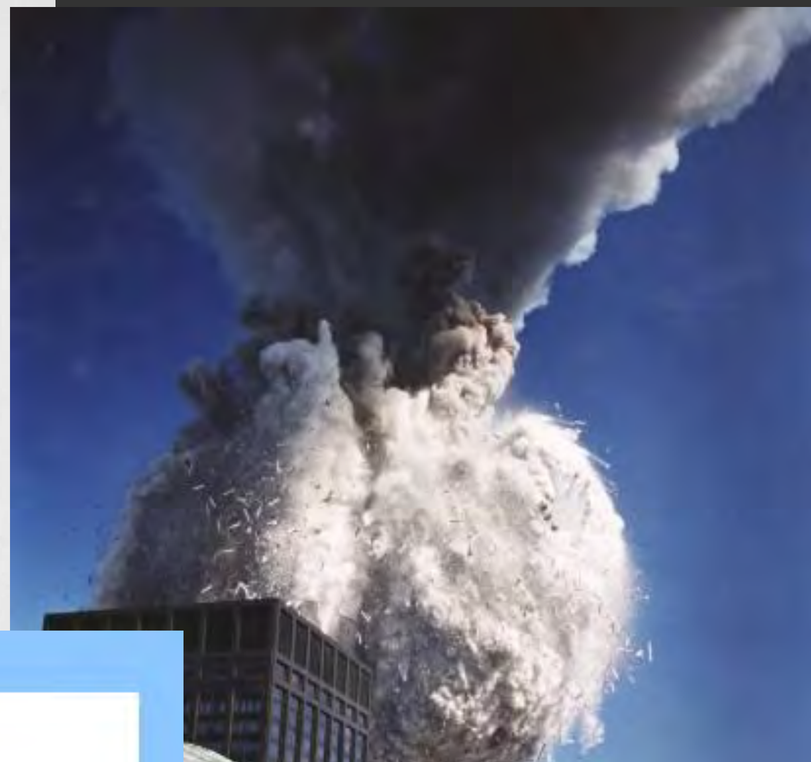
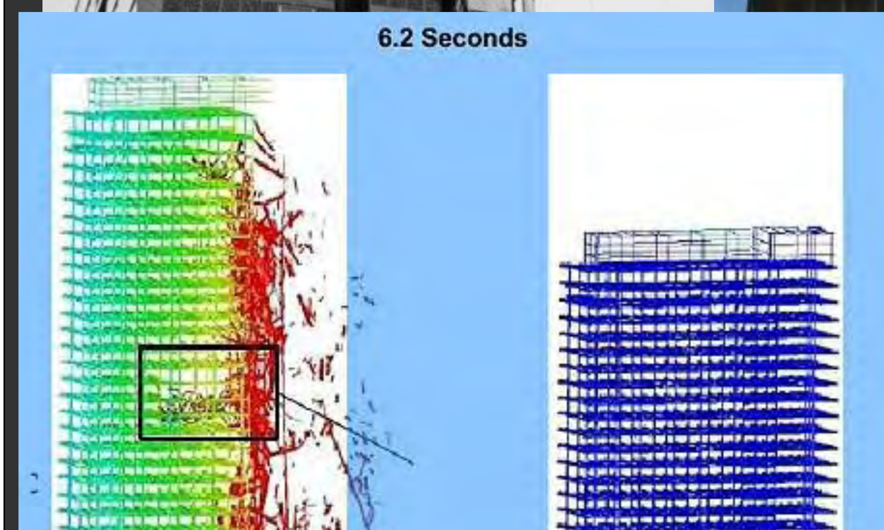
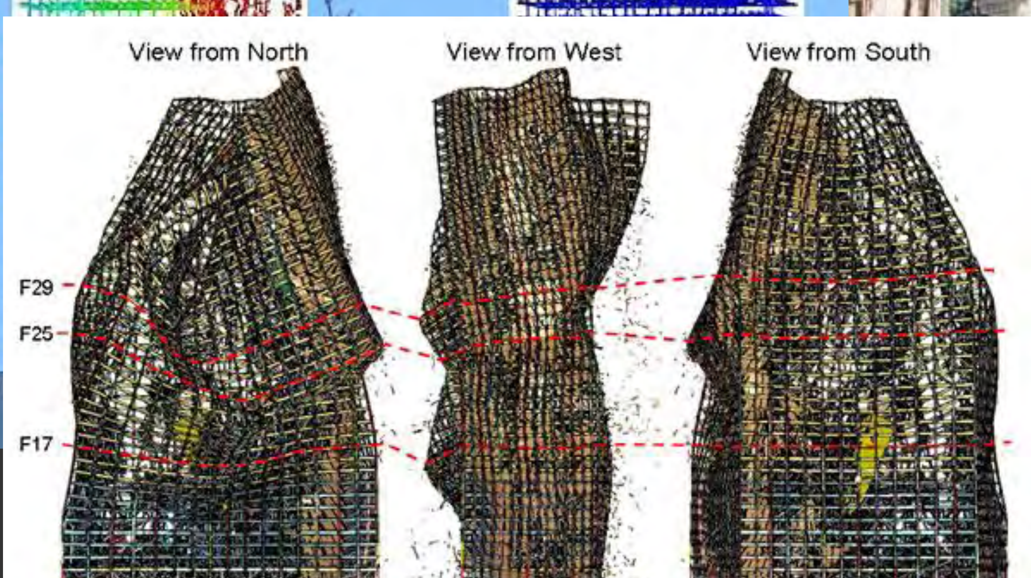
Progressive Collapse Analysis:

A Case Of A Furnance Steel Tower In Dubhai







Thank yous

I would like to thank for all their shared knowledge, support, help and patience:

- *my diploma thesis lead instructor and faculty advisor Dr. Dimitris Sophianopoulos*
- *my diploma thesis advisor Dr. Panagiotis Tsopelas*
- *my parents, Anastasia and Stanley Karagiannis*
- *my girlfriend and Architect Nikoletta Lekka*
- *my friend and colleague Dr. Polinikis Vazouras*
- *my friend and colleague Spyridoula Papathanasiou*

This page intentionally left blank.

Progressive Collapse Analysis:
A Case Of A Furnance Steel Tower In Dubhai

Diploma Thesis

By

Nicolas Karagiannis

Faculty Advisor: Dr D. Sophianopoulos

B

This page intentionally left blank.

ABSTRACT

This document is intended to provide owners and practicing engineers with current information on the best available design methods to reduce the risk of progressive collapse of buildings in the event of abnormal loading. This script includes an analysis of an acceptable risk approach to progressive collapse, which involves defining the threat, event control, and structural design to resist postulated event. Different ways for reducing risk for new and existing buildings are being mentioned. All new discussions and techniques are provided about the new design methods used to avoid a buildings progressive collapse. These are the indirect method (giving sufficient tie forces), the specific local resistance method (designing key elements to withstand abnormal loads), and the alternate load path method (allowing for redistribution of load in the event of the loss of a key member). Other approaches concerning different structural materials are mentioned. The methodology for evaluating and mitigating progressive collapse potential in existing buildings is also discussed. Appendix A presents a worldwide review of progressive collapse provisions in various national design standards. Appendix B identifies knowledge gaps related to progressive collapse that require research. This document is not intended to provide a step-by-step design guidance for practicing engineers, however the existing international design standards are summarized in Appendix A.

This page intentionally left blank.

PREFACE

This document is prepared collecting current state of the Art information on the subject of progressive collapse and the best practices to prevent it. A detailed Effort had been made by NIST in response to one of the recommendations from the July 2002 industry workshop on prevention of progressive collapse, which was held in Chicago, Illinois.Another document was facilitated by the Multihazard Mitigation Council (MMC) of the National Institute of Building Sciences. The MMC contracted with the three principal authors to prepare an initial draft and organized a workshop in February 2004 to solicit public comments on the initial draft document.

Using the information and papers from the NIST Workshops, other papers and further research this paper was formed collecting further information and being targeted mostly on steel and concrete frames.

Disclaimer: Certain trade names or company products are mentioned in the text to specify adequately certain products. In no case does identification imply recommendation or, nor does it imply the product is the best available for the purpose.

This page intentionally left blank.

TABLE OF CONTENTS

Abstract... C
Preface D
Table of Contents... E
List of Acronyms and Abbreviations... F
……G
Chapter 1Progressive Collapse ... 1
1.1Purpose and Scope... 2
1.2Typology of progressive collapse... 2
1.2.1Pancake-Type Collapse... 2
1.2.2Zipper-Type Collapse... 2
1.2.3Domino-Type Collapse 2
1.2.4Section-Type Collapse 2
1.2.5Instability-Type Collapse... 2
1.2.6Mixed-Type Collapse... 2
1.3Classification 2
1.4Collapse Promoting Features... 2

Chapter 2 Acceptable Risk Bases... 5
2.1 Introduction 5
2.2 Fundamentals of Risk Assessment for Natural and Man-Made Hazards ... 6
2.2.1 Perception and Analysis of Risk... 6
2.2.2 Mathematical and Decision Analysis Tools 8
2.2.3 Measurement of Risk... 9
2.3 Hazards ... 10
2.3.1 Aircraft Impact 11
2.3.2 Design or Construction Error 11
2.3.3 Fire 11
2.3.4 Gas Explosions 12
2.3.5 Overload Due to Occupant Misuse... 13
2.3.6 Transportation and Storage of Hazardous Materials 14
2.3.7 Vehicular Collision... 14
2.3.8 Bomb Explosions 14
2.3.9 Summary and Critical Appraisal 16

2.4 Building Vulnerability... 17
2.5 Design to Reduce Risk of Progressive Collapse... 18
2.5.1 Performance Objective... 18
2.5.2 Performance Matrix... 18
2.5.3 Specific Design Strategies... 18
(i) Event control ... 19
(ii) Structural engineering - probabilistic bases... 19
(iii) Evaluation of existing construction... 24
2.6 Concluding Remarks... 25

Chapter 3 Means for Risk Reduction ... 27
3.1 Introduction... 27
3.2 Project Viability Assessment... 27
3.2.1 Threat Evaluation ... 28
3.2.2 Means to Reduce Exposure and Threat... 28
3.2.3 Evaluation of Consequences of Failure... 29
3.2.4 Evaluation of Alternative Means to Reduce Consequences... 29
3.2.5 Determination of Qualification of Structure for Upgrade Consideration... 30
3.3 Assessment of Potential For Progressive Collapse... 30
3.3.1 Site Evaluation ... 30
3.3.2 Structural Systems Evaluation... 30
3.4 Design approaches ... 30
3.4.1 Goals... 30
3.4.2 Siting, Plan Layout, and Hazard Consideration ... 31
3.5 Design Principles... 33
3.5.1 Failure Propagation ... 33
3.5.2 Structural Robustness... 34
3.5.3 Connection Strength... 36
3.5.4 Key Elements ... 36
3.5.5 Alternate Load Paths ... 37

Chapter 4 Indirect and Direct Design Approaches... 39
4.1 Introduction... 39
4.2 Indirect Method... 39
4.2.1 General ... 39
4.2.2 Tie Requirements ... 40
(i) General ... 40
(ii) Reinforced concrete... 40
(iii) Precast concrete... 41
(iv) Steel structures ... 42

(v) Masonry structures 42
(vi) Existing buildings... 42
4.2.3 Seismic Details... 42
4.3 Direct Methods ... .43
4.3.1 General 43
(i) Specific local resistance method ... .43
(ii) Alternate load path method ... .43
4.3.2 Analysis Techniques... 44
(i) Small deformation and finite deformation ... .44
(ii) Linear elastic constitutive models ... .44
(iii) Inelastic, rate dependent, and rate independent models ... .44
(iv) Material level versus structural level... 45
(v) Stability analysis... 45
4.3.3 Performance Criteria ... .46
4.3.4 Specific Local Resistance... 47
(i) Key elements... 47
(ii) Transfer structures... 48
(iii) Ductile detailing 48
(iv) Analysis methods ... .48
4.3.5 Alternate Load Path Analysis... ... 49
(i) Elastic static... 50
(ii) Inelastic static... 51
(iii) Elastic dynamic 52
(iv) Inelastic dynamic... 52
4.3.6 General Purpose Approach for Progressive Collapse Potential 53
4.4 Summary... 53

Chapter 5 Practical Design To Prevent Progressive Collapse 55
5.1 introduction... 56
5.2 Design Guidance for Different Structural Systems....56
5.2.1 General Beneficial Features of Structures and Foundations ... .56
(i) Structural features... 56
(ii) Compartmentalization ... .56
(iii) Foundation design 57
5.2.2 Reinforced Concrete Structures... 57
(i) Beam design ... .57
(ii) Column design... 58

(iii) Slab design 58

(iv) Wall design... 58

5.2.3 Steel Structures... 59

(i) Beam design ... .59

(ii) Column design... 60

(iii) Slab design 60

5.2.4 Masonry Structures... 60

(i) General features... 61

(ii) Reinforced masonry ... .61

(iii) Unreinforced masonry... 61

5.2.5 Large Precast Concrete Panel Construction ... .61

(i) Precast concrete floor systems... 61

(ii) Tilt-up construction...61

5.2.6 Post-tensioned (Unbonded) Construction... 62

(i) General 62

(ii) Beam design ... .62

(iii) Slab design 62

5.3 Considerations for Upgrading Existing Buildings... 62

5.3.1 Local Strengthening to Prevent Failure Initiation ... .62

(i) Upgrade vulnerable elements for specific threats... 63

(ii) Upgrade vulnerable elements for non-specific threats ... .63

(iii) Constraints originating from existing structural system..63

5.3.2 Retrofit Upgrade of Specific Materials ... .64

(i) Means for upgrading ... .64

(ii) Reinforced concrete and masonry structures... 65

(iii) Steel elements and structures..66

Chapter 6 Case Study of a furnance steel tower 68

6.1 Introduction... 68

6.2 Steel tower and model description... 70

6.3 Time history analysis results... 73

6.4 Conclusions... 80

Chapter 7 Summary 81

7.1 Introduction... 81

7.2 Definition... 81

7.3 Reducing Risk... 81

7.4 Design Methods... 82
7.5 General Guidelines....82
7.6 Existing Buildings....82
7.7 Design Guides....83

Chapter 8 References... 86
Appendix A Comparison of Design Standards... 91
A.1Genesis of Progressive Collapse Provisions... 91
A.1.1Ronan Point....91
A.1.2 1968 U.K. Standards ... .91
A.1.3Initial Approach for Design Criteria: ... .92
A.1.4Operation Breakthrough....93
A.1.5U.S. Approach....95
A.1.6Other Early Studies ... .95
A.1.7Present Interest ... .96
A.1. 8 Scope....96
A.2. Approaches to Mitigating Progressive Collapse... 97
A.2.1 Requirements for New Buildings....97
A.2.2 Requirements for Existing Buildings ... .97
A.3Summary of Standards on Progressive Collapse 97
A.3.1 British Standards ... .97
A.3.2Eurocode... 98
A.3.3NationalBuilding Code of Canada... 98
A.3.4Swedish Design Regulations....98
A.3.5ASCE 7... 98
A.3.6ACI 318....98
A.3.7New York CityBuilding Code....98
A.3.8Department of Defense Unified Facilities Criteria... 99
A.3.9Interagency Security Committee....99
A.3.10General Services Administration... 99
A.4 Comparison of Progressive Collapse Provisions ... .99
A.4.1Provisions Compared ... .99
A.4.2Definition of Progressive Collapse, Local Collapse, and Structural Integrity ... .99
A.4.3Threshold... 103
A.4.4General strategy... 107
A.4.8Continuity, Ductility, and Other Details... 154

A.4.9Existing Buildings...156

A.5Summary...158

A.5References...159

Appendix B Research Needs...165

B.1Connections Details...165

B.1.1 Analytical Representation—Explicit Modeling ...165

B.1.2 Analytical Representation—Simplified Modeling...165

B.1.3 Investigation of Properties of Connections in Existing Buildings...166

B.1.4 Component Testing—Develop Force-Displacement and Moment-Rotation Properties...166

B.1.5 Evaluate Whether Seismic Connection Requirements Improve Resistance to Progressive Collapse ...166

B.2Failure Criteria...167

B.2.1 Acceptance Criteria for Different Analysis Methods...167

B.2.2 Applicability of Demand Capacity Ratio to Define Failure for Linear Analysis Methods...167

B.2.3 Applicability of Limiting Ductility or End Rotation to Define Failure...167

B.3 Flat Slab Construction...167

B.3.1 Development of Catenary Slab Resistance...168

B.3.2 Testing of Full Scale Flat Slab Structure...168

B.4 Fire induced Progressive Collapse...168

B.4.1 Coupling Thermal, Mechanical, and Structural Analysis...168

B.4.2 Material Behavior at Elevated Temperatures...168

B.4.3 Fire Exposure...168

B.4.4 Investigation of Case Histories...169

B.5 Development of Impact Factors for Slab “Pancake” Failure...169

B.6 Quantification of Risk Associated with Different Upgrade Approaches...169

B.7 Performance of Prescriptive Design Requirements...169

B.8References...170

This page intentionally left blank.

List of Acronyms and Abbreviations

Acronyms

ACI	American Concrete Institute
AGA	American Gas Association
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
ASCE	American Society of Civil Engineers
BS	British Standards
CFRP	Carbon fiber-reinforced polymer
CMU	Concrete masonry unit
CSA	Canadian Standards Association
D	Dead load
DCR	Demand-Capacity Ratio
DOD	Department of Defense
DTRA	Defense Threat Reduction Agency
FBI	Federal Bureau of Investigation
FE	Finite element
FEMA	Federal Emergency Management Agency
FRP	Fiber-reinforced polymer
HLOP	High level of protection
GSA	General Services Administration
ISC	Interagency Security Committee
L	Live load
LLOP	Low level of protection
LRFD	Load and resistance factor design
MLOP	Medium level of protection
NBCC	NationalBuilding Code of Canada
NTSB	National Transportation Safety Board
NYCDOB	New York City Department of Buildings
OPSO	Office of Pipeline Safety Operations

PCI	Precast/Prestressed Concrete Institute
S	Snow load
T	Load due to temperature effects
UFC	Unified Facilities Criteria
U.K.	United Kingdom
U.S.	United States
VLLOP	Very low level of protection
W_n or W	Wind load

Unit Abbreviations

d	day
ft	foot
f_y	specified yield strength
h	hour
in	inch
kg	kilogram
kip	1000 lb
kN	kilonewton
kPa	kilopascal
ksi	1000 psi
lb	pound
m	meter
mi	mile
mph	miles per hour
psf	pounds per square foot
psi	pounds per square inch
s	second
yr	year

Chapter 1
INTRODUCTION

1. PROGRESSIVE COLLAPSE

The term “progressive collapse” has been used to describe the spread of an initial local failure in a manner analogous to a chain reaction that leads to partial or total collapse of a building. The underlying characteristic of progressive collapse is that the final state of failure is disproportionately greater than the failure that initiated the collapse. ASCE Standard 7-05 defines progressive collapse as “the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it” (ASCE 2005). The disproportionality refers to the situation in which failure of one member causes a major collapse, with a magnitude disproportionate to the initial event. Thus, “progressive collapse” is an incremental type of failure wherein the total damage is out of proportion to the initial cause. In some countries, the term “disproportionate collapse” is used to describe this type of failure. Based on the above description, it is proposed that the professional community adopt the following definition, which is based largely on ASCE 7-05:

> ***progressive collapse***—the spread of local damage, from an initiating event, from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it; also known as disproportion at collapse.

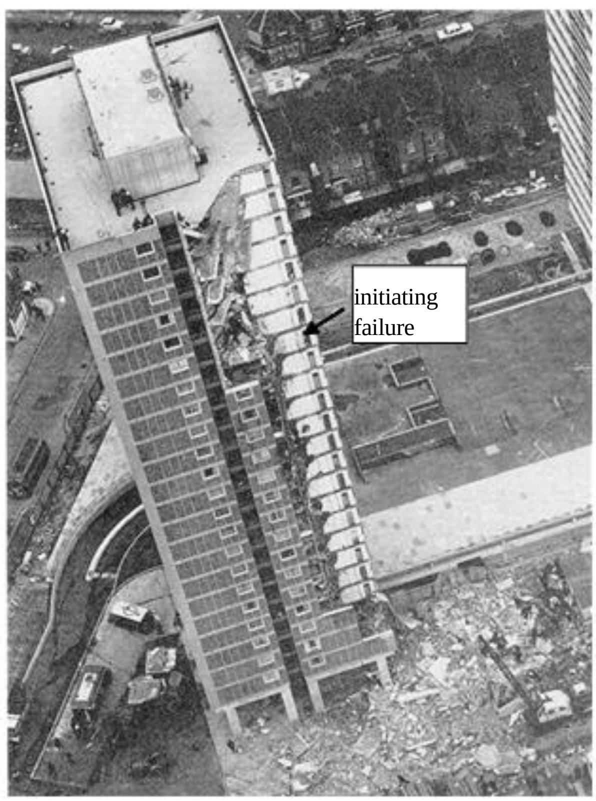


Figure 1-1 Ronan Point collapse: a gas explosion on the 18th floor resulted in a progressive collapse.

