HANDBOOK FOR THE STRUCTURAL ASSESSMENT OF LARGE PANEL SYSTEM (LPS) DWELLING BLOCKS FOR ACCIDENTAL LOADING

This handbook presents new guidance on the structural assessment and strengthening options for large panel system (LPS) dwelling blocks, focusing primarily on their resistance to accidental loading associated with gas explosions, and supported by extensive background information.

The progressive collapse of part of Ronan Point tower block in east London in 1968 was a significant event in structural engineering in relation to the understanding of disproportionate damage to structures. Extensive research and investigations since then, including full-scale structural load tests on a block in Liverpool, are taken fully into account.

This handbook:

• defines the requirements to be met and the criteria against which the results of a structural assessment of this particular class of building should be judged. These are seen to effectively supersede the previous guidance set down in the Ministry of Housing and Local Government Circulars dating back to 1968
• gives guidance on how to undertake the structural assessments which are required, drawing on previously unpublished technical information
• details the historic background to these requirements, with these being brought up to date and set in the contemporary philosophical context of the requirements of the recently introduced structural Eurocodes
• explains the risk environment which applies to this class of building
• provides an overview of durability assessment/intervention and strengthening options.

RELATED TITLES FROM IHS BRE PRESS

STRUCTURAL APPRAISAL OF EXISTING BUILDINGS, INCLUDING FOR A MATERIAL CHANGE OF USE
DG 366 (4 Parts), 2012

DYNAMIC COMFORT CRITERIA FOR STRUCTURES
A review of UK codes, standards and advisory documents
FB 33, 2011

STRUCTURAL FIRE ENGINEERING
AP 283, 2011

CONCRETE REPAIRS
EP 81 (2-volume set), 2007

CONCRETE STRUCTURES IN FIRE
Performance, design and analysis
BR 490, 2007
HANDBOOK FOR THE STRUCTURAL ASSESSMENT OF LARGE PANEL SYSTEM (LPS) DWELLING BLOCKS FOR ACCIDENTAL LOADING

S L Matthews and B Reeves
The progressive collapse of part of Ronan Point in 1968 has clearly proved to be a very significant event not only in the history of structural engineering but also in terms of some of the central tenets of contemporary structural engineering philosophy, particularly to the ongoing and relevant discussions on matters concerning disproportionate damage and associated topics. However, it is in the nature of things that, with the passage of time, priorities change and past events feature less prominently in people’s consciousness. Those unfamiliar with the Ronan Point incident may be unaware of the origin of some of today’s performance requirements and the associated provisions, which need to be made.

This Handbook seeks to counter this trend by bringing together the relevant background information and putting it into the context of current performance expectations. It does this by providing insight into the evolution of the Building Regulations and the current Requirement A3 of Approved Document A – Structure[1]. This is supported by a review of the available statistical data for the hazard environment within which large panel system (LPS) dwelling blocks exist, linking this information to contemporary risk assessment and management concepts (ie ALARP/SFARP principles) to allow the adoption of rational risk-based strategies in order to facilitate a more unified and consistent approach to the assessment and through-life management of this class of buildings.

The existing guidance used for the structural assessment of LPS dwelling blocks for accidental loading is the Ministry of Housing and Local Government Circulars 62/68[2] and 71/68[3], which were produced shortly after the partial collapse of Ronan Point in 1968. It is necessary to update this guidance, not only to account for subsequent BRE work involving full-scale load testing in three LPS dwelling blocks and the general development of assessment methodologies, but also to make the guidance consistent with the approach to accidental actions employed in the structural Eurocodes and the general philosophical approach employed in contemporary risk management.

This Handbook is intended for engineers involved in the structural assessment of existing multi-storey LPS dwelling blocks, and a structural assessment methodology for this class of building is outlined in section 12. A risk-based methodology for structural assessment of LPS dwelling blocks and through-life management is presented, which has a particular focus upon accidental loads and actions.

The guidance given in this Handbook is necessarily general. It will be necessary for engineers to interpret and apply the information presented, together with the associated recommendations, to the particular circumstances of the LPS dwelling block being appraised on the basis of their engineering experience and judgement. This Handbook contains a large amount of supporting background information, some of which is highly specialised. It may be appropriate to seek specialist advice upon aspects of undertaking risk assessments and/or hazard identification.

