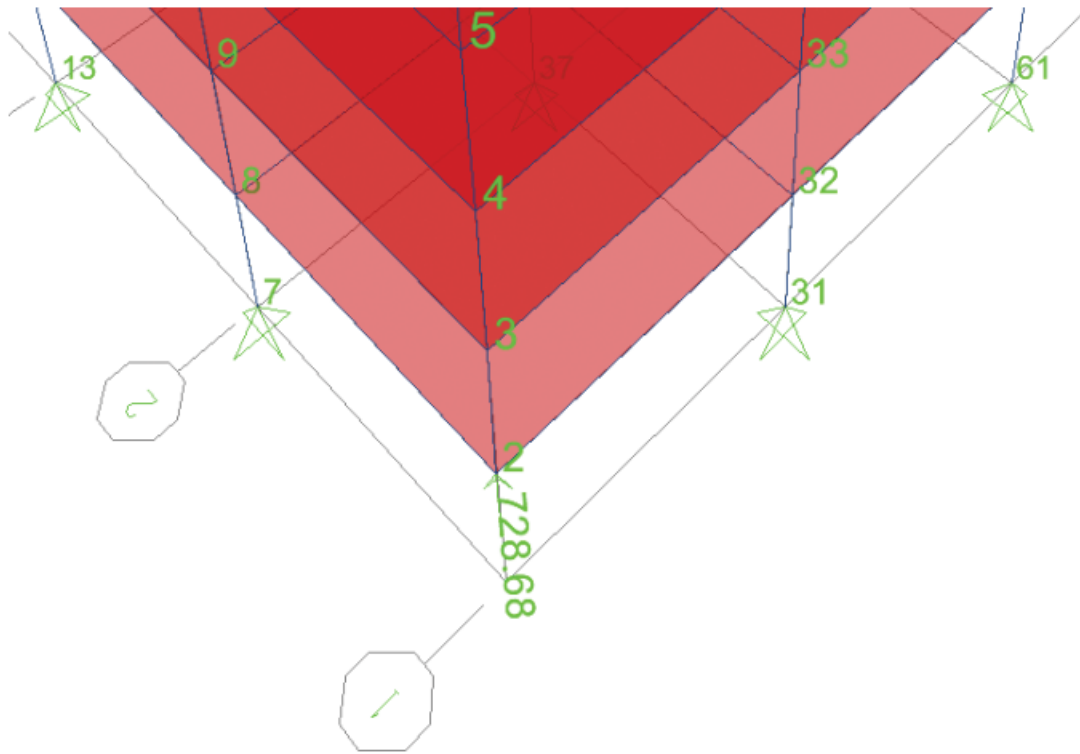


Fig. 24- Reversed column load applied to the building

Step 4: Define the plastic hinges to column and beams

The plastic hinge can be defined and assigned. For the rules to define the plastic hinges, please refer to GSA and DOD for detailed instructions.

Step 5: Run removal analysis

After the plastic hinge definition, run the removal analysis.

Step 6: Result interpretation

- Go to *Display – Show Plot Function*; a new window will pop up as shown in Figure 25
- Click *Define Plot Functions* in Figure 25.
- A new window will pop up as shown in Figure 26, choose *Add Frame Forces* (you can choose displacement or internal force of the beam or column to investigate: here, we choose the frame force for demonstration).
- Put in the element number (beam or column ID, you can check this in SAP2000)

you want to investigate, as shown in Figure 27.

- And select the axial force or any other internal force you want to check.
- Display the plot function traces

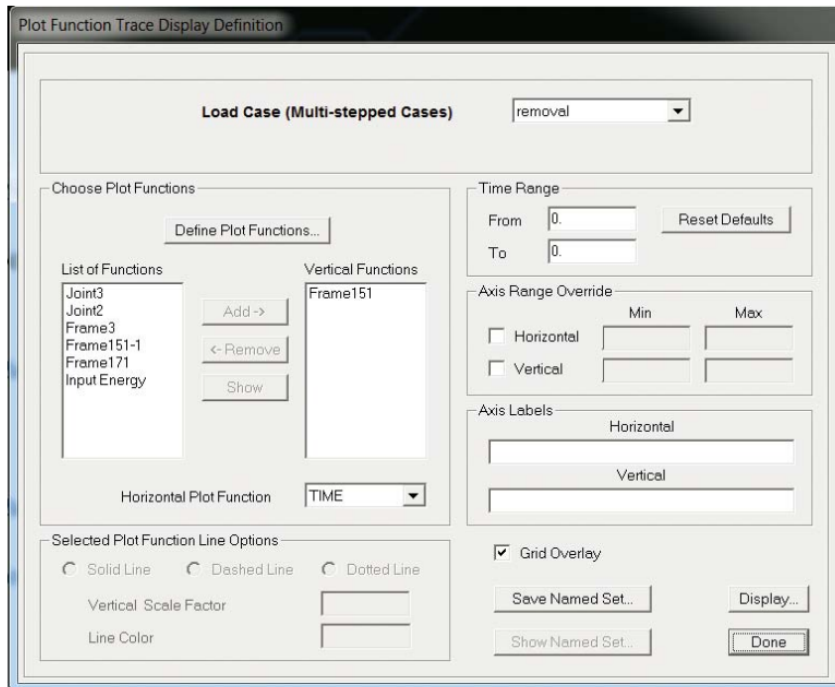


Fig. 25- Plot time history.