1.1 PURPOSE AND SCOPE

Immediately following the Ronan Point collapse, some countries, such as the U.K. and Canada adopted some form of regulatory standards to address prevention of progressive collapse. In 1976 the U.K. building regulations required that buildings not sustain collapse to an extent disproportionate to the initiating failure. The regulations required that buildings be designed to resist disproportionate failure by tying together building elements, adding redundant members, and providing sufficient strength to resist postulated abnormal loads. These requirements are considered to produce more robust structures, that is, structures that are strong, ductile, and capable of redistributing loads (refer to Section 3.5.2 for a discussion of factors that affect structural robustness).

In the 1980s, design standards in the U.S. began to incorporate requirements for “general structural integrity” to provide nominal resistance to progressive collapse (ANSI 1982). Structural integrity was to be achieved by providing continuity, redundancy and ductility in structures. At present, U.S. model building codes and standards do not include specific provisions to provide resistance against progressive collapse. Some materials design standards, however, such ACI 318 (ACI 2005) and the PCI guide for precast concrete bearing wall buildings (PCI 1976), include provisions for minimum levels of structural integrity. Although historical data indicate that the risk of progressive collapse in buildings is very low, loss of life and severe injuries would be significant when a fully occupied multi-story building sustains a large partial or total collapse. As a result of recent terrorist attacks on buildings throughout the world, particularly U.S. owned and occupied buildings, several U.S. government agencies with large construction programs have developed their own design requirements (GSA 2003; DOD 2005) to provide resistance against progressive collapse. Each agency, however, with its own mission, has adopted different performance objectives for buildings subjected to abnormal loads. Furthermore, the design approach to provide resistance to progressive collapse is not standardized among these documents. In the private sector there is, however, a diverse range of professional opinion regarding the extent and nature of changes to present practices that may be warranted to enhance the resistance of buildings to progressive collapse. A consensus has yet to be reached on the thresholds to delineate when design against progressive collapse needs to be considered and what level of resistance is acceptable.

The design of a building to resist progressive collapse may require analytical approaches that are not used in routine design. The purpose of this document is to provide the “best practices” for design professionals to reduce the likelihood of progressive collapse of buildings in the event of abnormal loading. It is not intended as a design standard. Guidance is provided that is based on existing knowledge and includes input from design professionals with a broad range of interests. This document addresses design of new buildings and upgrading of existing buildings. It does not address wood or cold-formed steel, low-rise construction.

In chapter 6 there is a detailed analysis of a steel furnance tower in Dubhai being forced with seismic activity and getting results on how it withstands progressive collapse. Appendix A presents a detailed worldwide review of progressive collapse provisions in various national design standards. Appendix B identifies knowledge gaps related to progressive collapse that require research. The report concludes with Appendix C, which provides case studies of progressive collapses. This document is not intended to provide step-by-step design guidance for practicing engineers, but it is intended to acquaint engineers with considerations involved in designing buildings for resistance to progressive collapse. Applicable design procedures are available as indicated in Appendix A.

1.2 TYPOLOGY OF PROGRESSIVE COLLAPSE

Progressive collapse of structures is characterized by a disproportion in size between a triggering event and the resulting collapse. Although the disproportion between cause and effect is a defining and common feature, there are various differing mechanisms that produce such an outcome. The amenability to theoretical treatment, approaches for quantifying indices, and possible or preferable countermeasures can vary accordingly. It thus seems useful to distinguish and describe the different types of progressive collapse and to attempt a classification on that basis. The term propagating action, used in the subsequent discussion, refers to the action that results from the failure of one element and leads to the failure of further similar elements. A typology and classification of progressive collapse of structures is developed that is founded on a study of the various underlying mechanisms of collapse. Six different types and four classes are discerned, the characteristic features of each category are described and compared, and a terminology is suggested. On this basis, the theoretical treatment of progressive collapse and the development of countermeasures are facilitated because they differ for different types of collapse. Some conclusions drawn here concern analogies that should be pursued further, collapse-promoting features, and possible countermeasures.

1.2.1 Pancake-type collapse

This type is exemplified by the collapse of the World Trade Center (WTC) towers. The impact of the airplanes and the subsequent fires initiated local failures in the areas of impact. The ensuing loss in vertical bearing capacity was limited to a few stories but extended over the entire cross section of the respective tower [1, 2]. The upper part of the structure started to move downwards and accumulated kinetic energy. The subsequent collision with the lower part of the structure, which was still intact, caused large impact forces which were far beyond the reserve capacities of the structure. This in turn led to the complete loss of vertical bearing capacity in the area of impact. Failure progressed in the same manner and led to a total collapse. A pancake-type collapse exhibits the following features:

- initial failure of vertical load-bearing elements
- partial or complete separation and fall, in a vertical rigid-body motion, of components
- transformation of potential energy into kinetic energy
- impact of separated and falling structural components on the remaining structure
- failure of other vertical load-bearing elements due to the impact loading
- collapse progression in the vertical direction

1.2.2 Zipper-type collapse

For the design of cable-stayed bridges, the PTI Recommendations [3] require that the sudden rupture of one cable shall not lead to structural instability and specify a corresponding load case “loss of cable”. Such requirement is intended, among other things, to prevent a zipper-like collapse initiated by the rupture of one cable and propagating by overloading and rupture of adjacent cables. Such failure is visible in the movie of the collapse of the original TacomaNarrowsBridge. After the first hangers of that suspension bridge snapped due to excessive wind-induced vibrations of the bridge girder, the entire girder peeled off and fell. A similar kind of failure can be envisioned in anchored retaining walls where a progressive collapse is possibly triggered by the failure of one or a few anchors [4]. Zipper-type collapse is not conditioned, however, on the initial failure of tension elements. Before identifying further possible cases, it is attempted to specify some characteristics of this kind of progressive collapse. A zipper-type collapse exhibits the following features:

- initial failure of one or a few structural elements
- redistribution of forces carried by these elements in the remaining structure
- impulsive loading due to the suddenness of the initial failure
- dynamic response of the remaining structure to that impulsive loading

- due to the combined static and dynamic effects, a force concentration in and failure of elements which are similar in type and function to and adjacent to or in the vicinity of the initially failing elements
- collapse progression in a direction transverse to the principal forces in the failing elements

1.2.3 Domino-type collapse

A trail of dominoes collapses in a fascinating chain reaction if one block falls at the push of a finger. The mechanism behind this type of collapse is as follows:
- initial overturning of one element (i.e., of one domino block),
- fall of that element in an angular rigid-body motion around a bottom edge,
- transformation of potential energy into kinetic energy,
- lateral impact of the upper edge of that element on the side face of an adjacent element; the horizontalpushing force transmitted by that impact is of both static and dynamic origin because it results from both the tilting and the motion of the impacting element,
- overturning of the adjacent element due to the horizontal loading from the impacting element,
- collapse progression in the overturning direction.

The occurrence and importance of impact forces suggests similarity with a pancake-type collapse. On the other hand, the principal forces in the failing elements are orthogonal to the direction of failure propagation and it is a parallel load-transfer system (at least before the onset of collapse)—two properties that are shared by a zipper-type collapse (although in that case they also apply to the system after the onset of collapse). It thus appears reasonable to distinguish the domino-type collapse from the previously discussed types. Its distinguishing features are the overturning of individual elements and the fact that the forces that cause the next element to fail (i.e., the propagating action) do not act in the direction of the principal forces transmitted by that element prior to onset of collapse. The collapse thus explores a particular weakness of the system towards forces other than the principal forces.

1.2.4 Section-type collapse

A beam under a bending moment or a bar under axial tension is considered. When a part of the respective cross section is cut, the internal forces transmitted by that part are redistributed into the remaining cross section. The corresponding increase in stress at some locations can cause the rupture of further cross sectional parts, and, in the same manner, a failure progression throughout the entire cross section. While this kind of failure is usually not called a progressive collapse (but fast fracture), it is useful to include it in this description in order to possibly exploit similarities and analogies.

When comparing to the previously discussed types of collapse, a section-type collapse appears similar to a zipper-type collapse. Indeed, the same list of features applies when the terms “cross section” and “part of cross section” are substituted for the terms “structure” and “element,” respectively. The main difference is that a cross section is amorphous and homogeneous whereas a system, for instance a cable-stayed bridge, is structured, i.e., it consists of discrete elements of possibly different properties. Still, the similarities might be strong enough to apply by analogy methods for treating section failure, and in particular fracture mechanics, to zipper-type collapse

1.2.5 Instability-type collapse

Instability of structures is characterized by small perturbations (imperfections, transverse loading) leading to large deformations or collapse. Structures are designed such that instability will not normally occur. The failure of a bracing element due to some small triggering event, however, can make a system instable and result in collapse. This could apply to truss or beam structures where bracing elements are used to stabilize bars or cross-sectional elements in compression. Another example is the failure of a plate stiffener leading to local instability and failure of the affected plate, and possibly to global collapse. In any case, such incidents exhibit the defining characteristic of progressive collapse, namely a small triggering event resulting in widespread collapse. An instability-type collapse exhibits the following features:

- initial failure of elements which stabilize load-carrying elements in compression,
- instability of the elements in compression that cease to be stabilized,
- sudden failure of these destabilized elements due to small perturbations,
- failure progression.

The progression of failure can vary. If the element firstly affected by destabilization is one of a fewprimary components, say, the leg of a truss tower, complete collapse can ensue immediately withoutcascading failure of similar, consecutively affected elements (like in the other types of collapse discussed before). Although strong disproportion between cause and effect is apparent in such an event, it might befelt that this is not a progressive collapse. But then the definition of progressive collapse would have to be expanded by adding the feature of cascading failure of similar, consecutively affected elements. On the other hand, the element firstly affected by destabilization can also be a relatively small component, and failure can progress as a consecutively occurring stability failure of similar elements.

1.2.6 Mixed-type collapse

The types of collapse considered so far are relatively easily discerned and described. Some collapses that have occurred in the past do not neatly fit into these categories, however: In certain kinds of structures, particularly in buildings, it even seems possible that features of the fourbasic categories pancake-type, zipper-type, domino-type, and instability-type collapse combine and contribute to failure progression. In such a scenario, a feature of zipper-type collapse could consist in the buckling of columns in a continuous frames tructure leading to overloading and buckling of adjacent columns. Because failure progression also tends to reduce stiffness and bracing in a consecutive manner, the propagating action in this example can partly consist of destabilization, a feature associated within stability-type collapse. Such mixed-type collapses are less amenable to generalization because the relative importance of thec ontributing basic categories of collapse can, in principle, vary. Nevertheless, further study might lead to the definition of other well defined types of collapse.

1.3 CLASSIFICATION

The preceding discussion of types of collapse and their respective features allows further generalization and classification. Both zipper-type and section-type collapses are most strongly characterized by the redistribution of forces carried by failing elements in the remaining structure. They are thus subsumed under one class of collapse which is called the redistribution class. Pancake-type collapse and domino type collapse, in comparison, have fewer features in common, but in some important respects they are similar. In both, a substantial amount of potential energy is transformed into kinetic energy during the fall or overturning of elements and subsequently reintroduced into the structure. The reintroduction of energy occurs more or less abruptly. The latter two types of collapse or thus combined in one class of collapse which is called the impact class. This term is chosen for convenience and also refers to the abrupt deceleration of overturning elements in a domino-type collapse.

The instability-type collapse forms one class on its own. It is characterized by destabilization of load-carrying elements in compression through discontinuance of stabilizing elements. The transformation of potential energy plays a role but in a different way than for an impact-class collapse. Finally, mixed-type collapses also form one class, for which, however, it is difficult to identify general properties other than the fact that features of various types of collapse interact and combine to produce a collapse.

1.4 COLLAPSE-PROMOTING FEATURES

1.4.1 Dynamic action, force concentration, brittle material behavior

Despite the differences discerned in the preceding discussion, there are also some collapse-promoting features that are shared, to varying extents, by the various types of collapse. Although dynamic action is indispensable only for the explanation of a pancake-type collapse, it also plays a role in the other discussed types. A force concentration in the element that is to fail next, induced by the previous element failure, occurs in all types of collapse discussed except possibly in the instability-type of collapse. Such force concentration is another feature promoting the propagation of collapse. Dynamic action and force concentration become more detrimental as the material of the element that is prone to fail next becomes more brittle. Ductile material, on the other hand, is able to absorb kinetic energy and renders possible a redistribution of forces and thus a reduction of force concentration. The beneficial effect of ductile material behavior is less obvious in the prevention of a domino-type collapse. But even there, it can help to absorb kinetic energy when the affected element, say, the tower of a transmission line, is anchored to the ground or to other elements and overturning requires the failure of those anchors.

1.4.2 Overstrength and ductile material behavior

Intriguingly, there are also instances where strength and even ductility are detrimental. If the propagatingaction of a domino-type collapse is transmitted by mediating elements in tension (say, a transmissionline), these elements and their connections are likely to transmit forces larger than those occurring undernormal conditions. Thus, overstrength should be avoided in such elements. But even then, the actualstrength of a mediating element is possibly larger than the element prone to overturn next (a tower) canreasonably be designed for.

1.4.3 Structuredness

A further collapse-promoting feature appearing in some of the types of collapse discussed is the structuredness (as opposed to smoothness or compactness) of a structure. Structuredness is the degree to which a system possesses a definite pattern of organization of its interdependent parts [11]. In this sense, a high-rise building with its pattern of horizontal (beams, slabs) and vertical (columns) elements is highly structured whereas a reinforced-concrete industrial chimney (tube) is not. It seems that structuredness is a condition for pancake-type collapse.

Chapter 6
CASE STUDY OF A FURNANCE STEEL TOWER

6.1 INTRODUCTION

Based on the features presented in the previous Chapters, progressive collapse is a special way of collapse that is related with two different aspects: a) the geometry of the structure involved and b) the physical characteristics of the input cause. Concerning the design of new structures against facts that may possibly lead to progressive collapse (fire, explosion, earthquake, impact etc.), two approaches have been introduced apart from the traditional code provisions, as been presented by Prof. Fardis (proceedings of the 1st International Conference on Natural Hazards & Infrastructure, 2016, Chania, Greece): "a) either to include *key elements*, vital for the stability of the whole, to resist a prescribed threat, or b) to retrofit the rest of the system to sustain a postulated loss of one or more key elements, no matter the threat. In option a) the large inelastic deformations and internal forces induced to *key elements* by the impulsive loading should be estimated (e.g., on the basis of Smith and Hetherington 1994, Mays and Smith 1995, Ngo et al. 2007) and checked against the corresponding capacities. However, these capacities are largely unknown. Proposals made for their estimation are crude and arbitrary extensions from completely different loading conditions. Option b) normally requires a large-displacement, material-nonlinear dynamic analysis, with instant removal of incapacitated key elements. The analysis may instead be static and – under certain ill-justified conditions – even linear, with the gravity loads on all floors above and all bays around such an element multiplied by empirical dynamic load factors supposedly accounting for the inelastic deformations. The main weakness of the approach is that the so-estimated member deformation demands in the standing part of the structure are meant to be checked against largely unknown capacities. Their estimates in Xiao (2012) are again crude and ill-founded".

In order conclude over the previous statements, a case study of a furnance steel tower was introduced so as to be analyzed under simple earthquake input motions. The specific structure was chosen for a couple of different reasons: a) it is a real structure, that was designed and constructed in 2011 as a part of a perlite factory & offices at Technopark in Dubai after followed specific code provisions (British Standard BS–5950-1:2000/Load Combinations, British Standard BS–EN 1991–1–4:2003/2005/Wind Actions, UBC 97/1997 UBC Earthquake Design, British Standard BS - EN 1993-1-1:2005, Eurocode 3/Design of steel structures), b) due to its final use, it is suitable for resisting great input motions and entering forces and b) it was designed according to traditional code provisions, with X braces covering the total height of the tower between its elevations and special continuous steel ties all over the floors and along the vertical elements to arrest progressive collapse should a vertical support be lost.

Many FEM softwares [Abaqus 6.13-1 (Simulia) etc] that are available for engineers, are able to take into account the proposed "element deletion" when its capacity wears out. In the terms of this study we focused only on the geometry of the structure and how it can bear with an extreme seismic motion, although the yielding and plastic hinging takes place, without deleting any of the "weak" elements. In other words, dynamic implicit analyses were performed and the output were evaluated in the time domain for the estimation of the stability of the structure. The stability is investigated after the input motion is progressively amplified, until the tower is considered collapsed. The behavior of its vertical columns, as well as its diagonal X-braces is studied and critical conclusions are drawn.

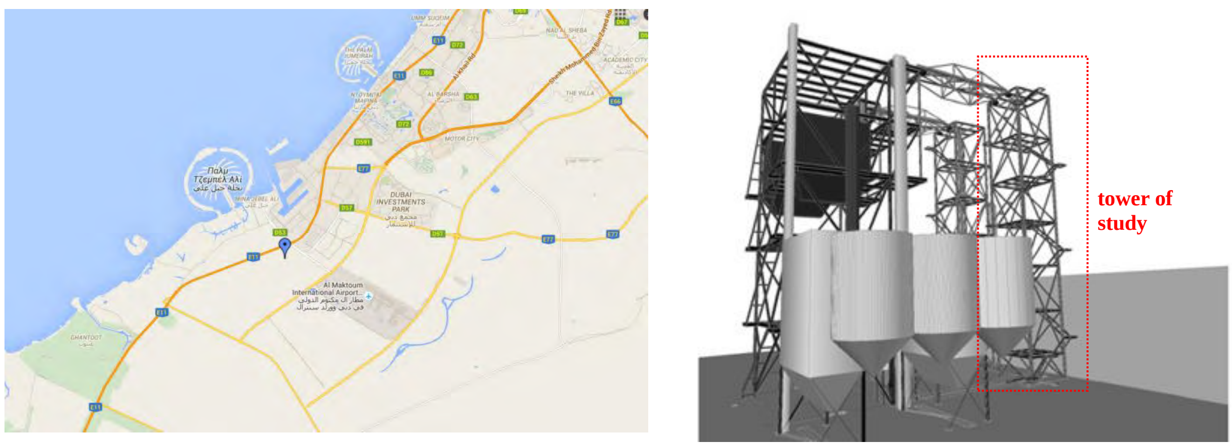
The collapse behavior is investigated through a suite of ground motion analyses. First the tower is analyzed under the time history event of El Centro Earthquake, recorded by El Centro station of Imperial Valley irrigation district, that took place in the Imperial Valley in 1940. Then the input motion is scaled until the tower collapses. It is shown that the tower collapses as a result column and brace buckling in the bottom segment. The Abaqus (Simulia) model of the tower reveals severe buckling in the bottom columns and at one of the two braces on the west face of the tower when the structure is hit by the El Centro pulse, resulting a tilt in the structure. This is followed by sequential compression buckling of braces on the south and north faces leading to $P - \delta$ instability and complete collapse of the tower. To aid in the evaluation of the collapse-prediction capability of competing methodologies, detailed results (time-history plots as well as ordinates of crests and troughs in these histories) are provided for the analysis at 10 scaling.

Figure 6-1: Picture of the constructed filter and furnance towers (http://www.uaeperlite.com/contact-us.html).

6.2.STEEL TOWER & MODEL DESCRIPTION

The tower under consideration is situated in Dubai, in the Technopark, Jebel Ali, in Dubai (see Fig.6-2). It was constructed in 2011 as a part of a bigger concept, that included a perlite factory and it is a twin tower, each one servicing as an expansion furnance tower attached on the main filter. The tower is made of structural steel of quality S275 (Fe430) and its total height is 26.40 m, while the dimensions of the rectangular plan view of each level are 4.96m x 3.96m.

The total project was originally studied and designed according to the provisions of British Standard with the help of the software Autodesk Robot Structural Analysis Professional 2014. The importance of the structure, imposed by its use and the materials processed, guided so as to bare with extremely strong input motions. Although it was obvious that the scale factor that would lead to collapse would be a large number, the structure is configured such that its collapse is always triggered by element buckling in the same region of the supporting lattice when excited by any "collapsogenic" ground motion.



(a) (b)

Figure 6-2: a) Location of the construction of the factory (Google Maps); Snapshot of the model of the towers in Robot (Autodesk).