In summary, this Handbook:

- Defines a set of requirements for LPS dwelling blocks exceeding four storeys in height (ie five storeys and higher), which are to be used to ascertain whether such LPS dwelling blocks are considered to satisfy Requirement A3 of Approved Document A – Structure[1].
- Presents a historical review of key events that have occurred since the partial collapse of Ronan Point in 1968 and details the associated evolution of requirements relating to disproportionate damage and associated matters as set down in Approved Document A - Structure, including the introduction of the structural Eurocodes.
- Reviews the hazards to which LPS dwelling blocks are exposed, and evaluates the foreseeable risks associated with accidental loadings and actions to which such blocks may be exposed. Justification is given for the 17 kN/m² accidental overpressure loading criterion used in the structural assessment of LPS dwelling blocks without a piped gas supply and without a basement. The risks associated with vehicular impacts and accidental external gas explosions are also considered.
- Describes recent structural tests on three LPS dwelling blocks undertaken by BRE, together with a range of earlier structural tests performed by others, which give insight into the potential behaviour of LPS dwelling blocks under accidental loadings and actions. The BRE tests, which were preceded by forensic investigations of the blocks concerned, included a variety of structural tests simulating overpressure loading (simultaneous loading of walls and floors) taken to failure to assess the strength and performance of the structural members forming the walls and floors of the test rooms. There was an associated test involving breaking the load path to the foundations.
of a loadbearing kitchen–lounge spine wall in a Bison Wallframe LPS dwelling block. This involved removal of the top section of one of these walls, with 14 storeys remaining above the test location, demonstrating that alternative load paths were mobilised. This work was complemented by finite element and other forms of structural analysis.

• Sets out the basic principles and steps of a methodology for the assessment of large panel buildings subject to accidental loading and actions, drawing upon a range of structural assessment techniques involving increasing degrees of sophistication/complexity.

• Describes options for undertaking strengthening and other preventive or remedial works on LPS dwelling blocks, should the structural assessment procedure be unable to justify the adequacy of the LPS dwelling block concerned for the accidental loads and actions considered.

• Contains a number of case studies, which illustrate some structural assessments that have been undertaken on LPS dwelling blocks by BRE and other consultants.

The approach adopted by BRE utilises the concept that all LPS dwelling blocks should be managed using a systematic risk assessment methodology to guide through-life management activities and associated structure related actions, with the goal of:

• eliminating hazards where practicable, and

• reducing hazards and controlling risks to the structure of these buildings as far as is practicable,

consistent with ALARP/SFARP principles, taking account that risks associated with structural damage and collapse are generally at very low levels/acceptable levels.

In this context through-life management activities would be expected to form part of an overall asset management system operated by the owning body or authority.

This Handbook has been written primarily from the perspective of The Building Regulations 2010 for England and Wales. However, it should be recognised that Scotland and Northern Ireland are governed by separate legislation. Whilst the general objectives and requirements are similar, there are subtle differences. In spite of this, the principles involved will be generally applicable.

This Handbook comprises 13 main sections and associated supporting information, glossary, etc.

Section 1 provides an introduction and sets out the scope of this Handbook.

Section 2 describes the requirements which are to be used as a basis for the structural assessment of LPS dwelling blocks exceeding four storeys in height (ie five storeys and higher) for accidental loading.

Section 3 summarises the evolution of the Building Regulations and LPS dwelling block structural assessment requirements associated with issues of progressive collapse and related matters.

Section 4 describes the wider context of through-life performance and assessment issues relating to LPS dwelling blocks, setting down the underlying premise of the structural assessment methodology developed by BRE for LPS dwelling blocks.

Section 5 considers the nature of the hazards which experience indicates apply to LPS dwelling blocks in the UK. The principal accidental loads and actions are internal and external gas explosions, impacts by various forms of vehicle and fire. The section also provides an overview of some of the available statistics on explosions and associated hazards.

Section 6 examines risk issues and the use of risk assessment procedures in the through-life decision-making process for LPS dwelling blocks, in particular the use of the ALARP and SFARP principles. The section provides a brief overview of risk issues applicable to LPS dwelling blocks.