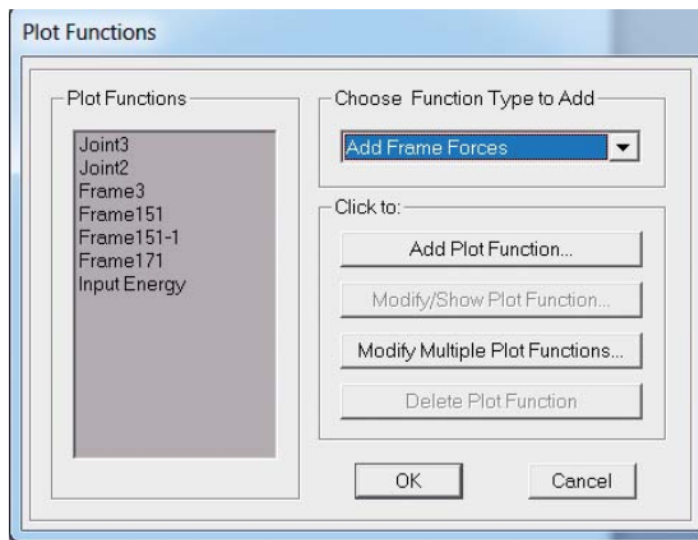


Fig. 26 - Add frame force

Frame Plot Function

Plot Function Name

Element ID

Component

☒ Axial Force ☐ Torsion

☐ Shear 2-2 ☐ Moment 2-2

☐ Shear 3-3 ☐ Moment 3-3

Mode Number

☒ Include all

☐ Include one

Location is at Output Station Nearest

☒ Relative Distance

☐ Absolute Distance

Fig. 27- Choosing the component and frame ID.

Figure 28 and Figure 29 show the result of the time history for Frame 151 and Frame 2: we can clearly see the dynamic response of the structural member after the column removal.