The tower's total weight is 15.21 Mg (tons). Figure 6-3 shows an axonometric image of the tower's design in Autocad 2011 (Autodesk) and also the horizontal footbridge connecting the tower with the filter production tower. The supporting lattice consists of seven (7) segments (labeled 1–7 in the figure) over its height, with four mega-corner-columns interconnected by beams and X-braces forming a rigid spine. The four mega-columns are made of HEB180 steel sections of steel quality S275, the 4 external beams that form the rectangular that connects the columns at each of the 7 elevation segments are made of IPE200 steel sections, while the internal grid of the same segments are made of IPE200 and HEA120 sections of the same steel quality. There is also a central column, which supports the structure along its length and is supported to the ground, which is made of TRON 406x8 section (pipe beam section of radius 0.203m and thickness 0.008 m), of S275 steel quality. The X-braces at the elevation that is close to the ground are made of TCAR100x7.1 and of TCAR100x5 at the rest of the structure, of the same steel quality.

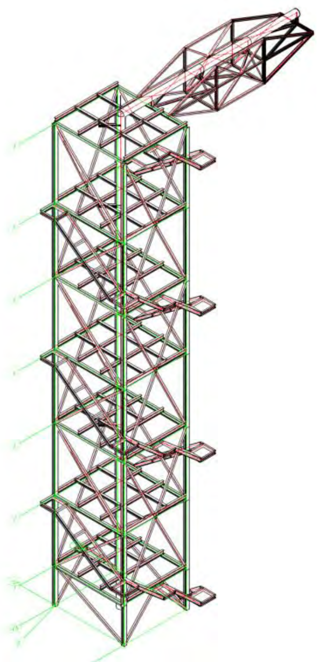
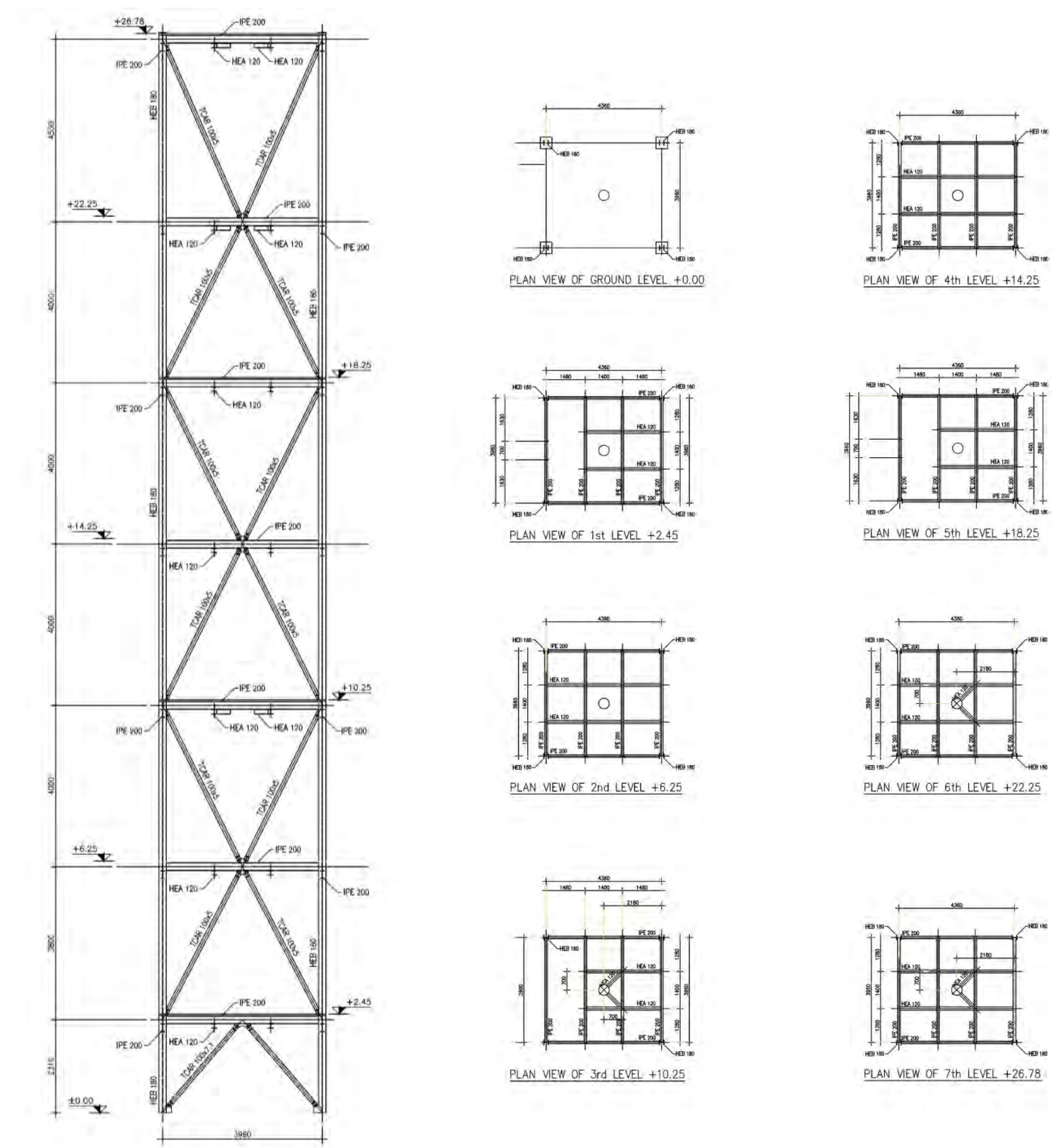


Figure 6-3: Axonometric view of the furnance tower of study, designed in Autocad 2011 (Autodesk).

For the purposes of this study, software Abaqus 6.13-1 (Simulia) was used. The model was based on the detailed plan views and sections that the study of the structure included (Figure 6-4). The staircases and rails that originally exist externally and along the height of the tower were ignored during the modeling. Full continuity was assumed at all the connections and a fixed boundary condition was assumed for the base nodes of the tower. For the time history analysis, the model was meshed to 905 linear beam elements of type B31 (2 node linear beam elements with single integration point per element). The material behavior was considered bilinear (plastic), experiencing kinematic hardening according to the law that Figure 6-5 indicates. The modulus of elasticity was considered E_s=210GPa, the mass density ρ_s=7500 kg/m3 and the Poisson's ratio ν=0.30. The bilinear behavior is based on a linear curve fitting to the experimental data of uniaxial stressing of the steel of quality S275 and was introduced for the purposes of the analyses. The yield was considered to occur at stress σ_{fy}= 340652 kPa and strain ε_{sy}=1,62 ‰, while the same parameters at the ultimate state get the values σ_{fu}= 469496,4 kPa and ε_{sy}=186,09 ‰.

The static loads introduced into this model, apart from the self weight that is uniformly distributed to all the elements to the direction of the global Z axis, act only to the horizontal members of each elevation and belong to two categories: the dead and the live loads and are also imposed along the global Z-axis (axis along the height of the tower). The wind loads were ignored in this study. The values of the loads were based on the technical report that accompanies the original study of the steel tower. The surface loads were distributed appropriately to each element and were considered as line loads. Table 6-1 shows the loads imposed at each elevation level.



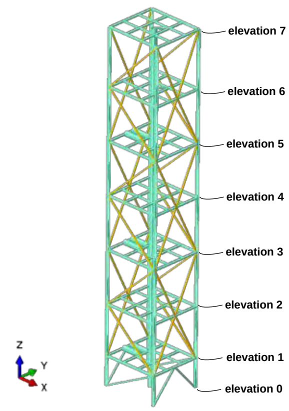
(a) (b)

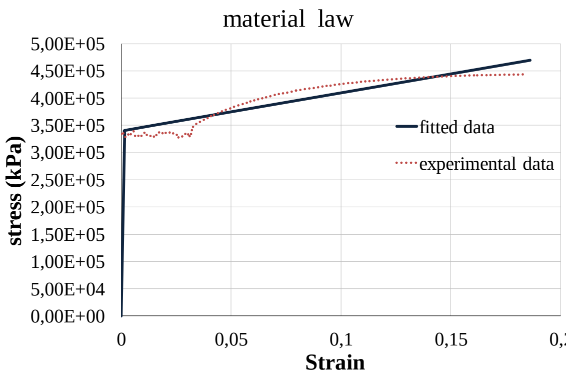
Figure 6-4: (a)Section of the tower; (b)Plan views of the elevations.

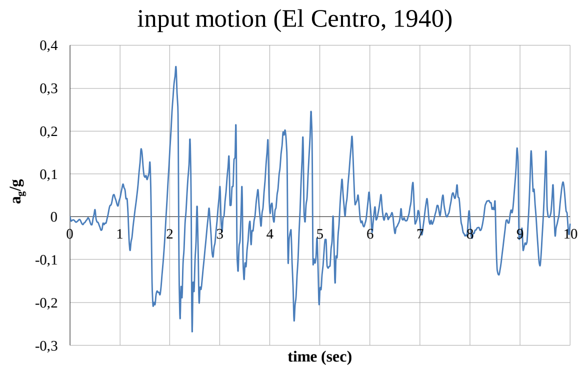
The dynamic loading for the time history analysis to take place, was chosen to be the El Centro motion, recorded by the Imperial Valley Irrigation District. The 1940 El Centro earthquake (or 1940 Imperial Valley earthquake) occurred at 21:35 Pacific Standard Time on May 18 (05:35 UTC on May 19) in the Imperial Valley in southeastern Southern California near the international border of the United States and Mexico. It had a moment magnitude of 6.9 and a maximum perceived intensity of X (Extreme) on the Mercalli intensity scale. It was the first major earthquake to be recorded by a strong-motion seismograph located next to a fault rupture. The earthquake was characterized as a typical moderate-sized destructive event with a complex energy release signature. It was the strongest recorded earthquake to hit the Imperial Valley, and caused widespread damage to irrigation systems and led to the deaths of nine people. The maximum acceleration recorded is almost 0.35g. Figure 6-5 shows the time history of the specific seismic event used as an input motion. A time-step size of 0.02s is used for the ground motion analyses.

Table 6-1: Total line loads of elements of each elevation of the tower.

Levels	Q_{tot} (kN/m)
Elevation 0 (ground)	0.00
Elevation 1	-3.80
Elevation 2	-3.80
Elevation 3	-3.80
Elevation 4	-3.80
Elevation 5	-3.80
Elevation 6	-1.95
Elevation 7	-1.95







(a) (b)

Figure 6-5: (a)Experimental data of uniaxial stress of S275 steel (red dotted line), Fitted data for the behavior of the S275 steel (blue continuous line); (b) El Centro time history (Station: El Centro, CA-Array Sta 9; Imperial Valley Irrigation District, Component:180).

6.3 TIME HISTORY ANALYSIS RESULTS

The model used in this study is shown in Figure 6-5 (Abaqus CAE). It consists of 778 node points and 905 beam elements. The input motion (as seen in Figure 6-3) is introduced along the Y-axis, to the nodes that are attached to the ground, according to the coordinate system of the Abaqus model, while the other degrees of freedom in X-axis and Z-axis are considered fixed. Full continuity was assumed at all the connections. In the first place, for analysis purposes, a static analysis was performed (STEP 1) with the Abaqus 6.13-1 software. Then, dynamic implicit analysis was conducted with a total duration of 10 secs, the same as the input motion's (STEP 2).

The first dynamic implicit analysis performed excited the model to the input motion with no amplification factor (scaling:1). The time history of the roof displacements (elevation 7) relative to the ground level (elevation 0) can be seen in Figure 6-6. It is obvious that no collapse takes place, as the roof oscillates around the initial position of equilibrium. The initial displacement U_x that is observed at time 0 sec is due to the static analysis that proceeded the dynamic (for software purposes). The same figure reveals a snapshot of the tower's motion at time 9.68 sec, at which the maximum displacement U_y takes place.

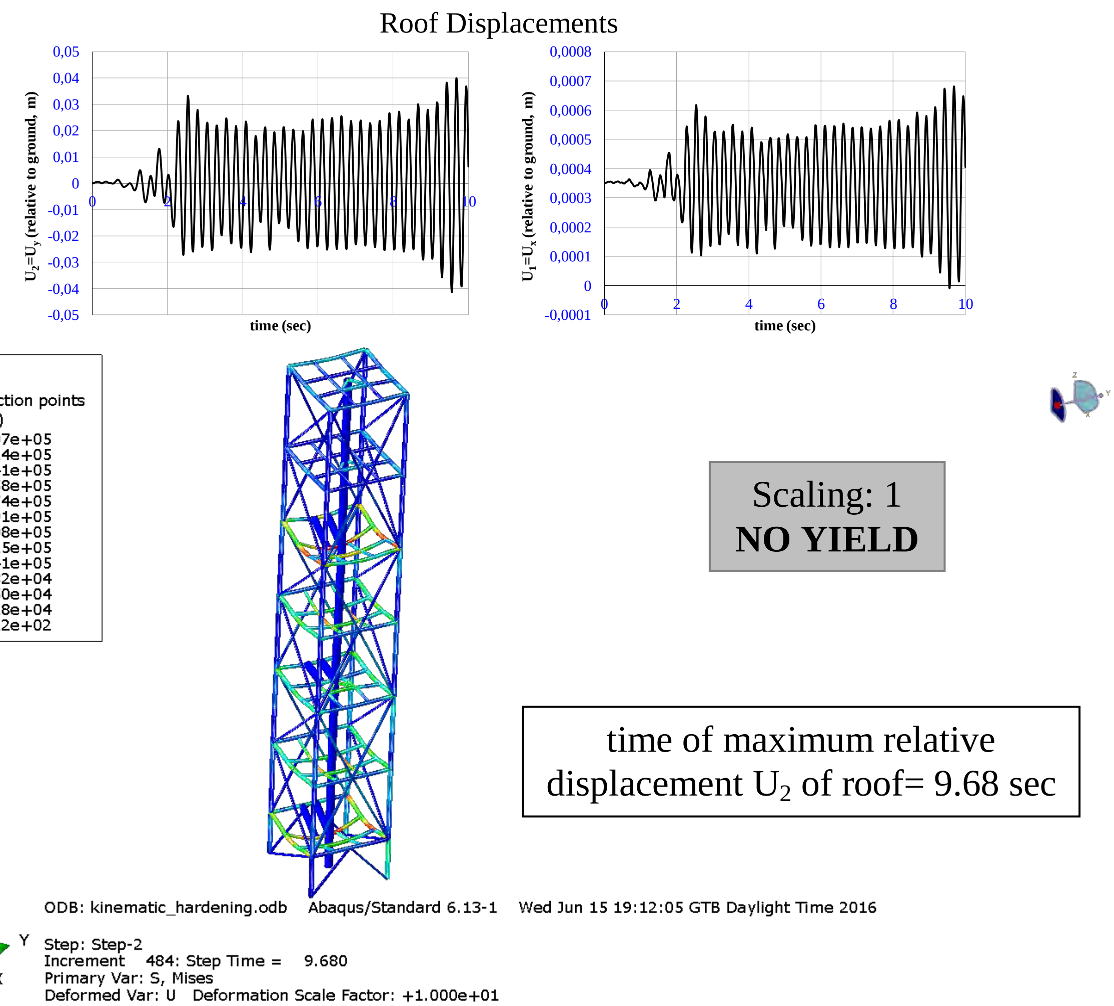


Figure 6-6: Snapshot of the deformed tower at t=9.68 sec of the excitation (maximum displacement U_y), with colored contours of stresses (Mises) which leads to no yield of the elements (SCALING: 1).

The second analysis performed included the excitation of the tower to the El Centro input motion scaled with a factor of 5. After observing the relative displacements of the roof tower with respect to the ground in Figure 6-7, it can be easily concluded that no collapse takes place. What actually happens is that due to the non symmetric geometry of the tower (non symmetric stiffnesses of beams between the elevations), it suffers severe torsion during the excitation and it results with great displacements at x direction that if the duration of the motion was larger would probably lead to collapse due to torsional effects.

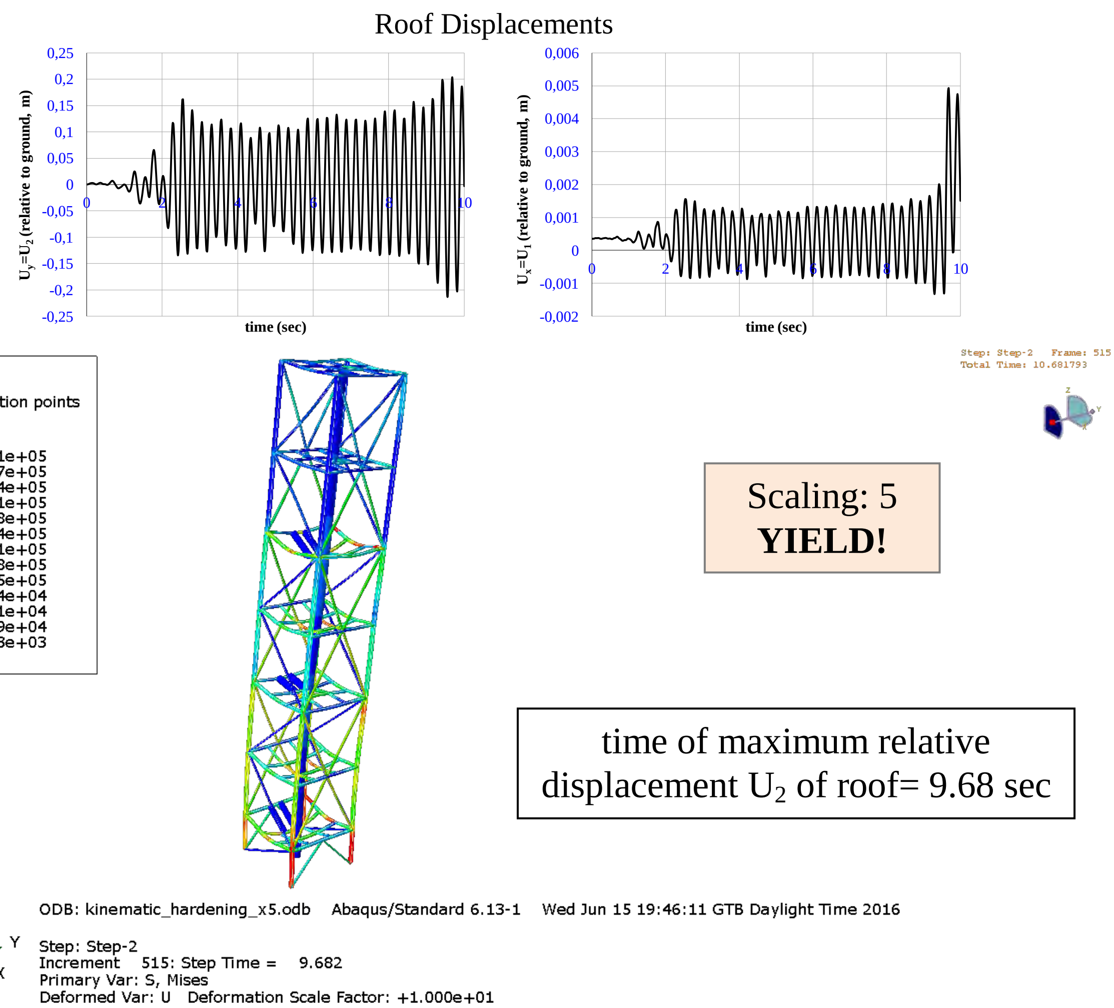
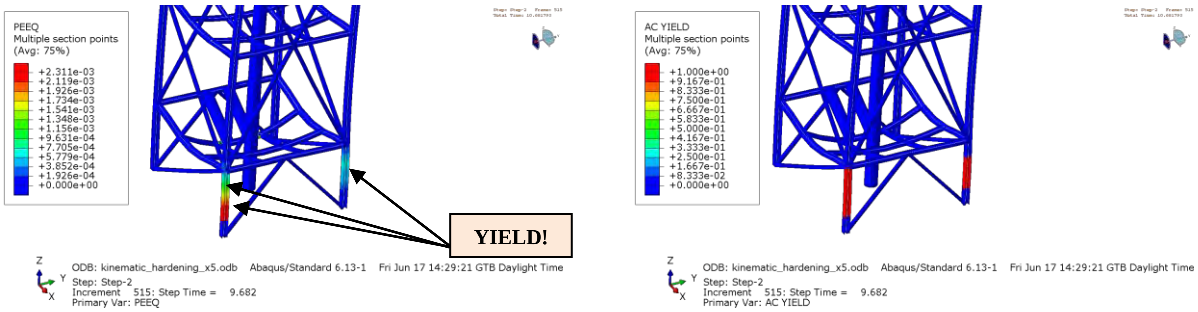


Figure 6-7: Snapshot of the deformed tower at t=9.68 sec of the excitation (maximum displacement U_y), with colored contours of stresses (Mises) which leads to yield of some elements (SCALING: 5, TIME=9.68sec).