Section 7 explains the origin of current national assessment overpressure criteria associated with internal gas explosions; namely the 17 kN/m² criterion for LPS dwelling blocks without a piped gas supply and the 34 kN/m² criterion for LPS dwelling blocks with a piped gas supply.

Section 8 provides an overview of the structural performance of three LPS dwellings blocks as evaluated by recent full-scale load tests carried out by BRE. These tests included a number of room overpressure tests up to the ultimate load condition which applied static loadings seeking to create forces comparable to those produced by accidental internal gas explosions involving cylinder gas or other gaseous substances. An element removal test involving breaking the load path in a wall panel with 14 storeys of building above was undertaken to establish if an alternative load path would be mobilised.

Section 9 discusses a number of factors which might potentially influence the behaviour of an LPS dwelling block under accidental loads or actions and which it might be appropriate to take account of during a structural assessment.

Section 10 gives consideration to a number of other factors concerned with the durability of concrete components forming an LPS dwelling block, undertaking a structural assessment after a severe fire and the risk of progressive collapse during demolition at the end of the useful life of the block.

Section 11 provides a summary of the requirements, hazards, risks and through-life performance issues affecting an LPS dwelling block.

---

1 In this document reference is made to managing risks on the basis of ALARP principles, that is seeking to ensure that risks are ‘as low as reasonably practicable’. The term ALARP is used widely in the technical literature and associated guidance. However, an alternative term that is commonly used in legislation is SFARP: ‘so far as is reasonably practicable’. In this document where reference is made to the term ALARP this should also be taken to include the alternative SFARP.

2 Asset management is a process of providing planned through-life care for constructed assets; it involves management and planning procedures together with the associated maintenance, preventive and/or remedial works activities. Asset management activities/systems involve the planning and financial aspects required to ensure that the appropriate resources are available for the associated maintenance, preventive and/or remedial works activities that may be necessary. In this document terms such as ‘through-life care’, ‘planned through-life care’ and ‘through-life management activities’ imply linkage to an overarching asset management scheme.
Section 12 sets down a methodology for assessing LPS dwelling blocks for accidental loads and actions.

Section 13 outlines potential strengthening options for LPS dwelling blocks and describes a methodology for the development of a strengthening/through-life management strategy for an LPS dwelling block.

Section 14 presents some concluding comments. Attention is drawn to the potential vulnerability of LPS dwelling blocks to progressive collapse. This could occur should a sufficiently large explosion or devastating incident occur, creating forces which exceed the collapse resistance of the LPS dwelling block. The recent progressive collapse of parts of a number of LPS dwelling blocks during demolition highlights this risk, particularly for poorly constructed blocks with deficient tying.

The above mentioned material is supported by an extensive reference list and bibliography.

There are also 11 Appendices to this Handbook. These present an overview of a number of key subject areas including the historical background to LPS dwelling blocks within the UK and associated guidance, together with a summary of previously unpublished information on load testing undertaken on LPS dwelling blocks or assemblies of components, spanning the last three and a half decades. The focus is upon full-scale load tests undertaken on LPS dwelling blocks by BRE over about the last decade.

Summary information is also included on various strengthening techniques that have previously been employed to enhance the strength and/or robustness of LPS dwelling blocks.

Case histories of several consultancy commissions have been included to demonstrate the various approaches that have been adopted by BRE and by other consultants when assessing various types of LPS buildings. A number of these case histories also include references to the remedial and/or strengthening works techniques that were adopted.

The 11 Appendices are titled:

Appendix A: Development of ‘regulatory’ requirements
Appendix B: Hazard environment
Appendix C: Risk issues

Appendix D: Outline historical review of LPS dwelling blocks in the UK since 1968
Appendix E: BRE tests on LPS dwelling blocks and laboratory structures undertaken before 2000
Appendix F: Overview of finite element analyses and calibration exercises for the 1990s’ BRE load tests on LPS dwelling blocks
Appendix G: BRE load testing of a Bison Wallframe LPS dwelling block, Liverpool
Appendix H: Overview of finite element analyses and calibration exercises for the Liverpool Bison Wallframe LPS dwelling block tested by BRE
Appendix I: Strengthening options
Appendix J: LPS dwelling block assessment case studies
Appendix K: BRE trials to determine coefficient of friction at base of wall panels

Stuart Matthews
Barry Reeves

November 2011

REFERENCES
ACKNOWLEDGEMENTS

THIS PROJECT
This document is an outcome of a project sponsored by the Department of Communities and Local Government (DCLG) for the development of an informative Handbook explaining how to undertake the tasks necessary to address the technical requirements for the structural assessment of large panel system (LPS) dwelling blocks. This document is that informative Handbook. The support provided by the DCLG for this work is gratefully acknowledged.