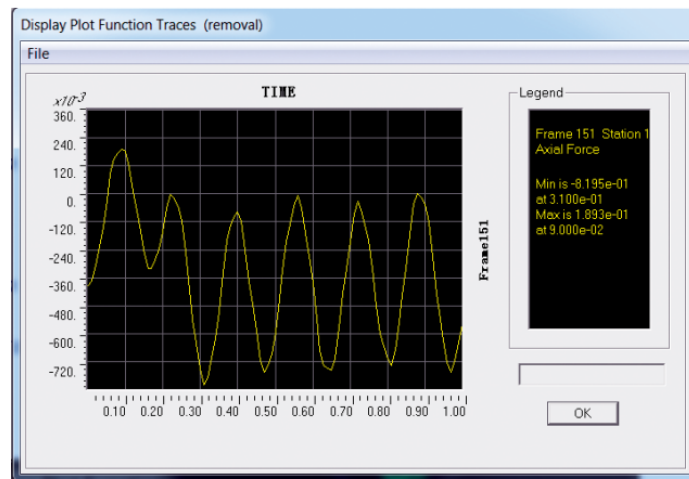


Fig. 28- Time history for axial force of Frame 151

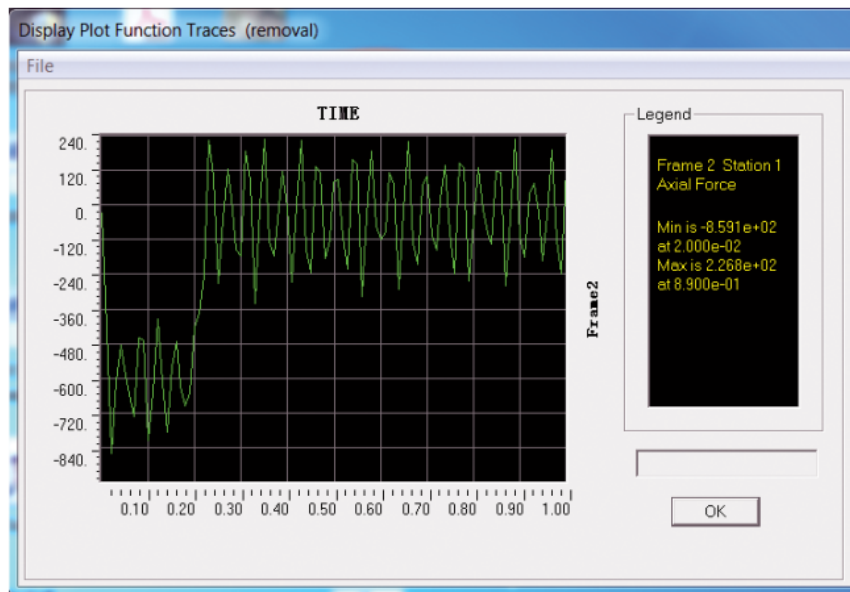


Fig. 29- Time history for axial force of Frame 2

9. Design Solutions against Progressive Collapse

There are two main philosophies typically adopted in progressive collapse solutions. The first philosophy aims to completely prevent collapse, which can be achieved through designing a structure to bridge over a lost element. Although this can be effective in structures of smaller spans and single-element loss scenarios, this class of solutions can be impractical and extremely costly in larger-scale projects. The second set of solutions proposes limiting or mitigating collapse rather than preventing it. To achieve this, for example, a structure can be divided into sections within which collapse is allowable as long as it does not propagate to other parts of the structure. This section discusses the two main philosophies adopted for most progressive collapse solutions as well as their attributed methods proposed in the literature.

9.1 Prevention methods

To date, most progressive collapse design proposals aim at preventing collapse rather than limiting it. Some proposed prevention methods, which will be discussed in this section, include member retrofitting, the implementation of steel cable systems, additional reinforcement and seismic design parameters in addition to other non-structural measures.

Steel cable systems. Cable systems were proposed as a prevention technique for new and retrofitted structures. One of the proposed systems consists of cables connected at beam ends running parallel to the columns. These cables are then connected to trusses located at the top of the structure, as shown in Fig. 30. The main function of this proposed system is to re-transfer loads from a lost column to other members in the structure through cables and trusses. Moreover, tension forces developing in the cables above the removed column can help critically reduce deflections in the members around and above the removed columns. This can help keep larger sections of the structure performing linearly to reduce the cost of any associated damages and restoration needed after the column loss event. Many researchers studied the applicability of this system using the nonlinear dynamic analysis procedure proposed by the United Facilities Criteria using the analysis software like ANSYS and SAP2000, the conclusions were relatively similar. An alternative implementation for steel cables as a progressive collapse prevention measure by placing cables within slabs or on top of girders along the exterior column lines of structures. The main function of this system is to ensure that if a perimeter column is lost, the structure can redistribute loads through the cables using catenary action. This system was experimentally tested using a full-scale specimen representing one floor of a steel structure. Both cable systems proved to successfully help in load redistribution, deformation reduction and thus prevention of progressive collapse. Cable systems can be beneficial in structures with no architectural or cost constraints. However, they might not be applicable to all types of structures, and these constraints become more apparent in lower-rise structures where the cost of installation and maintenance of this type of system might form a significant portion of the overall cost of the project.