In order to investigate further the above observation it is useful to comment over Figure 6-8, that is a graphical representation of the equivalent plastic strain that the steel tower develops. The equivalent plastic strain (PEEQ) is the effective scalar quantity equivalent to Mises stress and is equal to sqrt((2/3)*PE_{ij}*PE_{ij}), where PE_{ij} are the components of the symmetric tensor of the plastic strain quantity of each element, at its integration point. The figure focuses on the ground floor of the tower where the maximum stresses and strains occur at the time point of t=9.68 sec. Two of the four corner columns between elevation 0 and 1 experience yield, with different stresses and strains and this possibly leads to large deformations and rotations around the center of the mass/rotation.



(a) (b)

Figure 6-8: (a) Colored contours of the PEEQ (equivalent plastic strain-scalar quantity) of the ground level of the model; (b) Active yield flag (ACTIVE YIELD SURFACE=1, INACTIVE YIELD SURFACE=0) of the ground level of the model (SCALING:5, TIME=9.68sec)

In the same figure, active yield flag at the integration point of the elements of the ground floor of the model is plotted (Figure 6-8b), for the time moment 9.68 sec. When the yield surface of the element is active the flag takes the value of 1 and when it is inactive (no yield) the value of 0. The time history of the active yield flag of each element of the columns of the base floor or the time history of the equivalent plastic strains would lead to conclusions over the behavior of the total steel tower. Such parameters will be discussed later on.

The third analysis performed included the excitation of the tower to the El Centro input motion scaled with a factor of 10. The extremely strong input motion reaches an acceleration of 3.5g. The tower after baring with 2.56 sec of the motion, suffers large displacements that are permanent and as a result never returns to the initial position of oscillation. Figure 6-9 shows the displacements of the model's roof with respect to the ground. The roof displacements displayed in the Figures 6-9 correspond to the displacement of the central column of the elevation. The displacement's time history in combination with the PEEQ estimation (Figure 6-11), indicate that significant buckling is occurring in the vertical members of the ground floor, causing the upper structure to undergo free-fall while remaining virtually intact. The severe buckling effects can be observed at Figure 6-12, where the X-Z view of the tower model is shown at the three time snapshots. The scale of the deformation is not the real one, but was chosen to be uniform to all the three analysis results presented.

In combination with the above figures, the PEEQ time histories of the most sensitive elements of the the ground and 1st floor are presented in Figure 6-11. The vertical elements of the right column of the west view (Y-Z plane) of the tower seem to correspond to the maximum buckling effects that lead to collapse. What is more, the elements of the left column of the south view of the 1st floor of the tower (X-Z view) and of the corresponding X-braces of the east view (Y-Z plane) of the tower are also experiencing significant buckling that is related with its large length (Figures 6-13, 6-14).

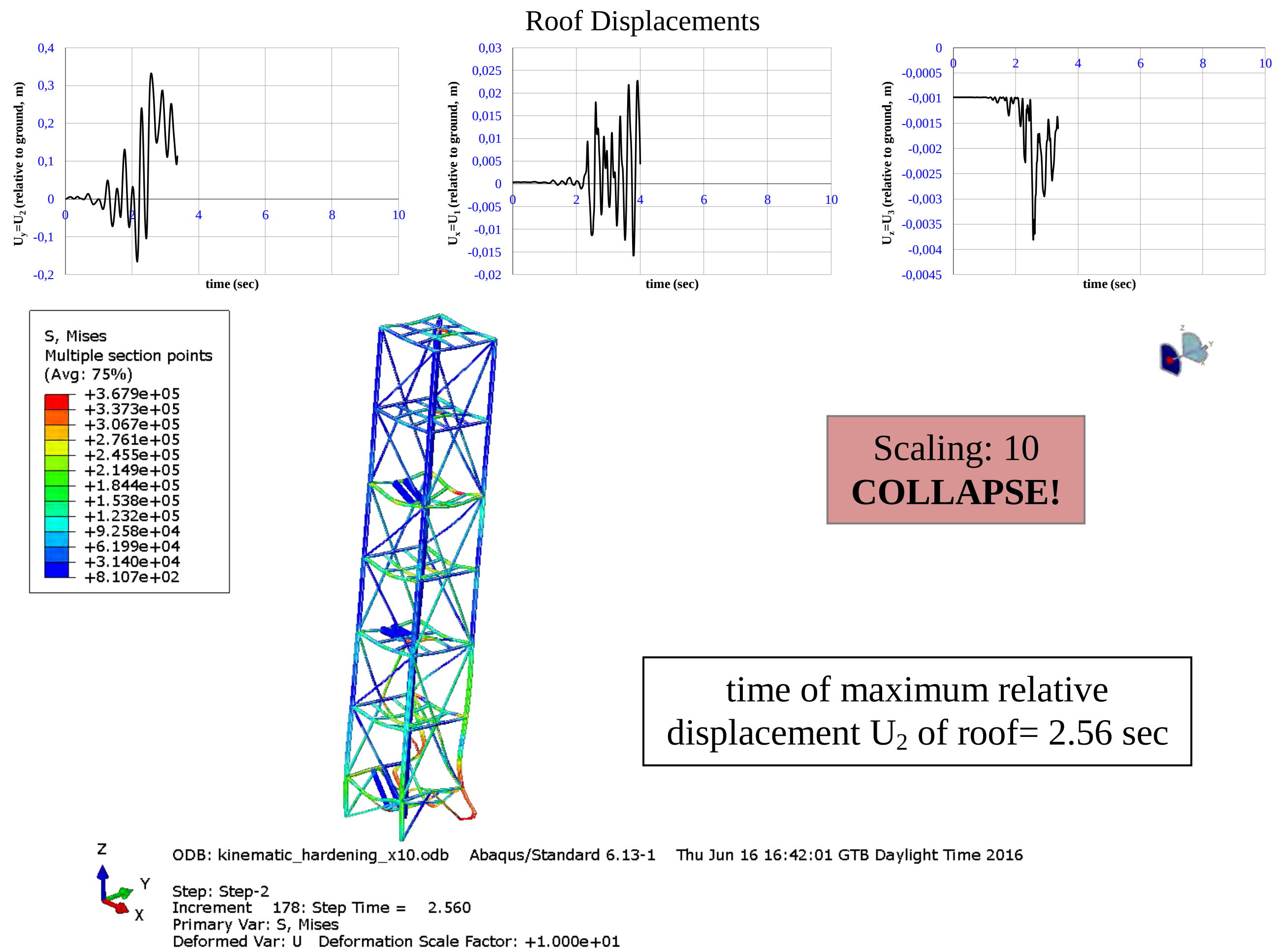


Figure 6-9: Snapshot of the deformed tower at t=2.56 sec of the excitation (maximum displacement U_y), with colored contours of stresses (Mises) which leads to collapse (SCALING: 10, TIME=2.56sec).

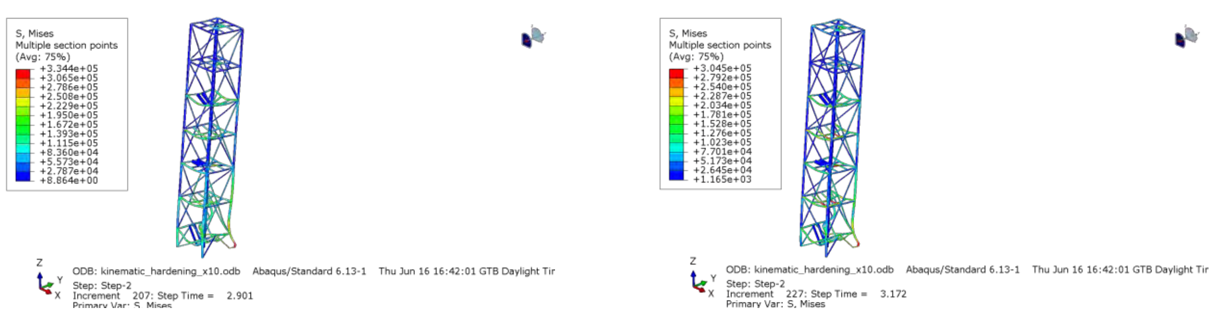
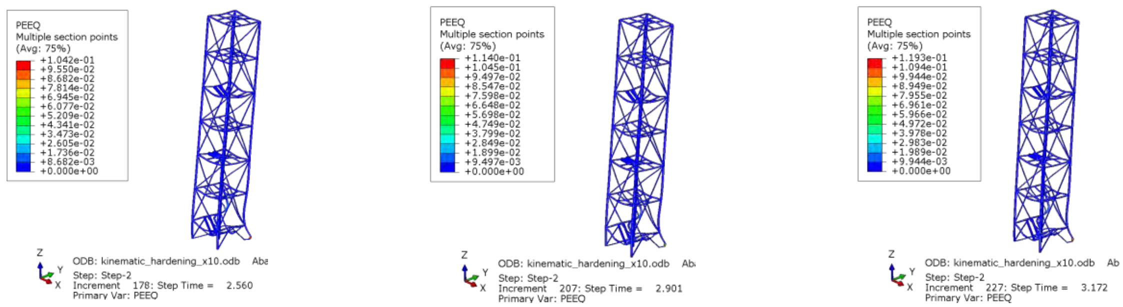
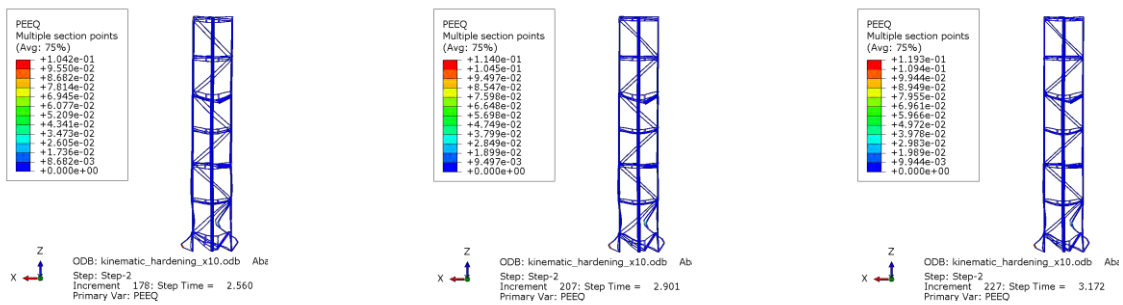


Figure 6-10: Snapshot of the deformed tower at t=2.90 sec and 3.17 sec of the excitation (maximum displacements U_y), with colored contours of stresses (Mises) which leads to collapse (SCALING: 10).



(a) (b) (c)

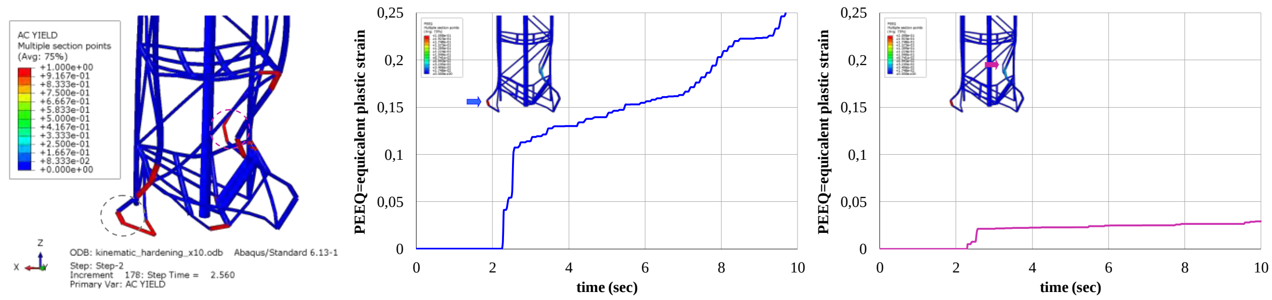
Figure 6-11: Axonometric snapshot of the deformed tower at t=2.56 sec, 2.90 sec and 3.17 sec of the excitation (maximum displacements U_y), with colored contours of equivalent plastic strains (PEEQ) which lead to collapse (SCALING: 10).



(a) (b) (c)

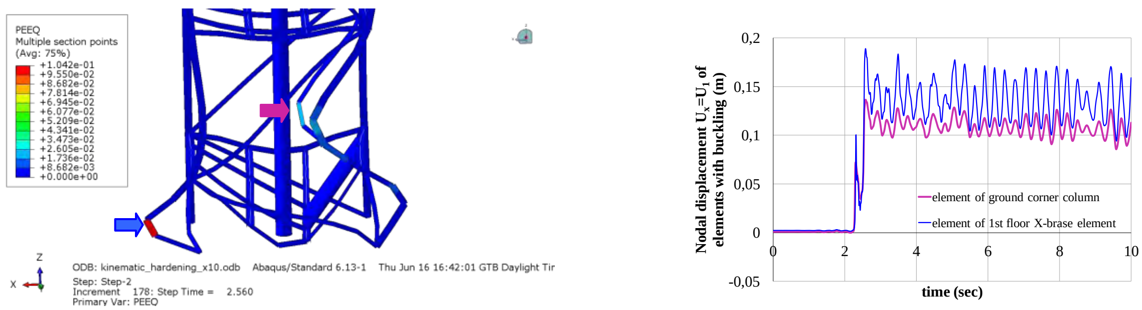
Figure 6-12: X-Z Snapshot of the deformed tower at t=2.56 sec, 2.90 sec and 3.17 sec of the excitation (maximum displacements U_y), with colored contours of equivalent plastic strains (PEEQ) which lead to collapse (SCALING: 10). Large buckling effects can be observed.

Although the previous results of displacements where shown up to the initiation of the tilt motion, the plastic strains are shown through the total duration of the seismic motion. In Figure 6-13b, the corner column of the ground floor seems to yield after 2.2 sec of excitation and afterwards it continues to deform until it reaches a strain of value 0.25. The kinematic law of the material is the one that leads the behavior of each element and that is the main reason of not total destruction of the elements takes place. By this way, the collapse due to the complete destruction of the elements is prevented. Significant but not severe plastic strains can be seen in Figure 6-13c, for the X-Brace member of the first floor. The comparison of the displacement time histories of the two elements experiencing maximum buckling effects is shown in figure 6-14. Although PEEQ values are much greater for the corner column of ground floor, the horizontal displacements are more intense for the X-Brace member of the 1st floor. The displacements are in phase concerning the specific seismic excitation and that probably leads to greater roof displacements and finally collapse. What is also interesting is that the buckling effects are observed at the plain X-Z and not towards the direction Y, which is the input motion's direction. This is probably related with the yielding of the two bottom level's corner columns, that had already happened since the excitation with the amplification factor 5 (see Figure 6-8).



(a) (b) (c)

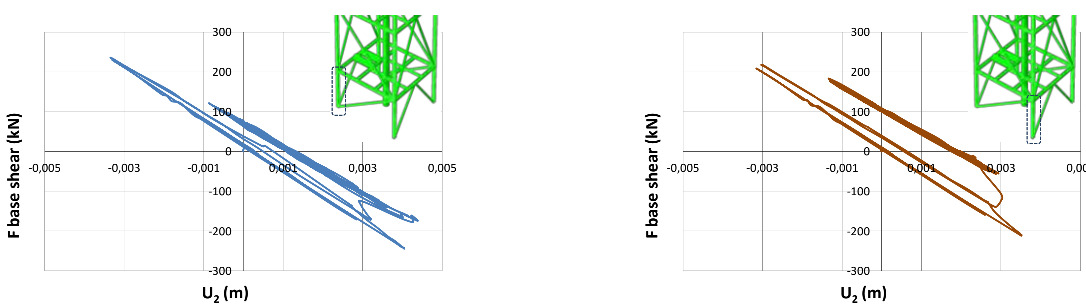
Figure 6-13: (a)Active Yield surfaces of the tower's ground and 1st elevation; (b)Time history of equivalent plastic strains (PEEQ) of a corner column element of ground floor; (c)Time history of equivalent plastic strains (PEEQ) of an X-brace member of 1st floor (SCALING: 10).



(a) (b)

Figure 6-14: (a)PEEQ of the tower's ground and 1st elevation; (b)Time history of horizontal displacements U_x of the nodes of the two elements shown, experiencing maximum buckling effects (SCALING: 10).

The base shear of the 4 corner columns of the tower model with respect to the displacement drift of the ground floor (elevation 1-elevation 0) is shown in Figures 6-15, towards the direction of the input motion (Y-axis). The loop reveals the energy consumption of the input motion by the base floor and it corresponds to the total seismic motion (duration= 10 sec). It is obvious that the greater energy dissipation takes place in the two columns that yield from the beginning of the excitation (see Figure 6-8). What is more, in Figure 6-16, the time history of the total base force of the tower model reveals that the maximum forces appear at the time moment that the structure tilt initiates.



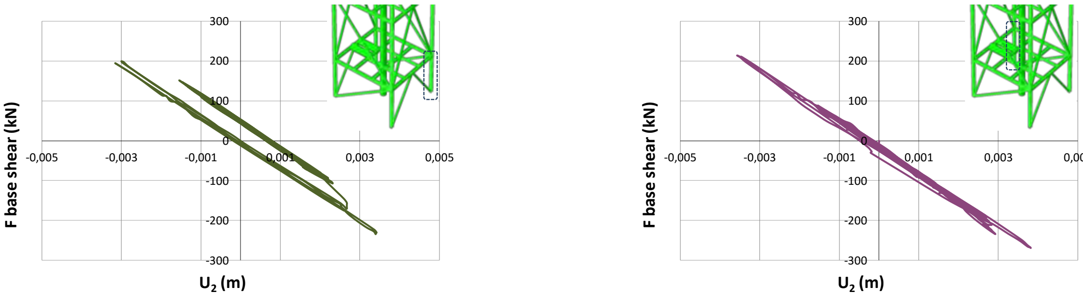


Figure 6-15: Loops of the Base shear of each of the 3 corner columns of the fround level with respect to the relative displacement of the top and the bottom (SCALING: 10).

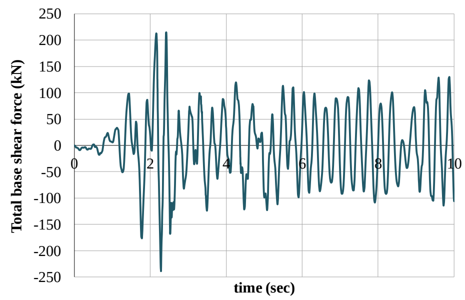


Figure 6-16: (b) Time history of the base shear force (SCALING: 10).

6.4 CONCLUSIONS

A case study of the collapse of a steel tower under earthquake loading is presented as a potential candidate for low-complexity benchmarking in the evaluation of the collapse-prediction capability of competing methodologies. It consists of four corner columns and one central that are tied together by seven levels of vertical and horizontal bracing. The structure is a uniaxially symmetric and its braces, columns, and beams have a quite uniform sizing for the entire height of the tower. This configuration makes it possible for the collapse to occur due to buckling of the mega-columns and braces in the bottom segment of the steel lattice, and overturning due to the ensuing $P - \delta$ instability. Incremental dynamic analysis is conducted on an Abaqus-CAE model of the tower, subjected to the motion from the 1940 El Centro earthquake. Linear beam elements are used to model all the members including braces. Collapse is found to occur at the threshold scaling factor of 10. Severe buckling occurs in the bottom corner column and one of the two braces on the east face of the tower when the structure is hit by the El Centro pulse, resulting a tilt in the structure. This is followed by sequential compression buckling of braces on the south and north faces leading to $P - \delta$ instability and complete collapse of the tower. Detailed results including response histories and ordinates of the crests and troughs in the histories are provided to aid in low-complexity benchmarking.

Chapter 7
SUMMARY

7.1 INTRODUCTION

Although the risk of progressive collapse in most buildings is low, recent terrorist attacks on buildings throughout the world have heightened an awareness of the need to limit the spread of damage in structures subjected to abnormal events, such as explosions and vehicular impacts, so as to avoid progressive collapse. In the U.S., model codes and national design standards do not provide explicit provisions for designing to resist progressive collapse. There are, however, general provisions for "structural integrity" to provide a minimum level of structural continuity and load redistribution capability. Thus an owner's decision to proceed with explicit consideration of progressive collapse for a particular structure has to be made on a case-by-case basis after a rational analysis of risk, as discussed in Chapter 2.