The authors wish to acknowledge the essential contributions made by current and former BRE colleagues, especially Julie Bregulla, David Brooke, Gerard Canisius, Brian Ellis, Jesper Friis, Haig Gulvanessian, Robert Mobbs, Rohan Rupasinghe and Susanne Woodman, in respect of the current project and the previous Partners in Innovation and DETR research projects.

The authors also wish to recognise the valuable contributions made to the project in terms of information provided, experiences shared and review of project deliverables by the previous project partners and steering group. The authors are particularly grateful to have been able to draw freely upon these contributions and acknowledge the valuable assistance that this material has given in the preparation of this document.

The authors are also particularly indebted to a number of colleagues who undertook the task of making independent reviews of this output, namely Professor Michael Baker (Emeritus Professor of Safety Engineering, School of Engineering, University of Aberdeen), Mr John Carpenter (Secretary to SC OSS), Mr Nick Clarke (IHS BRE Press), Mrs Sarah Fray (Director of Engineering at ISE), Mr Alistair Soane (Director CROSS) and Mr Peter Watt (Project Officer at DCLG).

PREVIOUS BRE RESEARCH PROJECTS
This report has been developed from the final output of a Department for Trade and Industry (DTI) part-funded Partners in Innovation research project aimed at improving the management of ageing assets by the use of advanced techniques to undertake structural assessment of existing multi-storey LPS dwelling blocks for accidental loading due to non-piped gas explosions.

This document draws on work undertaken in several research projects. The most recent work involved full-scale structural load tests up to failure within a Bison LPS dwelling block situated in Liverpool, which was carried out under the (previous) Partners in Innovation (PII) scheme via a project entitled ‘Improving the management of ageing assets by advanced techniques for assessing existing multi-storey LPS blocks’. Reference is also made to the results of an earlier programme of full-scale structural loading tests upon LPS dwelling blocks which were situated in Sandwell (a Bison LPS dwelling block) and Leeds (a Reema Conclad LPS dwelling block).

The authors gratefully acknowledge the support provided by the DTI and the project partners who are listed below, and that given by the then Department of the Environment, Transport and the Regions (DETR) for the earlier experimental work.

PREVIOUS LPS PII PROJECT PARTNERS
Consulting engineers
• Alan Conisbee and Associates
• Campbell Reith Hill
• Carter Clack
• Curtins Consulting
• Ian Bennett Associates
• Mitchell McFarlane and Partners

Local authorities and housing associations
• Association for Structural Engineers of London Boroughs (ASELB) via London Borough of Bromley
• Birmingham City Council
• CityWest Homes (London Borough of Westminster)
• Flintshire County Council
• Liverpool Housing Action Trust (who kindly provided an LPS dwelling block for testing)
• London Borough of Barking & Dagenham
• London Borough of Enfield
• London Borough of Greenwich
• London Borough of Hackney
• London Borough of Lambeth
• London Borough of Sutton
• Northern Ireland Housing Executive
• Sandwell Metropolitan Borough Council
• Stockport Borough Council
• Sunderland Housing Group

Industrial companies
• Birmingham City Laboratories
• Germann Instruments
• Wilde and Partners/TNO (Netherlands Organisation for Applied Scientific Research)
PREVIOUS LPS PII PROJECT FUNDING PARTNERS
- Department of Trade and Industry
- Flintshire County Council
- Northern Ireland Housing Executive

EARLIER DETR LPS RESEARCH PROJECT
COLLABORATION PARTNERS
- Leeds City Council (who kindly provided an LPS dwelling block for testing)
- Sandwell Metropolitan Borough Council (who kindly provided an LPS dwelling block for testing)
# CONTENTS