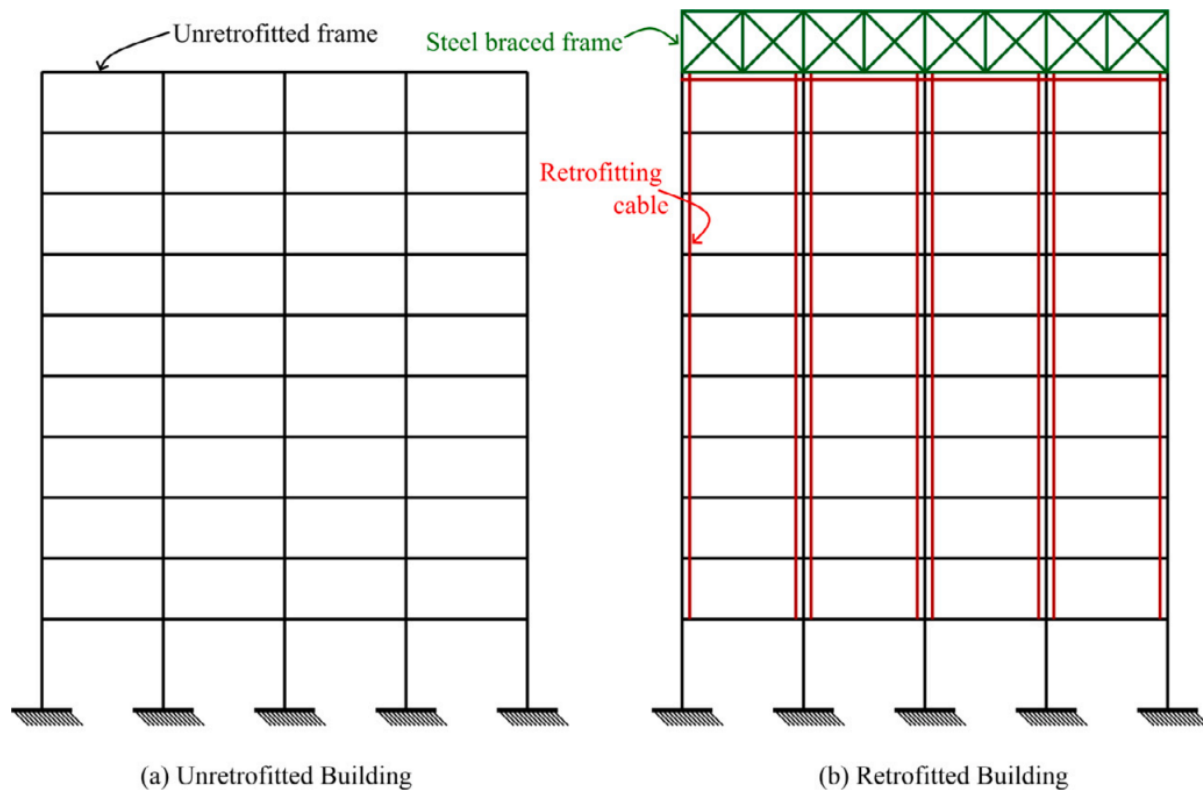


Fig30. Progressive collapse resistance cable system.

Member retrofitting. To limit the impact of an extreme event such as a blast, some researchers have proposed retrofitting members in a structure. Retrofitting can be applied to new and existing buildings and usually has one of three main aims: mechanism enhancement, strengthening, and energy absorption. Retrofitting aimed at mechanism enhancement focuses on trying to enhance the collapse-resisting mechanisms in a structure by acting as external reinforcement. For example, carbon-fibre reinforced polymers (CFRP) can encase beams to activate ALPs. The main function of this encasement is to help the beams bridge over lost columns by further enhancing the collapse prevention mechanisms such as catenary and flexural action. Thus, the external CFRP layer acts in a similar way to continuous reinforcement but provides more ductility and rotational capacity. Similarly, using of CFRP for retrofitting slabs can enhance a slab's load redistribution abilities.

In terms of strengthening, steel jacketing is most commonly used. Steel jacketing helps in increasing the serviceability of a member in near-field explosions. Less initial local damage can help reduce the overall subsequent damage in a building. In an attempt to also reduce the initial local damage endured by a member, some studies tried to use sacrificial cladding elements

made out of reinforced resin panels and insulation. A series of experiments to represent the impact of blasts on retrofitted members using resin panels and steel jacketing were conducted. From this experimentation, it was concluded that steel jacketing can lead to a 57.4% decrease in deformation when compared to un-retrofitted members. Moreover, reinforced resin cladding was found to offer a 66% decrease in deformation in a blast event when compared to un-retrofitted members. The beneficial effects of steel jacketing and sacrificial cladding are demonstrated in Fig. 31 since the retrofitted members endured much less damage than the unprotected ones. Retrofitting can be one of the most effective and practical measures that can be applied to existing structures to help reduce their risk of progressive collapse. This solution, however, might not be the most cost-effective for new structures that can implement more sustainable measures in their design.

Additional reinforcement. One of the most economical methods to reduce the risk of progressive collapse in reinforced concrete structures is the optimization of the reinforcement itself, which forms the tying elements in RC structures. Reinforcement can have major effects on the strength and ductility of concrete members. Thus, various researchers aimed to further understand those effects to ensure that the reinforcement capabilities are best employed. Typically, in beams, there are two types of reinforcement: longitudinal and transverse. Longitudinal reinforcement was found to have more impact on a member's progressive collapse resistance characteristics, such as rotational capacity and strength.

On the other hand, there are some limitations like:

- a) Although increasing longitudinal reinforcement enhances and triggers catenary action earlier as well as increases deformation capacity, it can lead to a reduction in load-bearing capacity
- b) Over-reinforcement can also lead to accelerated bending failure and earlier onset of catenary action
- c) Additional reinforcement might not always lead to increased capacity due to the premature failure that can happen in the bars in progressive collapse events before reaching the full expected capacity due to the sudden dynamic load application usually associated with such events



Fig. 31. Member retrofitting impact

Reinforcement is crucial for the behaviour of reinforced concrete structures, and research has been conducted to optimize it for progressive collapse-resisting mechanisms. However, most studies have used static or quasi-static loads due to spatial, time, and cost constraints, which may not accurately represent the dynamic effects of progressive collapse. Additionally, experiments have assumed extremely stiff end conditions, which may not be feasible in real-life structures. Therefore, further investigation is needed to consider all contributing factors and produce informed recommendations.

Non-structural measures. Designing a structure to have sufficient inherent robustness and resilience against all potential threats, for direct habitability purposes, can be extremely costly. Therefore, rather than robustness, collapse resistance can be seen as the goal of many mitigation methods, CoPs, standards and guidelines. As mentioned in Section 2, collapse-resistant structures can use non-structural members to help mitigate the risks of an abnormal loading event. Those measures can be divided into two main types. The first type helps ensure that a potential source of impact is not within a distance that can influence the building. An example of this kind of system is the pillar barriers, which prevent vehicles from being within a certain distance of a structure. The second type of system is based on the use of sacrificial energy-dissipating elements. In this system, sacrificial elements, such as energy-absorbing cladding, can be installed to dissipate energy from a potential threat, leaving the main structural system minimally damaged. Overall, this method aims to reduce the probability of initial local failure rather than prevent failure propagation.