Designing for reducing the risk of progressive collapse requires a different way of thinking compared with traditional design to resist prescribed vertical and lateral loads. The design process must focus on what may go wrong and must identify the performance requirements to be met. The design scenario may be threat-specific or non threat-specific. The design team must determine which abnormal load events and damage scenarios should be considered and what are the acceptable levels of risk.

The purpose of this report is to acquaint owners, engineers, and building officials with best practices for designing buildings to reduce the likelihood of progressive collapse in the event of local damage from an abnormal load. Guidance is provided on the basis of existing knowledge. This chapter summarizes the main points associated with design to prevent progressive collapse that are covered in the report.

7.2 DEFINITION

The professional community needs to adopt a common definition of the term "progressive collapse" in order to reduce misunderstandings among design professionals and with the public. The following definition is recommended:

> progressive collapse—the spread of local damage, from an initiating event, from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it; also known as disproportionatecollapse.

7.3 REDUCING RISK

Chapter 2 provides a comprehensive discussion of a risk basis approach to progressive collapse. The probability of structural collapse, P(C), as a result of potentially damaging abnormal load, H, was presented as Eq. 2.6, and is reproduced here:

$$P(C) = P(C|LD)\, P(LD|H)\, \lambda_H \qquad (2.6)$$

where

λ=	rate of occurrence of the abnormal load or hazard,
P(LD|H) =	probability of local damage given that the abnormal load occurs, and
P(C|LD) =	probability of collapse given that local damage occurs.

This equation provides a convenient means of understanding the basic strategies for reducing the likelihood of progressive collapse. These strategies are:

- Event control;
- Specific local resistance; and
- Alternate load path.

Event control— This involves taking actions to minimize the effects of (or reduce the likelihood of) the hazards. This approach requires that the hazard or spectrum of hazards be identified. Such actions might include changes in the building site or access to it, use of perimeter barriers, or controlling hazardous materials. Event control is often the most cost-effective means of risk reduction, and Chapter 3 provides a comprehensive discussion of this topic.
Specific local resistance—This involves designing key structural elements to reduce the likelihood of local damage during an abnormal load event. This is a threat-specific approach and is often the most rational approach for retrofitting existing buildings. Chapter 4 discusses this approach.
Alternate load path—This involves designing the structure so that it can “bridge over” the local damage caused by an abnormal load event and thereby reduce the likelihood of progressive collapse. The design might be for a specific threat or it might be threat independent. This approach is discussed in Chapter 4.

7.4 DESIGN METHODS

As discussed in Chapters 2, 3, and 4, the design team can use an indirect method or one of two direct methods to provide resistance to progressive collapse. The indirect method is a prescriptive approach of providing a minimum level of connectivity between the various structural components. The designer is not required to analyze the structure for the effects of abnormal loads. The indirect method was used first in the U.K., and been adopted by the DOD in UFC 4-023 for use in buildings characterized as requiring a very low level of protection. Chapter 4 gives general guidance on the indirect method and Appendix A gives specific tie strength requirements from various design standards.

The direct methods include the method of specific load resistance and the alternate load path method. These methods require the designer to perform an explicit analysis of the effects of abnormal load events on the structure. The analyses may be threat specific or threat independent as discussed in Chapter 4. Various analysis techniques may be used ranging from a linear elastic, static load method to an inelastic dynamic load method. Required computational resources, fidelity of structural modeling, and experience of the analysis are factors to be considered in selecting the appropriate analytical method for a specific project. Accompanying any analytical method is the criteria for acceptable performance under the abnormal load event. Appendix A provides examples of acceptable performance criteria stipulated in different design guidelines.

7.5 GENERAL GUIDELINES

Good structural design integrates the gravity-load resisting system, including the system for resisting progressive collapse, with the lateral-force resisting system. Chapter 5 provides cost-effective guidelines for improving the progressive collapse resistance of buildings incorporating different construction materials or construction methods. In general, the design should provide a combination of strength, ductility, and continuity to permit the structure to absorb the effects of local damage. The design should avoid those features that will make it more difficult to provide progressive collapse resistance, such as large transfer structures and discontinuities in the structural system.

7.6 EXISTING BUILDINGS

Existing buildings pose challenges in developing cost-effective solutions for upgrading the resistance to progressive collapse. As discussed in Chapter 5, potential engineering solutions might include local strengthening to prevent initial failure or enhancing the redundancy of the existing structural system to limit the spread of a local failure. Any upgrade program begins with an evaluation of potential threats and an evaluation of the existing structural system. Early in the process, however, the engineer needs to establish whether the constraints established by the existing conditions pose technical of economic obstacles to viable upgrade options.

7.7 DESIGN GUIDES

The document is not intended to provide step-by-step guidance on designing buildings for resistance to progressive collapse. Such guidance is available in national standards and design guidelines developed worldwide. In the U.S., the prominent documents are: Progressive Collapse Analysis and Design Guidelines developed by the General Services Administration (GSA 2003) and Design of Buildings to Resist Progressive Collapse developed by the Department of Defense (DOD 2005). Appendix A provides excerpts from these design documents as well as other design standards.

This page intentionally left blank

Chapter 8
REFERENCES

- ACI Standard ACI 318-05, (2005), Building Code Requirements for Structural Concrete, American Concrete Institute, Farmington Hills, Michigan.
- AISC Standard ANSI/AISC 360-05, (2005a), Specification for Structural Steel Buildings, American Institute of Steel Construction, Chicago, Il., 198 p.
- AISC Standard ANSI/AISC 341-05 (341s1-05), (2005b), Seismic Provisions for Structural Steel Buildings Including Supplement No. 1, American Institute of Steel Construction, Chicago, Il., 333 p.
- AISC Standard ANSI/AISC 358-05, (2005c), Prequalified Connections for Special and Intermediate Steel Moment Frames for Seismic Applications, American Institute of Steel Construction, Chicago, Il., 91 p.
- Allen, D. and W.R. Schriever, (1973), "Progressive Collapse, Abnormal Loads and Building Codes," in Structural Failures: Modes, Causes, Responsibilities, American Society of Civil Engineers, New York, NY, pp. 21-48.
- ASCE/SEI/SFPE Standard 29-05, (2007), Standard Calculation Methods for Structural Fire Protection, American Society of Civil Engineers, Reston, Virginia, 80p.
- ASCE Standard ASCE/SEI 7-05, (2005), Minimum Design Loads for Buildings and Other Structures, American Society of Civil Engineers, Reston, Virginia.
- ASTM Test Method E119-02, (2002), "Standard Methods of Fire Tests of Building Construction and Materials," ASTM International, West Conshohocken, PA
- Bailey C.G. and Moore D.B., (2000a), "The Structural Behaviour of Steel Frames with Composite Floor Slabs Subject to Fire—Part 1 Theory," The Structural Engineer Vol. 78, No.11; June 2000.
- Bailey C.G. and Moore D.B., (2000b); "The Structural Behaviour of Steel Frames with Composite Floor Slabs Subject to Fire—Part 2 Design," The Structural Engineer Vol. 78, No.11; June 2000.
- Bennetts, I.D. and I.R. Thomas, (2002), "Design of Steel Structures Under Fire Conditions," Progress in Structural Engineering and Materials, 4(1), pp. 6-17.
- Breen, J.E. and C.P. Siess, (1979), "Progressive Collapse—Symposium Summary," ACI Journal 76(9), pp. 997-1004.
- Breen, J.E., Editor, (1975), "Summary Report, Research Workshop on Progressive Collapse of Building Structures," The University of Texas at Austin, November.
- Burnett, E.F.P., (1975a), "Abnormal Loading and Building Safety," SP-48, American Concrete Institute, Farmington Hills, MI, 1975.
- Burnett, E.F.P., (1975b), "The Avoidance of Progressive Collapse: Regulatory Approaches to the Problem," Report No. NBS-GCR-75-48, National Bureau of Standards, Washington, DC.
- Canadian Standards Association (CSA), (2000), Limit States Design of Steel Structures, CSA S16.1, Rexdale, Ontario, Canada.
- CEN, (1994), "Eurocode 1—Actions on Structures, Part 1 - Basis of Design," European Pre-standard ENV 1991-1," Comité Européen de Normalization 250, Brussels, Belgium.
- Chalk, P. and Corotis, R.B., (1980), "Probability Model for Design Live Loads," J. Struct. Div. ASCE 106(10), pp. 2017-2033.
- CIB W14, (1983), "A Conceptual Approach Towards a Probability-Based Design Guide on Structural Fire Safety," Fire Safety Journal 6(1), pp. 1-79.
- Citipitioglu, A.M., Haj-Ali, R.M., White, D.W., (2002), "Refined 3D Finite Element Modeling of

- Partially-Restrained Connections Including Slip," Journal of Constructional Steel Research, Vol. 58, No. 5-8, pp. 995-1013.
- Conrath, E.J., et al., (1999), "Structural Design for Physical Security—State of the Practice," Task Committee Report, American Society of Civil Engineers/SEI, Reston, VA.
- Corley, W. Gene; Sozen, Mete A.; Thornton, Charles H.; and Mlakar, Paul F., "The Oklahoma City Bombing: Improving Building Performance Through Multi-Hazard Mitigation," FEMA-277, Mitigation Directorate, August 1996, U.S. Government Printing Office, Washington, DC, 90 pp.
- Corley, W.G., P.F. Mlakar, M.A. Sozen and C.H. Thornton, (1998), "The Oklahoma City Bombing:
- Summary and Recommendations for Multihazard Mitigation," J. Perform. Constr. Fac. ASCE 12(3), pp. 100-112.
- Crawford, J. E., (2002), "Retrofit Measures to Mitigate Progressive Collapse," NIST/NIBS Multihazard Mitigation Council National Workshop on Prevention of Progressive Collapse, Chicago,Ill.July. http://www.nibs.org/MMC/ProgCollapse presentations/Crawfords_paper.pdf.
- Department of Defense (DOD), (2001), "Interim Antiterrorism/Force Protection Construction Standards, Guidance on Structural Requirements," Draft, March.
- Department of Defense (DOD), (2003), "Minimum Antiterrorism Standards for Buildings," Unified Facilities Criteria (UFC) 4-010-01, 8 October, 2003.
- Department of Defense, (DOD), (2005), "Design of Buildings to Resist Progressive Collapse," Unified Facilities Criteria (UFC) 4-023-03, 25 January, 2005.
- Ellingwood, B. and E.V. Leyendecker, (1978), "Approaches for Design Against Progressive Collapse." J. Struct. Div. ASCE, 104(3), pp. 413-423.
- Ellingwood, B., et al., (1982), "Probability-Based Load Criteria: Load Factors and Load Combinations." J. Struct. Div. ASCE, 108(5), pp. 978-997.
- Ellingwood, B., (1987), "Design and Construction Error Effects on Structural Reliability," J. Struct. Div. ASCE, 113(2), pp. 409-422.
- Ellingwood, B. and Corotis, R. B., (1991), "Load Combinations for Buildings Exposed to Fires," Engineering Journal, AISC 28(1), pp. 37-44.
- Ellingwood, B., (1994), "Probability-Based Codified Design: Past Accomplishments and Future Challenges," Structural Safety 13(3), pp. 159-176.
- Ellingwood, B., (2001), "Acceptable Risk Bases for Design of Structures," Progress in Struct. Engrg. and Mat. 3(2), pp. 170-179.
- Elms, D. G. (1992), "Risk Assessment," in Engineering Safety, D. Blockley, Ed., McGraw-Hill International, Berkshire, UK, pp. 28-46.
- Engelhardt, M. D. and Sabol, T. A., (1998), "Reinforcing of Steel Moment Connections with Cover Plates: Benefits and Limitations," Engineering Structures, Vol. 20, Nos. 4-6, pp. 510-520.
- Ettouney, M., (2004), "Development of a Progressive Collapse Analysis Method and the PROCAT Computer Program," Counter Terrorism Technology Support Office Technical Support Working Group (CTTSO/TSWG).
- Ettouney, M., Hansen, E., Hapij, A., Lawver, D., Tang, M., Tennant, D., Vaughan, D., and Xie, M., (2004), "Development of a Progressive Collapse Tool," Prepared for Combating Terrorism Technology Support Office/Technical Support Working Group (CTTSO/TSWG), Arlington, VA. Report WA-0403.
- Federal Bureau of Investigation, (1998), 1997 Bomb Summary, FBI Bomb Data Center General Information Bulletin 97-1, Washington, DC.
- Federal Emergency Management Agency (FEMA), (2003), "Primer for Design of Commercial Buildings to Mitigate Terrorist Attacks," FEMA 427, Dec.

- Federal Emergency Management Agency (FEMA), (2000a), "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," FEMA 356, Nov.
- Federal Emergency Management Agency (FEMA), (2000b), "Recommended Seismic Evaluation and Upgrade Criteria for Existing Welded Steel Moment-Frame Buildings," FEMA 351, June.
- Federal Emergency Management Agency (FEMA), (1997), "NEHRP Guidelines for the Seismic Rehabilitation of Buildings," FEMA 273, Oct.
- Galambos, T.V., et al., (1982), "Probability-Based Load Criteria: Assessment of Current Design Practice." J. Struct. Div. ASCE, 108(5), pp. 959-977.
- General Services Administration (GSA), (2003), "Progressive Collapse Analysis and Design Guidelines for New FederalOfficeBuildings and Major Modernization Projects," June.
- Geng, Z.-J., Chajes, M.J., Chou, T.-W., and Pan D. Y.-C., (1998), "The Retrofitting of Reinforced
- Column-to-Beam Connections, "Composites Science and Technology, Vol. 58, No. 8, pp. 1297-1305.
- Green, M. and Wong, Y., (2001), "Value of the Full Scale Cardington Fires Tests to the Structural Engineer," Proceedings of the 2001 Structures Congress & Exposition, P. C. Chang , Ed., ASCE Reston, VA.
- Karter, M. J., (2002), "Fire Loss in the United States During 2001," NFPA, Quincy, MA, 21 p. www.nfpa.org/assets/files/pdf/2001FireLoss.PDF
- Kasai, K., Hodgon, I., and Bleiman, D., (1998), "Rigid-Bolted Repair Methods for Damaged Moment Connections," Engineering Structures, Vol. 20, No. 4-6, pp. 521-532.
- Kruppa, J., (2000), "Recent Developments in Fire Design," Progress in Struct. Engrg. and Mat. 2(1), pp. 6-15.
- Lawver, D., Daddazio, R., Vaughan, D., Stanley, M. and Levine, H., (2003), "Response of AISC Steel Column Sections to Blast Loading," ASME PVP Conference on Problems Involving Thermal- Hydraulics, Liquid Sloshing, and Extreme Loads on Structures, PVP, Vol. 454, ASME, New York, July 2003.
- Leyendecker, E.V. and Burnett, E., (1976), "The Incidence of Abnormal Loading in Residential Buildings," Building Science Series No. 89, National Bureau of Standards, Washington, DC.
- Leyendecker, E.V. and Ellingwood, B.R., (1976), "Design Methods for Reducing the Risk of Progressive Collapse in Buildings," Building Science Series No.98, National Bureau of Standards, Washington, DC.
- Li, J., Bakoss, S.L., Samali, B., and Ye, L. (1999), "Reinforcement of Concrete Beam-Column Connections with Hybrid FRP Sheet," Composite Structures, Vol. 47, No. 1, December, pp. 805-812.
- Mainstone, R.J., (1974), "The hazards of Explosion, Impact and Other Random Loadings on Tall Buildings," Current Paper 64/74, Building Research Establishment, Garston, United Kingdom.
- Marchand, K.A. and Alfawakhiri, F., (2004), Blast and Progressive Collapse, Facts for Steel Buildings Number 2, American Institute of Steel Construction, Chicago, Il., 67 p.
- Marjanishvili, S.M., (2004), "Progressive Analysis Procedure for Progressive Collapse," J. of Performance of Constructed Facilities, Vol. 18, No. 2, May.
- Matousek, M., (1981), "A System for a Detailed Analysis of Structural Failures," Structural Safety and Reliability, Elsevier, Amsterdam, pp. 535-544.
- McGuire, W., 1974, "Prevention of Progressive Collapse," Proceedings of the Regional Conference on Tall Buildings, Bangkok, Asian Institute of Technology, Bangkok, Thailand.
- Melchers, R.E., (1999), Structural Reliability - Analysis and Prediction, John Wiley.
- Moore, D. B., (2002), "The U.K. and European Regulations for Accidental Actions," NIST/NIBS.
- Multihazard Mitigation Council National Workshop on Prevention of Progressive Collapse, Chicago, Ill. July. http://www.nibs.org/MMC/ProgCollapse presentations/Moore paper.pdf
- Mosallam, A.S., (2000), "Strength and Ductility of Reinforced Concrete Moment Frame Connections Strengthened with Quasi-Isotropic Laminates," Composites Part B: Engineering, Vol. 31, No. 6-7. October,

pp. 481-497.

- NationalBuilding Code of Canada (NBCC), (1985), Part 4 and Commentary C, National Research Council of Canada, Ottawa, Ontario.
- National Research Council (NRC), (2001), Protecting People and Buildings from Terrorism: Technology Transfer for Blast-effects Mitigation, Committee for Oversight and Assessment of Blast-effects and Related Research, National Research Council, National Academy Press, Washington, DC.
- National Research Council (NRC), (2000), Blast Mitigation for Structures: 1999 Status Report on the DTRA/TSWG Program, Committee for Oversight and Assessment of Blast Effects and Related Research, BICE, NationalAcademy Press, Washington, DC.
- Pate-Cornell, E., (1994), "Quantitative Safety Goals for Risk Management of Industrial Facilities," Structural Safety 13(3), pp. 145-157.
- PCI Committee on Precast Concrete Bearing Wall Buildings, (1976), "Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads," PCI Journal, Vol. 21, No. 2, March-April, pp. 18-51.
- Priestley, M. J. N. and Seible, F., (1995), "Design of Seismic Retrofit Measures for Concrete and Masonry Structures," Construction and Building Materials, Vol. 9, No. 6, pp. 365-377.
- SFPE, (2002), Guide to Performance-Based Fire Protection Analysis and Design of Buildings, Society of Fire Protection Engineers, Washington, DC.
- Somes, N. F., (1973), "Abnormal Loading on Buildings and Progressive Collapse," in Building Practices for Disaster Mitigation (Wright, Kramer and Culver, eds.), Building Science Series No. 46, National Bureau of Standards, Washington, DC.
- Starr, C., (1969), "Social Benefits Versus Technological Risk," Science, Vol. 168, pp. 1232-1238. Statistical Abstract of the United States, (2001), Department of Commerce, Washington, DC.
- Stewart, M.G. and Melchers, R.E., (1997), Probabilistic Risk Assessment of Engineering Systems, Chapman & Hall, London.
- Structural Stability Research Council (SSRC), (1998), Guide to Stability Design Criteria for Metal Structures, 5th Edition , T.V. Galambos, ed., John Wiley, New York.
- Taghdi, M, Bruneau, M., and Saatcioglu, (2000), "Seismic Retrofitting of Low-Rise Masonry and Concrete Walls Using Steel Strips, J. of Structural Engineering (ASCE), Vol. 126, No. 9, Septemebre, pp. 1017-1025.
- U.S. Army, (1990), Structures to Resist the Effects of Accidental Explosions," Technical Manual 5-1300, Department of the Army, Washington, DC
- van Mier, J. G. M., (1997), Fracture Processes of Concrete, CRC Press, Boca Raton, Fla., pp. 114-131.

Appendix A
COMPARISON OF DESIGN STANDARDS

A.1 GENESIS OF PROGRESSIVE COLLAPSE PROVISIONS

A.1.1 Ronan Point

The failure of an apartment building at Ronan Point, London, U.K., on May 16, 1968, brought the phenomenon of progressive collapse to the attention of the structural engineering community. A domestic gas explosion - estimated later to be between 14 kPa (2 psi) and 83 kPa (12 psi) - blew out a load-bearing flank wall (as well as a non-load bearing front wall) at a corner of an 18th floor apartment of this 24-story precast concrete building. The loss of support caused the floors above to collapse, and the impact and weight of the falling debris caused the floors below to collapse as well.

An inquiry found that there was no violation of any applicable building standards, nor any defect in workmanship in the design and construction of Ronan Point. It was revealed that the building standards, typically, gave detailed requirements for the design of individual members but provided little guidance for the stability design of the entire structural system. The walls in the Ronan Point building were unreinforced, and joint forces were resisted solely by bond, friction and gravity. The explosion reduced considerably, or reversed the gravity load, thus eliminating the friction force and bond between panels. Under these conditions, an estimated pressure of 21 kPa (3 psi) would have sufficed to blow out the panels clear of the building, which was designed for a wind pressure of 570 Pa or 12 psf (Firnkas, 1969). Upon removal of the walls, the connections above, which were designed for compression only, could not redistribute the loads nor provide alternate load paths, thus causing a chain reaction.