<table>
<thead>
<tr>
<th>Section</th>
<th>Pages</th>
</tr>
</thead>
<tbody>
<tr>
<td>PREFACE</td>
<td>iii</td>
</tr>
<tr>
<td>ACKNOWLEDGEMENTS</td>
<td>vii</td>
</tr>
<tr>
<td>LIST OF FIGURES AND PLATES</td>
<td>xv</td>
</tr>
<tr>
<td>LIST OF TABLES</td>
<td>xix</td>
</tr>
<tr>
<td>KEYWORDS AND ABBREVIATIONS</td>
<td>xxii</td>
</tr>
<tr>
<td>GLOSSARY AND DEFINITIONS</td>
<td>xxii</td>
</tr>
<tr>
<td>EXECUTIVE SUMMARY</td>
<td>xxix</td>
</tr>
<tr>
<td>1. INTRODUCTION AND SCOPE OF THE HANDBOOK</td>
<td>1</td>
</tr>
<tr>
<td>1.1 Background</td>
<td>1</td>
</tr>
<tr>
<td>1.2 Aims and objectives of the current project</td>
<td>3</td>
</tr>
<tr>
<td>1.3 Related BRE research studies</td>
<td>4</td>
</tr>
<tr>
<td>1.4 Previous BRE work on LPS dwelling blocks</td>
<td>4</td>
</tr>
<tr>
<td>2. REQUIREMENTS FOR STRUCTURAL ASSESSMENT OF LPS DWELLING BLOCKS FOR ACCIDENTAL LOADING</td>
<td>5</td>
</tr>
<tr>
<td>2.1 Alternative approach</td>
<td>6</td>
</tr>
<tr>
<td>3. BUILDING REGULATION AND LPS ASSESSMENT REQUIREMENTS, PROGRESSIVE COLLAPSE AND RELATED MATTERS</td>
<td>7</td>
</tr>
<tr>
<td>3.1 Introduction</td>
<td>7</td>
</tr>
<tr>
<td>3.2 The starting point – The partial collapse of Ronan Point</td>
<td>7</td>
</tr>
<tr>
<td>3.3 Ministry of Housing and Local Government Circulars 62/68 and 71/68</td>
<td>8</td>
</tr>
<tr>
<td>3.4 Changes to the Building Regulations and LPS assessment requirements with time</td>
<td>9</td>
</tr>
<tr>
<td>3.5 Other guidance on assessment of LPS dwelling blocks</td>
<td>9</td>
</tr>
<tr>
<td>3.6 Progressive collapse, disproportionate structural damage and related issues</td>
<td>10</td>
</tr>
<tr>
<td>3.6.1 Introduction</td>
<td>10</td>
</tr>
<tr>
<td>3.6.2 Progressive collapse</td>
<td>10</td>
</tr>
<tr>
<td>3.6.3 Robustness</td>
<td>14</td>
</tr>
<tr>
<td>3.6.4 Collapse resistance</td>
<td>15</td>
</tr>
<tr>
<td>3.7 Contemporary Building Regulations and related guidance</td>
<td>15</td>
</tr>
<tr>
<td>4. THROUGH-LIFE PERFORMANCE AND ASSESSMENT ISSUES</td>
<td>17</td>
</tr>
<tr>
<td>4.1 Introduction</td>
<td>17</td>
</tr>
<tr>
<td>4.2 Through-life performance of buildings</td>
<td>17</td>
</tr>
<tr>
<td>4.3 Approaches to the through-life management of LPS dwelling blocks</td>
<td>19</td>
</tr>
<tr>
<td>4.4 Underlying premise of assessment methodology developed by BRE</td>
<td>19</td>
</tr>
<tr>
<td>5. HAZARD IDENTIFICATION</td>
<td>21</td>
</tr>
<tr>
<td>5.1 Introduction</td>
<td>21</td>
</tr>
<tr>
<td>5.2 Sources of hazards</td>
<td>21</td>
</tr>
<tr>
<td>5.3 Accidental loads and actions on LPS dwelling blocks</td>
<td>21</td>
</tr>
</tbody>
</table>