9.2 Mitigation methods

The progressive collapse mitigation techniques/ devices that will be explored in this section are segmentation, structural fuses and energy absorption units.

Segmentation.

continuity can be the basis of collapse progression in many cases. This is due to the fact that when a section fails, and its loads get redistributed to neighboring sections, these sections might not be of adequate strength and capacity to carry the additional loading. This can lead to the failure of the neighbouring sections, causing additional successive damage until a considerably large area of a structure is damaged. This issue has two main solutions. The first, most commonly used one and currently recommended by the CoPs, is to ensure continuity but at the same time adequately design alternative load paths to ensure that they do not fail with the additional redistributed loads. This solution can result in extremely uneconomic designs, especially in structures with larger spans. The second method, segmentation, is still being developed to this date.

Segmentation mainly describes applying the concept of collapse isolation in a structure horizontally or vertically. Horizontal segmentation is usually applied in structures with a lower height-to-width ratio, such as bridges. Moreover, vertical segmentation is conceptually developed for structures with a high height-to-width ratio, such as high-rise structures. In both cases, a structure is strategically divided into sections, horizontally or vertically, based on predefined acceptable damage areas. The chosen sections are separated by elements that isolate the different parts of the structure. These elements are designed to either be weaker or stronger than the rest of the structure and mainly aim to prevent excessive load redistribution to neighboring sections and, consequently, damage.

Stronger isolating elements are typically considered in the form of strong floors and could be applied in vertical segmentation as illustrated in Fig. 32(a). This is since, in high-rise structures, one of the main causes of collapse progression, after the initial damage, is unaccounted for impact loading from falling debris, similar to WTC1 and 2 cases. The main function of strong floors is to try and dissipate as much kinetic energy as possible from the upper floors to mitigate the effects of the impact on subsequent floors. Strong floors should have considerable ductility and strength to ensure that they can undergo adequate deformation for energy dissipation purposes. This can be achieved by using thick reinforced concrete slabs coupled with energy absorption methods, as shown in Figs.32(b) and (c).

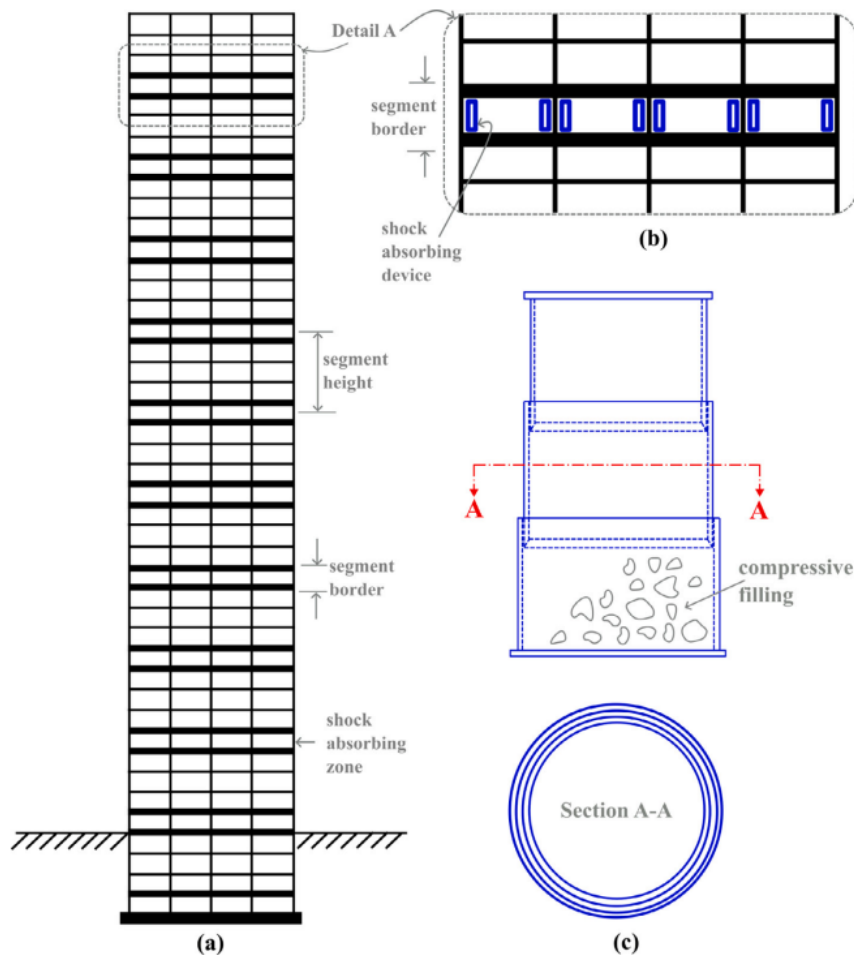
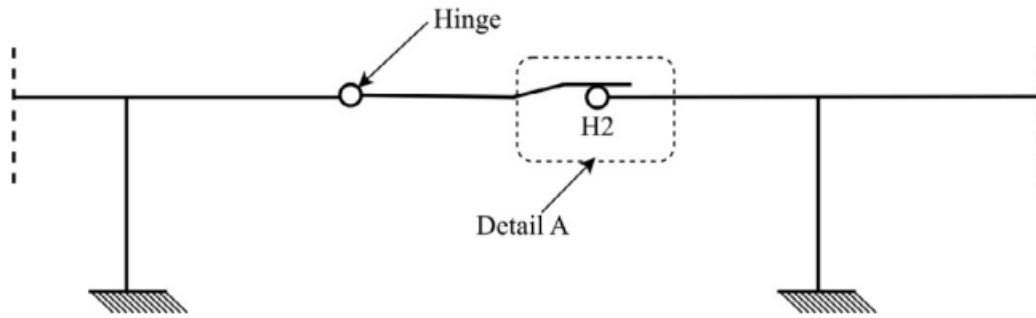