A.1.2 1968 U.K. Standards

On November 15, 1968, as a direct result of the inquiry of the disaster, the U.K. Ministry of Housing and Local Government issued "Standards To Avoid Progressive Collapse - Large Panel Construction", which listed two methods: "A) Provide alternate paths of support to carry the load, assuming the removal of a critical section of the loadbearing walls; and B) provide a form of construction of such stiffness and continuity as to ensure the stability of the building against forces liable to damage the load supporting members." The standards also specified an accidental static pressure of 34 kPa (or 5 psi, a town-gas explosion of "average intensity"), and derived minimum tie forces. These standards became part of the Fifth Amendment that the British Parliament approved in April 1970 as part of mandatory Building Regulations that required consideration of progressive collapse for buildings taller than five stories. Provisions for structural ties entered the British Standards in 1974.

These provisions, with certain modifications that put less emphasis on explosions and more on ductile performance, are still in force today in the U.K., namely, the notional removal of an essential structural element should cause only local collapse (70 m^2 or 750 ft^2 or 15 % of the plan area of the story), and buildings should be designed for an accidental pressure of 34 kPa or 5 psi acting simultaneously with dead and imposed loads. Similar provisions have been adopted in the Eurocode. The 2002 revision would allow more flexibility in the definition of the explosive pressure to be designed for, and the class of buildings where such considerations are required, based on the gravity of the consequences of failure. The City of New York has similar provisions on progressive collapse in its building standards since 1973, and is considering the adoption of provisions on ties similar to the British Standards (NYC-WTC 2003).

A.1.3 Initial Approach for Design Criteria:

Design provisions against progressive collapse are, by no means, universal. Structural engineers on the European mainland have pointed out that the collapse at Ronan Point might have been avoided, had the building been designed to the CEB 1967 Standards for the Design and Construction of Large Panel Structures, which warn of the "necessity of effectively joining the various components together to avoid the tendency for it to behave like a house of cards". In particular, Section R14 states:

"Horizontal joints and ties: Within the thickness of each floor, or close to it, continuous steel ties shall be provided in two directions; these ties shall interconnect the walls or façades on opposite sides of the building, shall include all the vertical panels, and shall be connected to all panels.

Peripheral ties: The total cross sectional area of the longitudinal reinforcement provided over a story height in a peripheral wall shall not be less than 2 cm^2 (0.3 $in.^2$) irrespective of the grade (strength or class) of steel employed.

Internal ties: The total cross sectional area of the tie reinforcement interconnecting two opposite external walls shall be able to carry a tensile force equal to 1 % of the direct force acting at the level considered on the external wall in question, but not less than 500 kg/m (4.9 kN/m or 340 lb/ft) of external wall. This cross sectional area may be concentrated at the cross walls or distributed in the floors".

In defense, Short and Miles (1969) pointed out, however, that similar general warnings were contained in British Code of Practice CP 116:1965, and had been made by engineers in the U.K. and elsewhere. Building standards in force in France before the Ronan Point disaster prohibited the use of cooking gas in buildings higher than 50 m (164 ft). If gas was used, ventilation requirements were more stringent than the British ones. Furthermore, structural continuity requirements were more demanding because design should be checked under normal and abnormal winds (Ferahian, 1972).

Good engineering judgment, design and construction practices should ensure redundancy, ductility and continuity, which are requisites for structural robustness and integrity. This view is adopted in a number of standards, for example, the ACI Code, which has recommendations for reinforcement continuity and connection details under the heading of "structural integrity," but makes no mention of progressive collapse.

A.1.4 Operation Breakthrough

In the U.S., much thought and research went into mitigating the problem of progressive collapse in the 1970s. "Operation Breakthrough" was launched on May 8, 1969, by the Department of Housing and Urban Development (HUD) to encourage industrialized housing concepts in the U.S. Proposals were requested for A) design, testing and evaluation of complete housing systems ready for production, or B) research and development of advanced concepts, materials and components not yet ready for production. To evaluate these innovative concepts, HUD consulted with the National Academies of Science and Engineering and commissioned the National Bureau of Standards (NBS) to develop performance criteria. Progressive collapse was of the utmost concern for the evaluation of concrete panel systems, and the criteria adopted by HUD were similar to the British Fifth Amendment:

"Explosions or other catastrophic loads on any one story level should not cause progressive structural collapse at other levels. The criterion applies to buildings four stories or higher. At a load level of 1.0 dead + 0.5 live, the accidental removal of any one of the following (load) supporting structural elements at one level should not cause collapse of the structure on another level:

a) two adjacent wall panels forming an exterior corner;

b) one wall panel in a location other than an exterior corner;

c) one column or other element of the primary structural support system.

This criterion is waived if the above-mentioned structural element or elements are capable of resisting a pressure of 5 psi (34.5 kPa), applied in the most critical manner within one story level to one face of the element and of all space dividers supported by the element or attached to it." (Building Research, 1970). Other "Breakthrough" criteria found in the 1971 HUD-FHA (Federal Housing Authority) "Provisions to Prevent Progressive Collapse" included:

"Joints between prefabricated structural elements used as columns, beams, bearing walls, or slabs should develop continuity similar to that provided by conventional cast-in-place concrete or structural steel framing systems. In regions not subject to severe seismic or wind action, connections should not be designed solely as gravity-type relying only on compression and friction. Where severe seismic forces are highly probable (Seismic Zones 2 and 1), connections, in particular should be examined. Their response to dynamic forces must be evaluated, e.g., vertical castellated and grouted joints may be completely satisfactory for quasi-static earthquake design loads, but could shake apart under actual dynamic, oscillatory earthquake forces. Joints between floor elements should develop adequate diaphragm action in order that the entire floor system may transmit lateral forces to the vertical elements. Peripheral ties completely encircling the building are also considered necessary to develop this diaphragm action. Vertical and horizontal joints between vertical structural elements must develop necessary continuity and deformability to transmit the lateral forces to the foundations." (Fuller, 1975).

The 1971 HUD-FHA criteria further stated that, if abnormal loading occurred, damage must be limited to 93 m^2 (1000 ft^2) or 20 % of horizontal floor area, whichever was less, and to three stories vertically. These criteria were similar to the British Standard Code of Practice 116, Addendum No. 1 for the Design of Large Concrete Panels (1970). Excerpts of the 1971 HUD-FHA "Provisions to Prevent Progressive Collapse" can be found in Popoff (1972), for example. These criteria were much more detailed and helpful to designers than the only other reference to progressive collapse that could be found at the time in U.S. building standards, namely, the 1972 American National Standards Institute A58, Minimum Design Loads in Buildings and Other Structures, which contained the following general recommendation: "Progressive collapse: Buildings and structural systems shall provide such structural integrity that the hazards associated with progressive collapse such as that due to local failure caused by severe overloads or abnormal loads not specifically covered herein are reduced to a level consistent with good engineering practice." (Somes, 1973).

As part of "Operation Breakthrough", Yokel, Pielert and Schwab (1975) examined five housing systems and concluded:

1. "The systems with clear spans between transverse bearing walls greater than 5.8 m (19 ft) had to use "strong" transverse bearing walls at least for the end walls and the transverse walls next to the end walls. In all cases, special provisions had to be made to provide lateral support to the end walls.

2.The systems with clear spans of 3.7 m (12 ft) or less relied principally on alternate paths of load support.

3. In short-span systems using an alternate path of load support the following joint reinforcement ties were the most critical: horizontal ties in the vertical joints between adjacent or intersecting bearing walls; continuous vertical ties throughout the building in the same joints; transverse horizontal ties between corridor floor panels and adjoining floor panels; and ties between transverse walls and corridor walls and between transverse walls and corridor floor panels. The alternate mode of load support was also assisted by longitudinal horizontal ties between adjoining floor panels on either side of transverse bearing walls, ties between transverse walls and connecting floor panels, and vertical ties between successive transverse bearing wall panels.

Thus depending on the span length, the systems used either the alternate path approach or a combination of the alternate path with strong points provided by specific local resistance.”

A.1.5U.S. Approach

Further U.S. efforts in the 1970s focused on quantifying the risk of various accidental loads, including a more rational determination of the pressures engendered by gas explosions, and the vulnerability of large panel, precast concrete structures. The results agreed in general with similar studies performed in Canada, the U.K., and the Netherlands. For buildings susceptible to progressive collapse in the U.S., the risk of fatality is comparable with that for fire (40 per million persons per year, Somes, 1973).

In the mid 1970s, HUD also commissioned the Portland Cement Association to develop standards for large panel structures. Particular attention was paid to internal floor and wall ties [Popoff (1975), PCI Committee on Precast Concrete Bearing Wall Buildings (1976)]. These studies indicated that U.S. building practices differed from European industrialized construction, for example joint details used in Ronan Point, which relied heavily on friction between elements, would be unacceptable in North America. U.S. large panel buildings are often connected by dry joints using bolting or welding, so construction can proceed more rapidly than if a masonry or concrete joint was used. Residential building layout is also different between European and American practices, as American buildings tend to have fewer intermediate walls and supports, and thus fewer alternate load paths. European walls are typically spaced at 4.6 m to 6.1 m (15 ft to 20 ft), compared with typical American spans of 6.7 m to 12.2 m (22 ft to 40 ft). Whereas European rooms tend to be surrounded by loadbearing walls, American living space tends to be defined by non-loadbearing partitions. The proportion of walls to slabs in U.S. construction is frequently as low as 1/3 of that of European buildings. Furthermore, the use of long, precast hollow-core slabs cut to length at U.S. job sites precludes protruding reinforcement used to develop continuity (Breen, 1980, Fintel and Schultz, 1976).

Mitigation of progressive collapse in U.S. buildings should therefore consider these differences rather than adopt European recommendations wholesale. Fewer vertical loadbearing elements in U.S. construction probably make the problem of stability of damaged buildings more severe than in Europe. Some U.S. engineers argue for the adoption of design principles regarding joints and continuity similar to those used for earthquake design as an approach to mitigate progressive collapse. Building standards would recommend minimum detailing requirements to ensure general structural integrity, and engineers would not have to directly consider abnormal loads or progressive collapse.

It is worth noting that 85 % of participants of the “Workshop on Progressive Collapse of Building Structures” held in Austin, Texas, in November of 1975, thought that “satisfactory control over progressive collapse can be provided by embodying in ACI 318 requirements for horizontal and vertical ties; and no reference need be made to “progressive collapse” either in the Code or Commentary.” (Breen, 1980). Even though the workshop participants agreed that substantial effort still needed to be made to justify tie forces, interest on the subject has not been strong in the U.S.

A.1.6 Other Early Studies

As one might expect, considerable testing was performed in the U.K. in the 1970s by the Building Research Establishment and ImperialCollege on alternate load paths in concrete panel structures (Wilford and Yu, 1973, Regan, 1975). Elsewhere in Europe, Hanson and Olesen (1969) studied keyed shear joints of prefabricated concrete wall panels in Denmark. In Sweden, Granstrom (1970) performed model tests of precast concrete buildings that had sustained local damage.

Progressive collapse has also been studied in Canada (Ferahian, 1972). Canadian researchers noted the rarity of progressive collapse. “As an analysis of newspaper articles shows, 75 incidents were reported in Canada in ten years from 1962 to 1972, of which almost 50 % occurred during construction. A well-known case occurred in February 1959 in Listowel, Ontario, where the local arena collapsed under high snow loads during a hockey game, and resulted in eight deaths and many injuries. Fracture of one of the laminated timber roof trusses led to a lateral progressive collapse of the whole roof and side walls." (The National Research Council of Canada, National Building Code of Canada, Commentary C4.1.1.8, 1975). A review of research on progressive collapse over the last 25 years is beyond the scope of this document.

A.1.7Present Interest

The risk of terrorist bombing has revived interest in the mitigation of progressive collapse. Studies of the Alfred Murrah building in Oklahoma and the KhobarTowers in Saudi Arabia (see Appendix C), both subjected to truck bombing, indicate that the kind of structural detailing recommended for seismic zones could be effective in reducing the risk of progressive collapse. Corley et al. (1998) recommended that compartmentalized construction, special moment frames, and dual systems be considered where a significant risk of seismic and/or blast damage exists. [A special moment frame is a frame in which members and joints are capable of resisting forces by flexure as well as along the axis of the members. A dual frame system is a structural system with an essentially complete space frame providing support for vertical loads. Seismic force resistance is provided by moment resisting frames and shear walls or braced frames (ASCE 7-05)].

Full scale tests of walls and columns subjected to explosions have been conducted by U.S. government civilian and military agencies and their contractors, to provide baseline behavior and measure the effectiveness of protective measures. Advanced computational methods have been applied to predict the behavior of individual structural members under blast, as well as that of entire buildings subjected to the sudden removal of a supporting column. Recent workshops have been held to map the direction of future research (July 10-12, 2002 [http://www.nibs.org/MMC/mmcactiv9.html] and September 10, 2001 (Carino and Lew, 2001)). This best practices report is one of the outgrowths of these workshops.

A.1.8Scope

This appendix provides a survey and comparison of existing building standards and standards for the mitigation of progressive collapse. Section A.2 is a general discussion of current approaches for new or existing buildings. Section A.3 compares provisions on progressive collapse from major international building standards. After a summary comparison, Section A.3.2.1 compares definitions of progressive collapse, also called disproportionate collapse, as opposed to local collapse. Section A.3.2.2 covers the threshold beyond which progressive collapse needs to be considered, usually having to do with the height of the building and the consequences of its failure. Section A.3.2.3 deals with various strategies against progressive collapse, namely mitigation of the hazard, direct design, and indirect design. Section A.3.2.4 presents load combinations recommended for consideration with accidental loads or local damage. Section A.3.2.5 covers the design of structural ties to ensure structural continuity and development of alternate load paths. Section A.3.2.6 covers the design of key elements, whose integrity is recognized to be crucial for the survival of the structure. Section A.3.2.7 deals with structural detailing to ensure continuity, ductility, and redundancy. Finally, Section A3.2.8 covers retrofitting of existing structures toimprove their resistance to progressive collapse. In this document, standards provisions are sometimes summarized and sometimes they are quoted verbatim.

A.2.APPROACHES TO MITIGATING PROGRESSIVE COLLAPSE

A.2.1 Requirements for New Buildings

This survey shows that a number of building standards around the world contain specific provisions for design against progressive collapse, whereas other standards rely on more general provisions dealing with structural integrity and robustness. They all emphasize the need for good structural layout, redundancy, ductility, and continuity.

A.2.1.1 Direct and indirect approaches

General approaches for mitigation of progressive collapse include minimizing exposure to hazards by preventive measures largely outside the scope of structural engineering; direct design methods, which include the alternate path method (bridging over local damage zones) and the specific local resistance method (hardening structural elements against specific hazards); and the indirect design method (provide strength, redundancy, continuity and ductility). Standards that address progressive collapse, whether by a direct or an indirect approach, usually contain specific requirements for tying together various structural elements within a building, as well as checking building stability under load combinations that take into account structural damage or accidental loads.

A.2.2 Requirements for Existing Buildings

For existing buildings, recent reports contracted by the U.S. government recommend that retrofit to improve resistance to progressive collapse should wait for other major renovation (such as seismic upgrade); or the decision should be based on the risk of exposure and consequences of failure, and the structural analysis would be similar to that of new buildings.

A.3 SUMMARY OF STANDARDS ON PROGRESSIVE COLLAPSE

The design standards that were included in the review are listed below. A brief description of each standard is provided here. More details are given in the Section A.4.

A.3.1 British Standards

Since shortly after the Ronan Point collapse, British Standards have taken the lead in stating explicit design provisions against progressive collapse. British Standards emphasize general tying of various structural elements of a building together, to provide continuity and redundancy. Ties enhance the resistance of wall panels to being blown away in the event of an explosion, and also the ability of a structure to bridge over a lost support. In designing for this possibility, various structural elements are considered lost one at a time. In addition, structural elements deemed vital to a building stability should be designed as key elements, able to withstand accidental loads, e.g., a pressure of 34 kPa (5 psi).

A.3.2Eurocode

Eurocode is a model code adopted by many European countries that may also supplement it with national standards (such as British Standards). In addition to providing general design guidelines to avoid progressive collapse, such as selection of a good structural layout, Eurocode also recommends tying the building together and defines values for tie forces. Buildings can be assigned to one of four safety classes, with only the two highest classes requiring consideration of progressive collapse.

A.3.3National Building Code of Canada

The National Building Code of Canada contains a general statement about the need for structural integrity, but its Commentary provides an extensive discussion on means to achieve that goal. The extent of the discussion reflects the importance accorded to the topic at the time, e.g., the 1975 version is much longer than the 1995 version. The Commentary covers recommendation for good structural layout, continuity of reinforcement, and structural mechanisms that would mitigate progressive collapse after local loss of support. No specific values are given for tie forces or accidental loads for key structural elements.

A.3.4Swedish Design Regulations

The Swedish Design Regulations BKR contains guidelines on the three safety classes of various buildings. Normally, requirements relating to accidental loads and progressive collapse only apply to Safety Class 3. These requirements are detailed in a separate handbook and consist of: a) checking the stability of a damaged building under dead and live loads, and b) checking that falling debris do not cause successive failure of floors by ensuring load transfer capability within floor structure and between floor and bearing walls (tension and shear forces of 20 kN/m or about 1400 lb/ft).

A.3.5ASCE 7

The commentary of ASCE 7-05 contains extensive discussion on general structural integrity. It lists the direct design approaches (alternate path method and specific load resistance method) and the indirect design approach. It provides design guidelines for general structural integrity, such as good plan layout and use of structural ties. As well, it recommends load combinations including extraordinary loads, and explains the underlying probabilities.

A.3.6ACI 318

The ACI 31-05 standard is an example of indirect design. It defines requirements for structural integrity such as continuity of reinforcement and use or ties in precast concrete construction.

A.3.7 New York City Building Code

The 1998 New York City Building Code is an example of direct design. It only mentions the alternate load path and the specific local resistance (34 kPa or 5 psi) methods.

A.3.8 Department of Defense Unified Facilities Criteria

Design for resistance to progressive collapse depends on the “level of protection” assigned to the building. For lower levels of protection the indirect design method is used by providing minimum tie forces. For higher levels of protection, the alternate load path method is used if sufficient ties cannot be provided.

A.3.9 Interagency Security Committee

The Interagency Security Committee (ISC) emphasizes the direct design methods (alternate load pathsand specific local resistance) and makes no mention of the indirect method or structural ties.

A.3.10General Services Administration

The General Services Administration (GSA) guidelines are based on the alternate load path method and removal of vertical load carrying members.

A.4COMPARISON OF PROGRESSIVE COLLAPSE PROVISIONS

A.4.1Provisions Compared

Following is a detailed comparison of provisions regarding progressive collapse from major international building standards. The comparison is organized by the following topics:

- Definition of progressive collapse, local collapse, and structural integrity
- Threshold for consideration of progressive collapse
- General strategy
- Loads
- Ties
- Key elements
- Continuity, ductility, and details
- Existing structures

A.4.2 Definition of Progressive Collapse, Local Collapse, and Structural Integrity

Most definitions of progressive collapse encompass the “house of cards” effect, whereby damage spreads beyond a local region, to an extent disproportionate to the initial cause (some unanticipated load). Damage is assumed local if it is limited to 15 % or 20 % of floor or roof area, or 100 m^2 (or about 1000 ft^2), depending on the standards; or to one structural bay or the floors immediately adjacent to the initial damage. Breen (1980) defines progressive collapse as an incremental type of failure, where the total damage is out of proportion to the initial cause. The word incremental eliminates from consideration the total collapse of statically determinate structures upon loss of a single member. This wording has not caught on, probably because the word incremental requires an explanation, just as the word progressive does.

In classifying structural collapses as progressive or not, Allen and Schriever (1973) found it convenient to use the “rule of three”: a collapse is progressive if it involves members that are three or more members away from the original failure or if three or more spans collapse. They concluded their survey by saying that, “only a few of the reported incidents should definitely be considered structurally from the point of view of progressive collapse.”

Because what constitutes local collapse has consequences on the number of casualties and the load of debris that must be resisted by the damaged structure to stop the progress of collapse, social and technical justifications for such definitions need to be developed further. For example, Leyendecker and Ellingwood (1977) proposed limiting damage in any story to 70 m^2 (750 ft^2) or 15 % of the floor area. This would limit the total average annual fatalities to less than the mortality associated with fires, and two orders of magnitude less than that associated with automobile accidents.