Cont'd...
5 HAZARD IDENTIFICATION (Cont’d)
5.4 Accidental internal gas explosions 22
  5.4.1 Classification of explosion incidents 22
  5.4.2 Single room explosions 23
  5.4.3 Multi-room explosions 23
5.5 Accidental external gas explosions 23
5.6 Accidental vehicle impacts 23
  5.6.1 Road vehicle impacts 23
  5.6.2 Aircraft impacts 24
  5.6.3 Train impacts 24
  5.6.4 Vehicle impacts – Summary 24
5.7 Fire 24
5.8 Statistics on explosions and associated risks 25

6 RISK ISSUES 27
6.1 Introduction 27
6.2 The use of risk assessments in the decision-making process 27
6.3 The ALARP and SFARP principles 27
6.4 Overview of risk issues applicable to LPS dwelling blocks 28

7 ORIGIN OF CURRENT NATIONAL ASSESSMENT OVERPRESSURE CRITERIA 31
7.1 Overpressure loading associated with internal piped gas explosions 31
7.2 Overpressure loading associated with internal non-piped gas explosions 32
7.3 Overpressure loadings associated with internal explosions involving petrol or other chemicals 33

8 STRUCTURAL PERFORMANCE OF LPS DWELLINGS BLOCKS EVALUATED BY RECENT FULL-SCALE LOAD TESTS CARRIED OUT BY BRE 34
8.1 Overview of the structural performance of LPS dwellings blocks during BRE full-scale tests 34
8.2 Summary – The structural performance of LPS dwellings blocks during full-scale tests relative to the overpressure loadings created by accidental internal gas explosions involving cylinder gas or other gaseous substances 35

9 LPS DWELLING BLOCK ASSESSMENT – SOME FACTORS INFLUENCING BEHAVIOUR UNDER ACCIDENTAL LOADS AND ACTIONS 37
9.1 Introduction 37
9.2 LPS dwelling block structural parameters 37
  9.2.1 Building height and form 37
  9.2.2 Direction of span of floor slabs 38
  9.2.3 Concrete strength parameters obtained by sampling existing LPS dwelling blocks 38
  9.2.4 Joints at the base of loadbearing wall panels: joint friction/adhesion parameters 41
  9.2.5 Ability to mobilise alternative load paths 42
9.3 Factors influencing behaviour during an internal gas explosion 45
  9.3.1 Introduction 45
  9.3.2 Venting mechanisms 45
  9.3.3 Magnitude of overpressure used in structural assessments 45
9.4 Issues associated with piped gas supplies 46
  9.4.1 Introduction 46
  9.4.2 Ability to mobilise alternative load paths when piped gas is present 46
  9.4.3 Safety issues relating to piped gas supplies and associated appliances 46

10 OTHER CONSIDERATIONS 48
10.1 Introduction 48
10.2 Strategy for the through-life durability management of an LPS dwelling block 48
10.3 Durability of the concrete components forming an LPS dwelling block 48
  10.3.1 Introduction 48
  10.3.2 Passivation of steel in concrete 50
  10.3.3 Depassivation of steel in concrete by carbonation 50
  10.3.4 Depassivation of steel in concrete by chlorides 51
  10.3.5 Durability investigation and assessment procedure 53
  10.3.6 Selection of an intervention method for concrete protection or repair 56
## 10 OTHER CONSIDERATIONS (Cont’d)