Fig. 32. Vertical segmentation in a high-rise structure: (a) vertically segmented structure, (b) shock absorbing zone, and (c) shock-absorbing device.

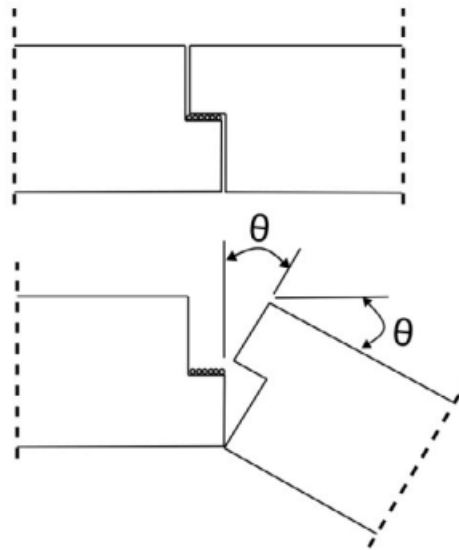
In terms of horizontal segmentation, weaker elements are usually considered since they can act as structural fuses. This conceptual design was successfully applied in seismic design. The application of structural fuses will be discussed in subsequent discussions. In terms of progressive collapse, weak elements can act as points of discontinuity since any additional loading might damage these elements and lead to their failure. This helps prevent load redistribution to neighboring sections beyond a certain limit. Other methods of implementing segmentation can be through utilizing expansion joints or hinges at the borders of segments depending on the nature of a structure.

The concept of segmentation using hinges was successfully applied in the Confederation Bridge. The Confederation Bridge is a 12.9 km highway bridge in Canada consisting of 43, 250

m main spans. Designing the bridge using the traditional ALP method would have led to an inefficient design with a dramatic cost increase. Thus, the bridge engineers decided to apply the concept of segmentation in its design. The most viable option was to have segmented sections in the bridge implemented through drop-in girders and hinges incorporated within every other span (Fig. 33). This ensures that in case of failure, drop-in girders will disengage and fall into the underlying watercourse, leaving independent, stable sections of the bridge behind.



(a) Hinge and drop-in girder utilisation



(b) Detail A: Disengagement of drop-in girder

Fig. 33. Confederation Bridge segmentation strategy

It is important to note that segmentation has only been studied conceptually and theoretically with very limited practical applications, making the Confederation Bridge the most prominent real-life example. Other examples where compartmentalization, or in other words, segmentation, has possibly helped prevent collapse propagation include the Pentagon Building in Virginia and Charles de Gaulle Airport Terminal in Paris. The Pentagon Building consists of three rings of buildings, each divided into five sections using expansion joints.

When the Pentagon was hit by a plane in 2001, one section of the outer rings was severely damaged. However, this damage did not propagate past the point of discontinuity, which is the expansion joint, to other neighboring sections. If continuity existed between the joints, collapse progression would have occurred since the impacted section of the structure was severely damaged. Regarding Charles de Gaulle Airport, the collapse was initiated by the failure of a portion of the roof due to poor workmanship. This collapse was stopped at the joints, which separated the collapsing section from the adjacent structure. Similar to the Pentagon building, in case this structure was continuous, it seems unlikely for the undamaged section to have sustained the additional induced forces from the collapse. This can be attributed to the construction deficiencies that were present within the adjacent sections as well. Although in both examples, segmentation was not used directly to address progressive collapse, these incidents portrayed the potential benefits of this method as a progressive collapse mitigation technique.