Following are the definitions of progressive collapse, local collapse, and structural integrity used in various building standards: British Standards BS 5950-1:2000, Structural Use of Steelwork in Building, Section 2.4.5.3—The British Standards do not use the words progressive collapse but rather structural collapse disproportionate to the initial cause. This contrasts with local collapse, which is limited to 15 % of floor or roof area or 100 m^2 (about 1000 ft^2), whichever is less, at the relevant level and at one immediately adjacent level, either above or below it. National Research Council of Canada, National Building Code of Canada (NBCC)—The level of detail about progressive collapse contained in the Commentary of the National Building Code of Canada has evolved over time. In 1975, Commentary C4.1.1.8 contained the following:

> “Progressive collapse is the phenomenon in which the spread of an initial local failure from element to element eventually results in the collapse of a whole building or disproportionately large parts of it.”

In the 1977 version of Commentary C4.1.1.8, there was the following discussion:

"Progressive collapse is the spread of an initial local failure from element to element eventually resulting in structural collapse disproportionate to the initial cause or to the initial local damage."

In an attempt to define "disproportionate," it was added that: "collapse should probably be limited

a) where the progression might be vertical, to the story where the abnormal event occurred and the story immediately above and below, and

b) where the progression might be horizontal,

• to the truss, beam or precast strip floor or roof panel initially damaged and perhaps to one on either side,

• to one bay of a full bay-sized floor or roof slab, except that where the principal support at one end of a slab is removed, two bay-sized slabs may hang together as a catenary".

In 1990, Commentary C4.1.1.3 introduced the term "structural integrity" with the following definition:

"Structural integrity is defined as the ability of the structure to absorb local failure without widespread collapse."

American Society of Civil Engineers (2005), ASCE 7-05—Section 1.4 on General Structural Integrity contains the following statement:

"Buildings and other structures shall be designed to sustain local damage with the structural system as a whole remaining stable and not being damaged to an extent disproportionate to the original local damage."

Commentary C1.4 provides the following definition:

"Progressive collapse is defined as the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it."

New York City Building Code (1998)—Chapter 18 on Resistance to Progressive Collapse Under Extreme Local Loads contains the following description of what constitutes a disproportionate spread of the initial local failure:

"…progressive collapse is interpreted as structural failure extending vertically over more than three stories, and horizontally over an area more than 1000 ft^2 [100 m^2] or 20 percent of the horizontal area of the building, whichever is less."

New York City Department of Buildings, World Trade Center Building Code Task Force—The February 2003 report on the findings and recommendations of the task force contained the following definition:

"Progressive collapse is the propagation of collapse to an extent disproportionate to the initiating zone of damage and is interpreted as structural failure beyond the point of initial damage extending vertically over more than three stories, and horizontally over an area more than one structural bay or 20 percent of the horizontal area of the building, whichever is less."

Department of Defense (2005) Unified Facilities Criteria (UFC) 4-023-03, Design of Buildings to Resist Progressive Collapse—The definition of progressive collapse given in ASCE 7 is adopted. The following describe the permitted "damage limits" for the spread of damage, as determined by structural analysis, resulting from the notional removal of a vertical load-bearing element (DOD 2005):

“3-2.6.1 Damage Limits for Removal of External Column or LoadBearing Wall For the removal of a wall or column on the external envelope of a building, the Damage Limits require that the collapsed area of the floor directly above the removed element must be less than the smaller of 70 m^2 (750 ft^2) or 15 % of the total area of that floor and the floor directly beneath the removed element should not fail. In addition, any collapse must not extend beyond the structure tributary to the removed element.”

“3-2.6.2 Damage Limits for Removal of Internal Column or LoadBearing Wall For the removal of an internal wall or column of a building, the Damage Limits require that the collapsed area of the floor directly above the removed element must be less than the smaller of 140 m^2 (1500 ft^2) or 30 % of the total area of that floor, and the floor directly beneath the removed element should not fail. In addition, any collapse must not extend beyond the bays immediately adjacent to the removed element.”

GSA Progressive Collapse Analysis and Design Guidelines for NewFederalOfficeBuildings and Major Modernization Projects (June 2003)—Section 2 on Definitions provides the following:

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

A.4.3 Threshold

Of the building standards that contain provisions for structural integrity, some make no mention of thresholds, and by default, apply these provisions to all buildings. Other standards recommend that progressive collapse only needs to be considered for buildings that are above a certain height (3 stories to 5 stories), or whose failure could cause severe loss of human life (consequence class).

BS 5950-1:2000, Structural Use of Steelwork in Building, Section 2.4.5 Structural Integrity and BS 8110-1:1997, Structural Use of Concrete, Section 2.2.2.2 Robustness—All buildings

BS 5628-1:1992, Code of Practice for Use of Masonry, Section 37.1 General Considerations— Category 2 buildings (five or more stories) require consideration of notional removal of a load-bearing element or provision of ties or both.

BS 5268-2:2002, Structural Use of Timber, Section 1.6.3 Accidental Damage—No special robustness requirements for buildings not exceeding four stories.

Department of Defense Unified Facilities Criteria (UFC) 4-010-01 Minimum Antiterrorism
Standards for Buildings (2003)—Standard 6 on progressive collapse avoidance states that buildings of three stories or more require consideration against progressive collapse in the context of terrorist threats. Basements are considered stories if they have one or more exposed walls.

Eurocode 1 - Section 2 - Actions on Structures, Part 1 - Basis of Design (CEN 250 1994) (pre EN 2002)—Four consequence classes are given::

1. Low: No consideration required for accidents.
2. Medium: No consideration beyond robustness and stability rules given in Eurocode 1 to 9.
3. High: Simplified analysis by static equivalent actions, or prescriptive design/detailing rules applicable.

4. Severe: Dynamic, non-linear analysis, load-structure interaction may be applicable.

A table shows the types of buildings associated with each consequence class:

1. Houses of three stories or less.
2. Houses between three and six stories. Offices of less than four stories.
3. Buildings of ten stories or less, public buildings of less than 200 m^2 (2200 ft^2).
4. Buildings of more than ten stories, public buildings of more than 200 m^2 (2200 ft^2).

Swedish Board of Housing, Building and Planning (Boverket, June 2000); Design Regulations BKR: Mandatory Provisions and General Recommendations, BFS 1993:58 with amendments up to BFS 1998:39, BFS 1999:7 and BFS 1999:46—The Swedish provisions recommend consideration against progressive collapse for all buildings in Safety Class 3, defined as those whose collapse would cause great risk of serious injury to humans; and for floors of multi-story buildings assigned to Safety Class 2, defined as those whose collapse would cause some risk of serious injury to humans. Structural elements in multi-story buildings are assigned to Safety Class 3 if their failure would cause collapse of a floor area greater than 150 m^2. Following are relevant excerpts from the Boverket 2000 recommendations:

"Special measures need not be taken in buildings in which the risk of serious accidents due to progressive collapse is slight, or in buildings which are so small that primary damage causes total destruction."

"The requirement relating to accidental actions and progressive collapse normally applies only to elements of structure assigned to Safety Class 3." [Safety Class 3 means great risk of serious injury to persons]

"In addition to the safety class requirement, which relates only to injury to persons, the building owner may stipulate more stringent requirements, for instance with respect to property damage."

"In selecting the safety class, the following principles shall be applied."

"Elements of structure shall be assigned to Safety Class 3 if the following conditions simultaneously apply:
- the design and use of the building are such that many persons are often present in or in the vicinity of the building,
- the element of structure is of such nature that collapse involves a high risk of injury to persons,
- and the element of structure has properties such that failure causes immediate collapse."

"The classification of other elements of structure shall be not lower than Safety Class 2."

"Examples of the choice of safety class.

"A. Buildings of two and more storeys, of the type residential building (with the exception of single-dwelling houses), office buildings, department stores, hospitals and schools."

“The following elements of structure should be assigned to Safety Class 3:

• The main structural system of the building inclusive of those elements of structure which are of essential importance for the stability of the system.

• Other structural elements such as columns, beams, shear panels, whose failure causes the collapse of a floor area > 150 m^2.

• Stairs, balconies, access balconies and other elements of structure which form part of the escape routes of the building.”

“The following elements of structure should be assigned to Safety Class 2:

• Floor beams not assigned to Safety Class 3.

• Floor slabs.

• Roof construction with the exception of lightweight stressed skin elements of non-brittle materials.

• Those parts of heavy external wall constructions (mass > 50 kg/m^2) which are situated higher than 3.5 m above ground level and which do not form part of the main structural system of the building.

• The fixings of external wall constructions which are situated higher than 3.5 m above ground level and which do not form part of the main structural system of the building.

• Heavy partitions (mass > 250 kg/m^2) which do not form part of the main structural system of the building.

• The fixings of heavy ceilings (mass > 20 kg/m^2).

• Stairs which are not assigned to Safety Class 3.”

“B. Single storey buildings of the open plan type whose roofs are of large span (> 15 m) and which are used as sports halls, exhibition halls, places of assembly, department stores, schools and industrial premises in which many people are present. (BFS 1998:39)”

“The following elements of structure should be assigned to Safety Class 3:

• The main structural system of the building inclusive of wind bracing and the stabilizing system.

• The barriers of stands etc. erected where there are large differences in level and where a large number of people may be present.

• Structures which carry large overhead cranes (> 15 m span and > 20 tonnes lifting capacity).”

Swedish Board of Housing, Building and Planning - Boverket, 1994, Handbook on Vibrations, Induced Deformations and Accidental Loads—Section 4.4 on Dimensioning states the following:
“For buildings with several stories, however, progressive collapse of floor structures within Safety Class 2 (some risk of serious injury to persons) must be considered. This is required to avoid the collapse of a floor structure onto the lower floor structures, which might bring about progressive collapse.”
PCI Committee on Precast Concrete Bearing Wall Buildings (1976)—The Precast Concrete Institute (PCI) recommends the use of horizontal ties in all buildings and, in structures over two stories in height, vertical ties also. Table A1 compares the threshold for consideration of progressive collapse specified in various building standards. Besides the information discussed above, Table A2 also includes the criteria adopted by GSA in 2000, in which buildings may be exempted from having to consider progressive collapse if certain conditions are satisfied.

Table A1 Threshold for consideration of progressive collapse

British Standards	
Steel	All buildings
Concrete	All Buildings
Masonry	Buildings ≥5 stories
Timber	Buildings ≥5 stories
Department of Defense - UFC 4-010-01	Buildings ≥3 stories
Eurocode 2002 **Consequence Classes 1 to 4:**	
1) Low: 1 to 3 stories. No consideration.	
2) Medium: 3 to stories, offices < 4 stories: Eurocode robustness and stability rules.	
3) High: 7 to 10 stories, public buildings < 200 m^2: Simplified static	
4) Severe: > 10 stories, public buildings >200 m^2: Dynamic,	
Swedish Regulations **Safety Classes 1 to 3:**	
1) Little risk of serious injury. No consideration.	
2) Some risk of serious injury. Consider only in multi-story buildings.	
3) Great risk of serious injury. Mandatory consideration.	
GSA Guidelines (2003) Exemption flowcharts regarding use, occupancy, building type, proximity of moving or parked vehicles, seismic design, and others.	
Precast Concrete Institute (1976) Horizontal ties in all buildings.Vertical ties in buildings over two stories	

A.4.4General strategy

With varying emphasis, most standards refer to three methods of mitigating progressive collapse. The first is to reduce exposure to hazards, for example, by erecting protective barriers against vehicular impact or increasing standoff distance against terrorist bombs, or forbidding the use of cooking gas in high-rise buildings. The other two methods are more under the control of structural engineers. The second method explicitly considers resistance to progressive collapse during the design process and is therefore called the direct design method. It can itself be subdivided into two methods:

- The specific local resistance method, which designs against specific accidents or misuse by providing sufficient strength to resist failure, and
- The alternate path method[4], which accounts for the possibility of local failure, and provides, by design, redundant, alternate load paths that bridge over the failed members and prevent collapse from progressing. In this method, ultimate strength analysis that accounts for plastic or large deformations, as well as catenary or membrane action may be an appropriate tool. Finally, the third method, called the indirect design method, considers implicitly resistance to progressive collapse by providing minimum levels of strength, continuity, and ductility. It also includes built-in planes of weakness to control the spread of collapse.

[4] A good description of the alternate path method is given by James E. Eads at the 1874 inauguration of his St. Louis Bridge: “The peculiar construction of the superstructure is such that any piece of it can be easily taken out and examined, and replaced or renewed, without interrupting the traffic of the bridge… In completing the western span, two of the lower tubes of the inside ribs near the middle of the span were injured during erection, and were actually ncoupled and taken out without any difficulty whatever, after the span was completed, and two new ones put in their place in a few hours.” (Morgan, 1971, quoted in Allen and Shriever, 1972). Allen and Shriever (1972) reported that the claim was further validated in 1969, when a tugboat knocked out a portion of the lower chord of one of the arches and did not cause progressive collapse.

It is generally agreed that the following approaches mitigate progressive collapse:

- select a good floor layout; consider structural isolation of various building parts;
- use ductile connections;
- tie the building together, with peripheral, internal and vertical ties; and
- design for load reversal (uplift), change of span direction, and membrane action in floor slabs.

A.4.7 Key Elements

Key elements are defined as structural elements whose notional removal would cause collapse of an unacceptable extent. They should therefore be designed for accidental loads, which are specified in several standards as 34 kPa or 5 psi. The origin of this value was discussed in Section A.3.2.4: Loads.

The difficulty with strengthening key elements is that it must be done with a specific threat in mind. In this context, it is instructive to learn from controlled demolition experts, who report, for example, using only 150 kg of explosives judiciously placed to bring down a 22-story building (Anonymous, 1985). Typically the structure to be demolished has to be pre-weakened by removing many redundancies, such as internal partition walls, stairwell or elevator walls (Williams, 1990). Following are details of various building standards:

BS 5950-1:2000, Structural Use of Steelwork in Building, Section 2.4.5 Structural Integrity

> "If the notional removal of a column, or of an element of a system providing resistance to horizontal forces, would risk the collapse of a greater area, that column or element should be designed as a key element… Any other steel member or other structural elements that provide lateral restraint vital to the stability of a key element should itself also be designed as a key element for the same accidental loading."

BS 8110-2:1985, Structural Use of Concrete, Sections 2.6.2.2-3

> "In all cases, a key element and its connections should be capable of resisting a design ultimate load of 34 kN/m^2 (5 psi), to which no partial factor of safety should be applied, from any direction."

BS 5628-1:1992, Code of Practice for Use of Masonry, Section 37.1.1

> "Protected members or key elements shall be designed to resist reduced design load and an accidental load of 34 kN/m^2 (5 psi) applied from any direction."

New York City Building Code (1998), Chapter 18, Resistance to Progressive Collapse Under Extreme Local Loads—

> "Specific load resistance methods.
>
> Any single element essential to the stability of the structure, together with its structural connections, shall not fail under the loads stipulated in this criterion after being subjected to a load equivalent to that caused by a uniform pressure of 720 psf [5 psi or 34 kPa]."

New York City Department of Buildings World Trade Center Building Code Task Force (2003)—

Under the heading of "The Specific Local Resistance Method," the draft progressive collapse guidelines recommend that key elements should be hardened locally against unanticipated loads without failing the connections or supporting members framing it. The structure should be detailed to permit load reversals.

Special attention is given to transfer structures, which, by definition, concentrate the load bearing system onto fewer structural elements.

> “Transfer structures should be continuous over several supports with substantial structure framing into these members to create a two-way redundancy that provides an alternate load path in the event of a localized failure. The column connections, which support the transfer structures should provide sustained strength despite inelastic deformations and designed as full moment connections. Transfer structures and the columns that support the transfer members should be hardened to the requirements of the specific local resistance.”

Department of Defense (2005) Unified Facilities Criteria (UFC) 4-023-03, Design of Buildings to Resist Progressive Collapse—The following guidance is provided for designing to resist a specific threat:

> “As the initiating event is unknown, the requirements in this UFC are not intended to directly limit or eliminate the initial damage. This is consistent with UFC 4-010-01, which applies where there is a known risk of terrorist attack, but no specific terrorist threat is defined; in this case, the goal is to reduce the risk of mass casualties in the event of an attack. For cases where specific explosive threats against a building have been identified, design guidelines for specific blast hardening can be found in UFC 4-013-01Structural Design to Resist Explosives Effects for New
>
> Buildings and UFC 4-013-02Structural Design to Resist Explosives Effects for Existing Buildings.Even if a structure is designed to resist an identified or assumed threat, the progressive collapse requirements of this UFC will still apply.”

A.4.8 Continuity, Ductility, and Other Details

Continuity of reinforcement, anchorage and joint requirements are specified in various standards to promote catenary action, resistance to uplift forces, and general structural integrity. Good details between structural elements are particularly important for prefabricated elements. A question that needs further study is to what extent structural details designed to resist earthquakes also help resist progressive collapse. Ferahian (1972) showed that structural elements designed to withstand an El-Centro earthquake should be capable of resisting a gas explosion also. The GSA (2000) design guidelines against progressive collapse rely heavily on seismic criteria.

Following are details from various building standards:

BS 5950-1:2000, Structural Use of Steelwork in Building, Section 2.4.5 Structural Integrity—

> "Where precast concrete or other heavy floor or roof units are used, they should be effectively anchored in the direction of their span, either to each other or over a support, or directly to their supports as recommended in BS 8110."

BS 8110-1:1985, Structural Use of Concrete, Section 2.6.3.1—

> "At each story in turn, each vertical load-bearing element other than a key element is considered lost in turn… If catenary action is assumed, allowance should be made for the horizontal reaction necessary for equilibrium."

BS 8110-1:1997, Structural Use of Concrete, Section 5.1.8.4, 3.12.3.2—

> "In buildings of five or more storeys where precast floor or roof members are not used to provide the ties required by 3.12.3, they should nevertheless be effectively anchored, such anchorage being capable of carrying the dead weight of the member, to that part of the structure which contains the ties."

Ties connecting floor and roof members should be arranged to minimize out of balance effects, i.e., minimize eccentricity. Proper anchorage and lapping of ties are required.

> "Bars should be lapped, welded or mechanically joined in accordance with 3.12.8.9. A tie may be considered anchored to another tie at right angles if the bars of the former tie extend:
>
> a) 12 diameters or an equivalent anchorage beyond all the bars of the other tie; or
>
> b) an effective anchorage length (based on the force in the bars) beyond the centre-line of the bars of the other tie. At re-entrant corners or at substantial changes in construction, care should be taken to ensure that the ties are adequately anchored or otherwise made effective."

Eurocode 2 - Design of concrete structures, Part 1, (prEN 1992-1-1: July 2002). General rules and rules for buildings, Section 9.10.3—

> "Ties in two horizontal directions shall be effectively continuous and anchored at the perimeter of the structure. They may be provided wholly within the in-situ concrete or at connections. Where ties are not continuous in one plane, the bending effects resulting from the eccentricities should be considered. Ties should not normally be lapped in narrow joints between precast units. Mechanical anchorage should be used in these cases."

American Concrete Institute International, Building Code Requirements for Structural Concrete and Commentary ACI 318-05 and 318R-05 (2005)—

> 7.13 Requirements for Structural Integrity, and R7.13 Commentary
>
> "7.13.1—In the detailing of reinforcement and connections, members of a structure shall be effectively tied together to improve integrity of the overall structure."
>
> "7.13.2.2—Beams along the perimeter of the structure shall have continuous reinforcement consisting of:
>
> a) at least one-sixth of the tension reinforcement required for negative moment at the support, but not less than two bars; and
>
> b) at least one-quarter of the tension reinforcement required for positive moment at midspan, but not less than two bars."
>
> "7.13.2.4—In other than perimeter beams, when stirrups as defined in 17.13.2.3 are not provided, at least one-quarter of the positive moment reinforcement required at midspan, but not less than two bars, shall be continuous or shall be spliced over or near the support with a Class A tension splice or a mechanical or welded splice satisfying 12.14.3, and at noncontinuous supports shall be terminated with a standard hook."
>
> "R7.13.3—…Connection details that rely solely on friction caused by gravity forces are not permitted."

A.4.9 Existing Buildings

Interagency Security Committee (ISC), Design Criteria for New FederalOfficeBuildings and Major Reorganization Projects (2001 Draft)

> Section 4.C Existing Construction Modernization, 4.C.2 Progressive Collapse
>
> "Existing buildings will not be retrofitted to prevent progressive collapse unless they are undergoing a structural renovation, such as a seismic upgrade. Prior to the submission for funding, all structures shall be analyzed according to requirements for new construction, and a written report shall clearly state the potential vulnerability of the building to progressive collapse."