10.4 Structural assessment after a severe fire

10.4.1 Introduction

10.4.2 Effects of heating and subsequent cooling

10.4.3 Investigation and structural assessment procedure

10.4.4 Procedures for structural repair after a severe fire

10.5 Demolition

## 11 SUMMARY – REQUIREMENTS, HAZARDS, RISKS AND PERFORMANCE

11.1 Assessment criteria

11.2 Risks

11.3 Collapse resistance

11.4 Future management

11.5 Durability assessment

11.6 Proposed assessment/inspection regime

11.7 Demolition

## 12 METHODOLOGY FOR THROUGH-LIFE MANAGEMENT AND ASSESSMENT OF LPS DWELLING BLOCKS FOR ACCIDENTAL LOADS AND ACTIONS

12.1 Introduction

12.2 Background

12.3 Overall through-life management methodology

12.4 Overview of structural assessment approach for LPS dwelling blocks

12.5 Assessment Stage 1 – Review of existing technical information

12.5.1 General

12.5.2 Presence of a piped gas supply to any part of the building

12.5.3 Material properties

12.6 Assessment Stage 2 – Collection of new technical information

12.7 Assessment Stage 3 – Assessment of block under normal loading

12.8 Assessment Stage 4 – Assessment of block under accidental loading

12.8.1 Introduction

12.8.2 Assessment Level 1 – Deterministic linear elastic analysis (eg spreadsheet based)

12.8.3 Assessment Level 2 – Deterministic non-linear finite element analysis

12.8.4 Assessment Level 3 – Probabilistic-based structural assessment methodology

12.9 Suggested supporting regime involving periodic inspection, monitoring and structural assessment

12.10 Risk reduction and risk management measures

12.10.1 Introduction

12.10.2 LPS dwelling block without a piped gas supply

12.10.3 LPS dwelling block with a piped gas supply

## 13 STRUCTURAL STRENGTHENING OPTIONS FOR LPS DWELLING BLOCKS

13.1 Background

13.2 Development of strengthening/through-life risk management strategy

## 14 CONCLUDING REMARKS

14.1 Historical perspective on LPS dwelling block structural assessment

14.2 Contemporary guidance on LPS dwelling block structural assessment

14.3 Some points from an owner’s perspective

14.4 Some issues from a consulting engineer’s perspective

14.5 Finally – A note of caution!

## 15 REFERENCES

## 16 BIBLIOGRAPHY AND FURTHER READING
APPENDICES

APPENDIX A: DEVELOPMENT OF ‘REGULATORY’ REQUIREMENTS

A1 Introduction
A2 Development of the requirements for the structural assessment of existing LPS dwelling blocks
A3 Evolution of the requirements for the design of new buildings to reduce their sensitivity to disproportionate damage in the event of an accident
A4 Workplace (Health, Safety and Welfare) Regulations 1992
A5 Construction (Design and Management) Regulations
A6 Other legislation
A7 Overview of the technical requirements for the structural assessment of existing LPS dwelling blocks

APPENDIX B: HAZARD ENVIRONMENT

B1 Introduction
B2 Accidental internal gas explosion statistics
B3 Accidental external gas explosions
B4 Accidental road vehicle impact statistics
B5 Accidental impact by other types of vehicle
B6 Fire
B7 Probability of occurrence and risk levels associated with internal gas explosions in a building without a piped gas supply
B8 Benchmarking – Comparison of risks from some other hazards

APPENDIX C: RISK ISSUES

C1 Introduction
C2 Risk and risk management
C3 Qualitative risk assessment
C4 Quantitative risk assessment approaches
  C4.1 Approaches for defining a socially acceptable level of risk: F–N curves
  C4.2 Approaches for defining a socially acceptable level of risk: Cost benefit analysis
  C4.3 Cost of preventing a fatality (CPF) in an LPS dwelling block
  C4.4 Judging the proportionality of measures to reduce risk: Illustration of the application of the ‘value of preventing a fatality’ (VPF)
  C4.5 Judging the proportionality of measures to reduce risk: Application of ‘value of preventing a fatality’ (VPF) to an LPS dwelling block

APPENDIX D: OUTLINE HISTORICAL REVIEW OF LPS DWELLING BLOCKS IN THE UK SINCE 1968

D1 Introduction
D2 LPS dwelling block types
D3 Structural assessment of LPS dwelling blocks for accidental loading
D4 Historical load testing of LPS structures/components (Concrete Ltd)
D5 Strength testing of cross- and flank-wall joints in Reema blocks
  D5.1 Cross wall/floor slab joints
  D5.2 Flank wall/floor slab joints
D6 Strength testing of other LPS dwelling block systems

APPENDIX E: BRE TESTS ON LPS DWELLING BLOCKS AND LABORATORY STRUCTURES UNDERNEATH BEFORE 2000

E1 Introduction
E2 Testing of reinforced concrete panels under static and dynamic loads
E3 Explosion testing in existing LPS dwelling blocks
E4 Full-scale load testing to failure of existing LPS dwelling blocks
E5 Influence of low concrete compressive strength
APPENDIX F: OVERVIEW OF FINITE ELEMENT ANALYSES AND CALIBRATION EXERCISES FOR THE 1990s' BRE LOAD TESTS ON LPS DWELLING BLOCKS