Despite the potential benefits of segmentation, concrete standard procedures for it have not yet been developed. Thus, this method should be studied in-depth as it could be very valuable to the field, especially if it is optimized economically and practically. To satisfy the CoPs while implementing this method efficiently, a structure can be designed to incorporate redundancy and load redistribution within the borders of its identified segments. This will ensure that a segment can withstand minor element losses without failure. However, if a certain loss limit is exceeded, the damage will not propagate to other segments of the structure.

Structural fuses. Structural Fuses are elements in a structure designed to endure most damage while keeping the rest of the structure undamaged. This is usually done by ensuring that the fuse elements have sufficient ductility and stiffness to be able to deform plastically while keeping the rest of the structure within the elastic deformation limits. This mainly aims to limit the damage to only the sacrificial fuse elements. Moreover, structural fuses should be designed to be easily replaced or repaired. This helps ensure that damaged members can be replaced in a rapid and economical manner, ensuring limited cost and interruption to the operation and functionality of a structure. To maintain the stability of a structure after damage sustained by fuses following an extreme event, they can be designed to have clear limits for the acceptable loss in strength levels that they can endure as they undergo deformation. The ductility utilization and undergoing plastic deformation should not cause more than a 20% degradation in the resistance of the fuse elements to ensure the stability of the rest of the structure. Alternatively, the remaining sections of the structure can be designed to be structurally independent if the fuses are damaged.

To date, most research and applications of structural fuses have been focused on seismic design in both buildings and bridges. For example, some studies investigated the utilization of shear keys in bridges as sacrificial elements to help limit transverse displacements in the

superstructure and control damage to the substructure. In terms of buildings, some of the proposals included the incorporation of fuses within elements, such as masonry infill walls and concrete filled steel members, or structural systems, such as H-frame systems.

The implementation of structural fuses in progressive collapse design has not yet been adequately studied. However, it can potentially be a viable option in the mitigation of progressive collapse. Some of the aspects that need to be investigated in such a method include the required initial and residual stiffness, ductility, and strength levels. Additionally, the impact of fuse damage and deterioration of strength on the rest of the structure should be investigated. Potentially, fuses can be applied in progressive collapse design to utilize the advantages of both continuity and segmentation. For example, for small initial failures, fuses could provide some degree of continuity to help in load redistribution and the mobilization of alternative load paths. However, for larger initial failures, where collapse progression is inevitable, fuses can enable the implementation of segmentation in a structure to limit collapse propagation to other sections of the structure..

Energy absorption units. Energy-based progressive collapse analysis reveals that energy in such events is divided into kinetic and potential forms. Kinetic energy is involved in debris impact, while potential energy is stored in static storeys before being impacted by falling material. Kinetic energy from moving particles is typically dissipated through buckling or deformations. If a collapsing storey's kinetic energy exceeds the energy dissipated by crushing a subsequently impacted story, progressive collapse cannot be halted.

Furthermore, from the column removal progressive collapse analysis using the implicit dynamic analysis, it was deduced that beams critically contribute to the energy redistribution in a building during a progressive collapse. Additionally, the buckling of various members in the structure can significantly assist in further energy dissipation. This led to highlighting the importance of investigating buckling from an energy rather than force-based perspective and the effect on member strength that has over time. Moreover, from investigating the WTC incident, some studies found that the damage and plastic deformations endured by exterior columns led to the dissipation of only around 6.7% of the kinetic energy involved with the impacting plane and the rest of the energy was dissipated by the crushing and deformation of internal sections of the structure. Thus, this indicates that the collapse of the rest of the structure happened due to the kinetic energy associated with the debris free-falling through the storeys of the building rather than from the initial impact itself.

From the previous energy-based research in progressive collapse, one main thing can be concluded, which is that energy dissipation and absorption can be one of the key mitigation techniques. This can be achieved through two main ways: utilizing buckling and deformation capacities of members and through installing additional energy absorption devices. In the case of the WTC, the plastic reserve of the towers was enough to stop the plane from further propagating. Still, it was not enough to halt the impact of falling debris from damaging lower storeys and thus causing further collapse propagation. Thus, some studies proposed the installation of highly ductile, energy absorbing devices. The concept of the proposed devices is to undergo crushing to dissipate the maximum amount of energy from the collapse of preceding storeys to halt collapse progression rather than prevent its initiation. To do so, the devices are to be designed similarly to a 'stocky column' with enough cross-sectional area to

ensure compressing and crushing rather than buckling. Fig. 34 shows typical energy absorption lattice structures similar to the original proposed aluminum design by *Zhou and Yu*. He argued that their proposal is much more cost-effective than a proposal aimed at completely preventing collapse, as this will require achieving impractical levels of strength and ductility in the overall structure.

The application of energy absorption devices in high-rise structures could have major benefits. However, the main issue with such devices is capital cost. This is mainly because although the costs and losses associated with progressive collapse events are extremely high, they are very rare events. Therefore, it could be infeasible to allocate large budgets to mitigation devices that will most likely never be used in the lifetime of a structure. However, because of their potential advantages, these suggested devices can be improved in terms of size, design, and material to provide a practical, dependable, and affordable alternative.