General Services Administration (GSA), Progressive Collapse Analysis and Design Guidelines For New FederalOfficeBuildings and Major Modernization Projects (2003)

The GSA guidelines incorporate an exemption process that takes into account the use, occupancy, and type of the facility, proximity of moving or parked vehicles, as well as structural features such as seismic design, to help the user decide whether the potential for progressive collapse needs to be considered. The following statement is provided for existing construction:

> "For existing construction, if the facility is determined no to be exempt from further consideration for progressive collapse, the methodology for existing construction outlined in Section 4.2 or 5.2, as applicable, shall be executed. The potential for progressive collapse determine3d in this process (whether low or high) must be quantified and analysis procedure and results documented."

Section 4.2 is for reinforced concrete structures and section 5.2 is for steel structures. The following guidance is provided for existing reinforced concrete facilities:

> "4.2 Existing Construction
>
> Existing facilities undergoing modernization should be upgraded to new construction requirements when required by the project specific facility security risk assessment and where feasible. In addition, facilities undergoing modernization should, as a minimum, assess the potential for progressive collapse as the result of an abnormal loading event. The flowchart, shown in Figure 4.8, outlines the process for assessing the potential for progressive collapse in existing facilities. Findings of this analysis should be incorporated into the project-specific risk assessment, and shall be documented in accordance with the provisions in Section 1.5. The 'analysis' provisions contained in Section 4.1.2 concerning analysis techniques, procedure, analysis considerations and loading criteria, analysis criteria, material properties, and modeling guidance, shall also apply to existing construction."

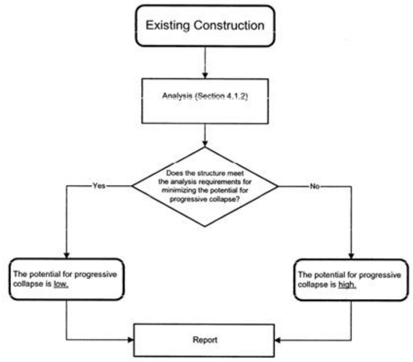


Figure 4.8 Process for assessing the potential for progressive collapse in existing construction.

Section 5.2 provides a similar statement for steel structures. Refer to A.2.3 for a summary of the analysis procedures regarding notional removal of supporting members and calculating extent of damage.

Department of Defense (2005) Unified Facilities Criteria (UFC) 4-023-03, Design of Buildings to Resist Progressive Collapse—The UFC for design to resist progressive collapse are applicable equally to new and existing facilities. The following actions are specified for different levels of protection:

> Very Low Level of Protection—“If a structural element does not provide the required horizontal tie force capacity, it must be re-designed in the case of new construction or retrofitted in the case of existing construction.”
>
> Low Level of Protection—“For elements with inadequate horizontal tie force capacity, the Alternate Path method cannot be used. In this case, the designer must re-design the element in the case of new construction or retrofit the element in the case of existing construction.”
>
> Medium and High Level of Protection—“For elements with inadequate horizontal tie force capacity, the Alternate Path method cannot be used. In this case, the designer must re-design the element in the case of new construction or retrofit the element for existing construction.”

A.5SUMMARY

This survey shows that a number of building standards around the world contain specific provisions for design against progressive collapse, whereas other standards rely on more general provisions dealing with structural integrity and robustness. They all emphasize the need for good structural layout, redundancy, ductility, and continuity. General approaches for mitigation of progressive collapse include minimizing exposure to hazards by preventive measures largely outside the scope of structural engineering; direct design methods, which include the alternate path method (bridging over local damage zones) and the specific local resistance method (hardening structural elements against specific hazards); and the indirect design method (provide strength, redundancy, continuity and ductility). Specific provisions also address requirements for tying together various structural elements within a building, constructing planes of weakness to limit the spread of damage, as well as checking building stability under load combinations that take into account damage or accidental loads.

When remaining structural elements must bridge over damaged ones, they oftentimes must perform very close to their ultimate strength. For masonry structures in the U.S., the advent of strength methods with the 2002 Building Code Requirements for Masonry Structures (ACI 530-02/ASCE 5-02/TMS 402-02) should make it easier to consider provisions against progressive collapse. This may be more challenging than for concrete structures, because masonry wall panels themselves, and not just the joints, may fail.

Catenary or membrane behavior of slabs with ties has been tested in the U.K., with a maximum deflection of 15 % of span. For U.S. practice, it is desirable to test longer spans, under larger deflections, up to 50 % of story height. It is also necessary to ensure that connections and splices can withstand such high level of deflection and rotation. Tests at PCA of large panel construction confirm the importance of proper details and analysis for the shear capacity of horizontal joints (Breen 1980). More research, that takes into account the possible shifting of load application points, for example, is needed in this area.

For existing buildings, recent reports contracted by the U.S. government recommend that retrofit to improve resistance to progressive collapse should wait for other major renovation (such as seismic upgrade); or the decision should be based on the risk of exposure and consequences of failure, and the analysis would be similar to that of new buildings. Analytical methods that account for nonlinear geometric and material properties, and various member failure criteria can be very challenging (see, for example, Blandford, 1997).

A.6 REFERENCES

Building Standards

American Concrete Institute, (2005), “Building Code Requirements for Structural Concrete and Commentary,” ACI 318-05 and 318R-05, Section 7.13.

American Institute for Steel Construction, (2001), “Manual of Steel Construction: Load and Resistance Factor Design,” 3rd Ed.

American Society of Civil Engineers (ASCE), (2005), “Minimum Design Loads for Buildings and Other Structures,” SEI/ASCE 7-05, Reston, Va.

AustralianBuilding Code Board, (1996), “Building Code of Australia.”

Australian/New Zealand Standard, (2002), “Structural Design Actions, Part 0: General Principles,” AS/NZS 1170.0:2002.

Australian/New Zealand Standard, (2002), “Structural Design Actions: General Principles - Commentary,” AS/NZS 1170.0 Supplement 1:2002.

Boverket (Swedish Board of Housing, Building and Planning) BFS 1993:58 with Amendments up to BFS 1998:39, BFS 1999:7, BFS 1999:46, “Design Regulations BKR-Mandatory Provisions and General Recommendations.”

Boverket (Swedish Board of Housing, Building and Planning), 1994 Handbook on Vibrations, Induced Deformations and Accidental Loads

British Standards Institute, (2000), “Structural Use of Steelwork in Building, Part 1: Code of Practice for Design - Rolled and Welded Sections,” BS 5950-1:2000.

British Standards Institute, (2002), “Structural Use of Timber, Part 2: Code of Practice for permissible stress design, materials and workmanship,” BS 5268-2:2002.

British Standards Institute (1992), “Code of Practice for Use of Masonry Part 1: Structural Use of Unreinforced Masonry,” BS 5628-1:1992.

British Standards Institute, (1997), “Structural Use of Concrete, Part 1: Code of Practice for Design and Construction,” BS 8110-1:1997.

British Standards Institute (1985), “Structural Use of Concrete, Part 2: Code of Practice for Special Circumstances,” BS 8110-2:1985.

British Standards Institute, (1996), “Loading for Buildings, Part 1: Code of Practice for Dead and Imposed Loads,” BS 6399-1:1996.

British Standards Organization, (1998), European Prestandard ENV 1991-2-7:1998 Eurocode 1: 'Basis of design and actions on structures, Part 2-7: Accidental actions due to impact and explosion', London.

CAN/CSA-S16-01, “Limit States Design of Steel Structures,” Section 6.1.2, Structural Integrity.

Comité Européen de Normalisation, (2002), Draft prEN 1991-1-7: March 2002, Eurocode 1 - Actions on structures. Part 1.7: General Actions - Accidental actions due to impact and explosions. First Project Team (Stage 32) draft, Amended version 8, Brussels.

Comité Européen de Normalisation, (2002), Draft prEN 1992-1-1: July 2002, Eurocode 2 - Design of concrete structures, Part 1: General rules and rules for buildings, Brussels.

ComitéEuropéen de Normalisation, (1994), ENV 1992-1-3:1994 E, Eurocode 2 - Design of concrete structures, Part 1.3: General rules - Precast concrete elements and structures, Section 5.5: Limitation of damage due to accidental action

Comité Européen de Normalisation, (1996), DD ENV 1992-1-5: 1996, Eurocode 2 - Design of concrete structures, Part 1.5: General rules - Structures with unbonded and external prestressing tendons.

Comité Européen de Normalisation, (1994), ENV 1991-1:1994 E, Eurocode 1- Basis of design and actions on structures, Part 1: Basis of design.

Comité Européen de Normalisation Du Batiment (General Commission for Building Standards- France) BAEL 91: Regles Techniques De Conception Et De Calcul Des Ouvrages Et Constructions En Beton Arme Suivant La Methode Des Etats Limites (Technical Rules On Limit State Design And Calculation Of Reinforced Concrete Structures And Construction).

Department of Defense (DOD), (2003), “DOD Minimum Antiterrorism Standards for Buildings,” Unified Facilities Criteria (UFC) 4-010-01, 8 October, 2003.

Department of Defense, (DOD), (2005), “Design of Buildings to Resist Progressive Collapse,” Unified Facilities Criteria (UFC) 4-023-03, 25 January, 2005.

Department of Defense, (DOD), (2001), “Interim Antiterrorism/Force Protection Construction Standards, Guidance on Structural Requirements,” Draft, March.

Eurocode No. 1, (1990), “Common Unified Rules for Different Types of Construction and Material,” Commission of the European Communities, Brussels.

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

Federal Emergency Management Agency, (2000), “Prestandard and Commentary for the Seismic Rehabilitation of Buildings,” FEMA 356, Nov.

General Services Administration (GSA), (2003), “Progressive Collapse Analysis and Design Guidelines for New FederalOfficeBuildings and Major Modernization Projects,” June.

Interagency Security Committee, (2001), “Design Criteria for New FederalOfficeBuildings and Major Reorganization Projects”

Japan Ministry of Land, Infrastructure and Transport, (2002), Building Standard Law and Enforcement Order

National Research Council of Canada, (1975, 1977, 1980, 1990, 1995), "NationalBuilding Code of Canada," Ottawa, Canada.

New York CityBuilding Code, (1998), "Resistance to Progressive Collapse Under Extreme Local Loads," Chapter 18.

New York City Department of Buildings, World Trade Center Building Code Task Force, (Feb. 2003), "Draft Progressive Collapse Guidelines," Section 5.2 of Findings and Recommendations, http://www.nyc.gov/html/dob/html/publications.html

New Zealand Standard, (1982), Code of Practice and Commentary for the Design of Concrete Structures, Parts 1 and 2. NZS-3101-1982, Standards Association of New Zealand, Wellington.

New Zealand Standard, (1992), Code of Practice for General Structural Design and Design Loadings for Buildings (known as the Loadings Standard): Volumes 1and 2, Code of Practice and Commentary, Standards Association of New Zealand, Wellington.

Norges Byggstandardiseringsrad (Norwegian Council for Building Standardization) NS 3473 E, "Concrete Structures Design Rules", 4th Ed. Nov. 1992.

PCI Committee on Precast Concrete Bearing Wall Buildings, (1976), "Considerations for the Design of Precast Concrete Bearing Wall Buildings to Withstand Abnormal Loads," PCI Journal Vol. 21, No.2, March-April, pp. 18-51.

Standards Association of Australia, (1989), "Minimum Design Loads on Structures (known as SAA Loading Code)" Part 1: Dead and Live Loads and Load Combinations AS 1170.1-1989.

Standards Association of Australia, (1994), "Concrete Structures" AS 3600-1994. Standards Association of Australia, (1998), "Masonry Structures" AS 3700-1998. Standards Association of Australia, (1998), "Steel Structures" AS 4100-1998.

Standards Association of Australia, (1996), "SAA Timber Structures Code: Part 1: Design Methods" AS 1720.1-1996.

Standards Association of Australia, (1992), "National Timber Framing Code" AS 1684-1992.

Other References

Allen, D.E. and Shriever, W.R., (1973), "Progressive Collapse, Abnormal Loads and Building Codes," ASCE National Meeting on Structural Failures: Modes, Causes, Responsibilities, Cleveland, Ohio, April, pp. 21-47.

Anonymous, (1985), “Demolition of Concrete Structures - A Fine Art”, The Civil Engineer in South Africa, V. 27, No. 10, Oct., pp. 523-524.

Alexander, S.J. and Hambly, E.C., (1970), “The Design of Structures to Withstand Gaseous Explosions,” Concrete, Feb. pp. 62-65, Mar. pp.107-116.

U.S. Army, (1990), “Structures to Resist the Effects of Accidental Explosions,” TM 5-1300, Department of the Army, WashingtonDC.

Blandford, G.E., (1997), “Review of progressive Failure Analysis for Truss Structures,” ASCE Journal of Structural Engineering, V. 123, No. 2, Feb. pp. 122-129.

Building Research Division Team, (1970), “Guide Criteria for the Evaluation of Operation Breakthrough Housing System,” PB-212055, 212056, 212058, NTIS, Springfield, VA.

Breen, J., (1976), “Progressive Collapse of Building Structures,” Proceedings of a Research Workshop held at the University of Texas at Austin, 18-20 Nov. 1975.

Breen, J., (1980), “Developing Structural Integrity in Bearing Wall Buildings,” PCI Journal, V. 25, No.1, Jan-Feb. pp. 42-73.

Breen, J.E. and C.P. Siess, (1979), “Progressive Collapse - Symposium Summary.” ACI Journal 76(9):, pp. 997-1004.

Burnett, E.F.P., (1975), “The Avoidance of Progressive Collapse: Regulatory Approaches to the Problem,” NBS-GCR 75-48, National Bureau of Standards, Washington, DC.

Carino, N.J. and Lew, H.S., (2001), “Summary of NIST/GSA Workshop on Application of Seismic Rehabilitation Technologies to Mitigate Blast-Induced Progressive Collapse,” Proceedings of a Workshop held in Oakland, CA, on 10 Sept. 2001 by NIST, Gaithersburg, MD, NISTIR 6831.

Chalk, P. and Corotis, R.B., (1980), “Probability Model for Design Live Loads.” J. Struct. Div. ASCE 106(10), pp. 2017-2033.

Corley, W.G., Mlakar Sr., P.F., Sozen, M.A. and Thornton, C.H., (1998), “The Oklahoma City Bombing: Summary and Recommendations for Multihazardous Mitigation,” ASCE J. of Performance of Constructed Facilities, V. 12, No.3, Aug. pp. 100-112.

CCPS, (1994), Guidelines for Evaluating the Characteristics of Vapor Cloud Explosions, Flash Fires, and BLEVEs, Center for Chemical Process Safety, John Wiley & Sons.

Creasy, L.R., (1972), “Stability of Modern Buildings,” The Structural Engineer, London, V. 50, No. 1, January, pp. 3-6, and discussion, V. 50, No. 7, July, pp. 275-288.

Dusenberry, D. and Juneja, G., (2002), “Review of Existing Guidelines and Provisions Relating to Progressive Collapse,” Multihazard Mitigation Council National Workshop on Prevention of Progressive Collapse, Chicago, Ill. July.

Ellingwood, B., (1981), “Treatment of Accidental Loads and Progressive Failures in Design Standards,” Proceedings of ICOSSAR ’81 , The 3rd International Conference on Structural Safety and Reliability, Trondheim, Norway, June, pp. 649-665.

Ellingwood, B. and E.V. Leyendecker, (1978), “Approaches for design against progressive collapse.” J. Struct. Div. ASCE 104(3), pp. 413-423.

Ellingwood, B. and R. B. Corotis, (1991), “Load combinations for buildings exposed to fires,” Engineering Journal, AISC 28(1), pp. 37-44.

Ellingwood, B., Galambos, T.V., MacGregor, J.G., and Cornell, C.A., (1982), “Probability-based Load Criteria: Load Factors and Load Combinations.” J. Struct. Div. ASCE 108(5), pp. 978-997.

Ellis, B.R.and Currie, D.M., (1998), “Gas Explosions in Buildings in the UK: Regulation and Risk,” The Structural Engineer, V. 76, No. 19, 6 October, pp. 373-380.

Engstrom, B., (1988), “Tests of the Dynamic Behavior of Precast Floors at Sudden Removal of an Exterior Support,” Nordic Concrete Research, No.7, pp. 52-72.

Ferahian, R.H., (1972),”Buildings: Design for Prevention of Progressive Collapse,” Civil Engineering, ASCE, Feb. pp. 66-69.

Fintel, M. and Schultz, D.M., (1976), “A Philosophy for Structural Integrity of Large Panel Buildings,” Proceedings, 8th Joint Panel Conf. of US-Japan Coop. Program in Natural Resources: Wind and Seismic Effects, NBS, Gaithersburg, MD. May 18-21, pp. VI.220-VI.251.

Firnkas, S., (1969), “Concrete Panel Buildings Systems: Progressive Collapse Analyzed,” Review of the Report on the Inquiry into the Collapse at Ronan Point, Canning Town, England, by H. Griffiths, A. Pugsley and O. Saunders, 1968, Civil Engineering, ASCE, Nov. p. 96.

Fuller, G.R., (1975), “Industrialized Concrete Construction for HUD,” ACI-SP 48, Industrialization in ConcreteBuilding Construction, pp. 7-33.

Galambos, T.V., Ellingwood, B., V., MacGregor, J.G., and Cornell, C.A., (1982), “Probability-based Load Criteria: Assessment of Current Design Practice.” J. Struct. Div. ASCE 108(5), pp. 959 - 977.

Granstrom, S., (1970), “Stability of Buildings after Accidental Damage. Forces in Element Joint - Model Tests,” Swedish Building Research Report R20: 1971, summarized in Concrete (U.K.), V. 4, No.3, March, 1970.

Hanson, K. and Olesen, S.O., (1969), “Failure Load and Failure Mechanism of Keyed Shear Joints,” Danish Academy of Engineering, Building Department Report 69/22, Copenhagen, June.

Leggatt, A.J., (1971), “The Economic and Social factors of Design for Accidental Damage,” The Structural Engineer, London, V. 49, No. 7, July, pp. 299-305.

Lewicki, B. and Pauw, A., (1972), "Joints, Precast Panel Buildings," Proceedings of the International Conference on the Planning and Design of TallBuildings, LehighUniversity, Bethlehem, Pa. Aug. Vol. 3, pp. 171-189.

Leyendecker, E.V. and Ellingwood, B.R., (1977), "Design Methods for Reducing the Risk of Progressive Collapse in Buildings," NBSBuilding Science Series 98, Gaithersburg, MD. April.

Moore, D. B., (2002), "The U.K. and European Regulations for Accidental Actions," NIST/NIBS Multihazard Mitigation Council National Workshop on Prevention of Progressive Collapse, Chicago, Ill. July. http://www.nibs.org/MMC/ProgCollapse%20presentations/Moore%20paper.pdf

Morgan, A.E., (1971), "Dams and Other Disasters," Porter Sargent, Boston, p. 119

Mlakar, P.S., (1999), "Structural Design for Physical Security: State of the Practice," ASCE, Reston, Va.

Popoff, A., (1972), "Stability of Precast Concrete System Buildings," Proceedings of the International Conference on the Planning and Design of TallBuildings, LehighUniversity, Bethlehem, Pa. Aug. Vol. 3, pp. 571-583.

Popoff, A., (1975), "Design Against Progressive Collapse," PCI Journal, Mar-Apr. pp.45-57.

Regan, P.E., (1975), "Catenary Action in Damaged Concrete Structures," ACI SP 48-9, American Concrete Institute, Farmington Hills, MI.

Short, A. and Miles, J.R., (1969), "An Introduction to the New Code of Practice for Large Panel Construction," Concrete, London, V. 3, No. 4, Apr. pp. 121-123.

Somes, N.F., (1973), "Abnormal Loadings on Buildings and Progressive Collapse," NBSBuildingScience Series 46, Building Practices for Disaster Mitigation, pp. 427-470.

Taylor, D.A., (1975), "Progressive Collapse," Canadian Journal of Civil Engineering, Vol. 2, No. 4, pp. 517-529.

Wilford, M.J.C. and Yu, C.W., (1973), "Catenary Action in Damaged Structures - Tests on RC Beams and Slabs," DOE/CIRIA Seminar on The Stability of Precast Concrete Structures, London, March.

Williams, G.T. (1990), "Explosive Demolition of Tall Buildings in Inner City Areas," Municipal Engineer (Institution of Civil Engineers), V. 7, No. 4, Aug. pp. 163-173.

Wilson, R. and Crouch, E.A., (1987), "Risk Assessment and Comparisons: An Introduction," Science, 236 pp. 262-270.

Yokel, F.Y., Pielert, J.H. and Schwab, A.R., (1975), "The Implementation of a Provision Against Progressive Collapse," NBSIR 75-715, National Bureau of Standards, Washington, DC.