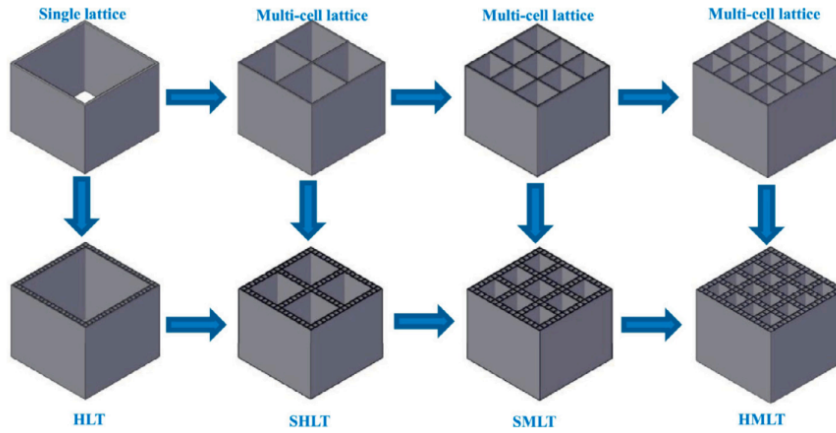


Fig. 34 Typical energy absorption lattice structures (HLT: hierarchical lattice tube, SHLT: super hierarchical lattice tube, SMLT: super multi-cell lattice tube and HMLT: hierarchical multi-cell lattice tube)

9.3 Summary

From this section, as summarized in Table 7, it can be concluded that there are numerous proposals for collapse prevention and mitigation techniques. The primary issue with most of these approaches is that they are either not practical or adaptable enough to be used regularly in the industry or need additional research and validation. For a proposed method to be widely accepted, it must fulfil the following criteria: functionality, cost-effectiveness, sustainability, applicability to various structures, thorough testing, and codifiability. To meet these criteria, various concepts may be implemented simultaneously to ensure effectiveness while eliminating high additional costs.

Moreover, as was discussed previously in Section 4, current codes of practice have various gaps associated with them. Some of these gaps, specifically related to the proposed mitigation and prevention methods, have been addressed by researchers in various ways, as summarized within this section. In order to address some of the codes' gaps while ensuring efficient and economical design, Fig. 35 proposes a possible framework that can be followed in the progressive collapse design of most buildings. This framework aims to ensure code compliance by incorporating the main guidance provided by the discussed codes while addressing several identified gaps, through incorporating various research proposals.

Table 7-
Proposed methods for progressive collapse resistance following local failure

Type	Proposed method	Description	Advantages	Disadvantages
Mitigation	-Segmentation	-Some failure is allowed	-Economic design	-Some sections of the structure are allowed to fail
	-Structural fuses -Energy absorption devices	-Overall structural integrity is prioritised		
Prevention	-Steel cable system -Member retrofitting	-All failure should be prevented -Structure is designed to bridge over failed elements	-Additional failure is not anticipated	-Uneconomic design -Infeasible for longer spans and irregular structures
	-Additional reinforcement -Seismic design -Bracing systems -Dampers			

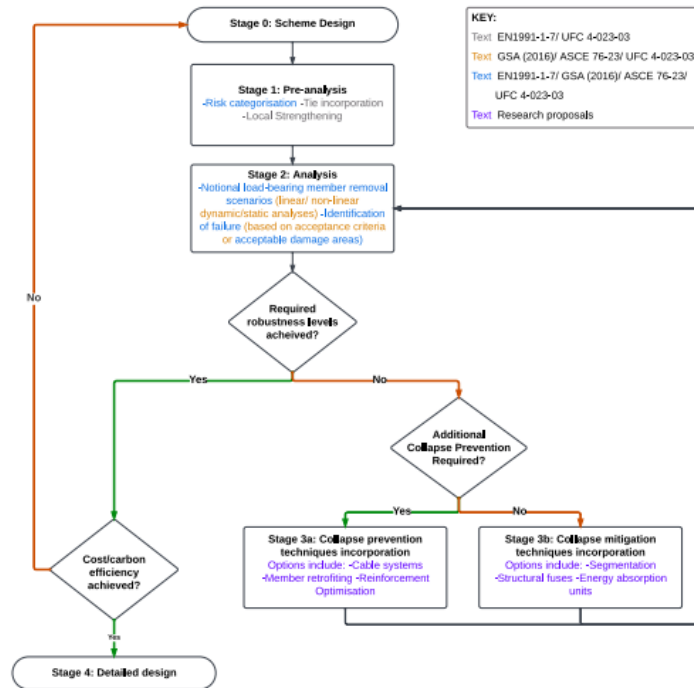


Fig. 35. Proposed framework for progressive collapse design based on current knowledge

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