F1 Introduction 137
F2 Methodology for calculating ‘equivalent’ uniformly distributed load 137
F3 Method of analysis: Combined wall and floor overpressure tests
   F3.1 Floor slab analyses 138
   F3.2 Wall panel analyses 138
F4 Method of analysis: Floor overpressure tests 138
F5 Reema Conclad LPS dwelling block, Leeds 138
   F5.1 Floor slab analyses: 4th floor lounge: Floor overpressure test 138
   F5.2 Floor slab analyses: 8th floor bedroom: Combined wall and floor overpressure test 142
   F5.3 Wall panel analyses: 8th floor bedroom: Combined wall and floor overpressure test 144
F6 Bison Wallframe LPS dwelling block, Sandwell 145
   F6.1 Floor slab analyses: 19th floor lounge: Combined wall and floor overpressure test 145
   F6.2 Wall panel analyses: 19th floor lounge: Combined wall and floor overpressure test 150
   F6.3 Wall panel analyses: 19th floor mid-kitchen: Combined wall and floor overpressure test 152

APPENDIX G: BRE LOAD TESTING OF A BISON WALLFRAME LPS DWELLING BLOCK, LIVERPOOL

G1 Introduction – Background and aims of the load testing 155
G2 Scope of overpressure load testing and element removal programme 155
G3 Investigation to assess quality of construction of the LPS dwelling block 156
G4 Overpressure tests: Combined wall and floor overpressure tests 157
G5 Ability to mobilise alternative load paths: Spine wall test 160

APPENDIX H: OVERVIEW OF FINITE ELEMENT ANALYSES AND CALIBRATION EXERCISES FOR THE LIVERPOOL BISON WALLFRAME LPS DWELLING BLOCK TESTED BY BRE

H1 Introduction 164
H2 Modelling of LPS dwelling block response to the removal of a kitchen–lounge spine wall element 164
H3 Modelling the behaviour of the flank wall flat lounge floor slabs under overpressure and other forms of loading 165

APPENDIX I: STRENGTHENING OPTIONS

I1 Introduction 169
I2 Overview of some strengthening options 169
I3 Vertical extent of strengthening required 169
I4 Component strengthening
   I4.1 Principle 172
   I4.2 Options 172
I5 Dry packing/in-situ joint enhancement
   I5.1 Principle 173
   I5.2 Options 173
I6 Supplementary tying
   I6.1 Principle 173
   I6.2 Options 174
I7 Supplementary load bearing structural elements
   I7.1 Principle 174
   I7.2 Options 174
I8 Strengthening of joints between components
   I8.1 Principle 174
   I8.2 Options 174

APPENDIX J: LPS DWELLING BLOCK ASSESSMENT CASE STUDIES

J1 Introduction 175
J2 Case study A: Pre-Ronan Point Bison Wallframe block: Midlands
   J2.1 Introduction 175
   J2.2 Floor panels 175
   J2.3 Wall panels 175
   J2.4 Probabilistic risk assessment 176

Cont’d . . .
APPENDIX J: LPS DWELLING BLOCK ASSESSMENT CASE STUDIES (Cont’d)

J3 Case study B: Pre-Ronan Point Jesperson block: London Borough
   J3.1 Introduction
   J3.2 Structural assessment
   J3.3 Results of structural assessment

J4 Case study C: Reema Scissor Type block: South Coast
   J4.1 Introduction
   J4.2 Assessment
   J4.3 Findings

J5 Case study D: Pre and Post Ronan Point Fram Russell blocks: North-West England
   J5.1 Introduction
   J5.2 Fram Russell Type A blocks
   J5.3 Fram Russell Type B blocks
   J5.4 Alternative strategies for the future management of risk

J6 Case study E: Pre-Ronan Point Bison Wallframe block: North-East England
   J6.1 Introduction
   J6.2 Review of existing documentation
   J6.3 Assessment
   J6.4 Results of assessment
   J6.5 Alternative remedial options
   J6.6 Durability assessment
   J6.7 Outcome

APPENDIX K: BRE TRIALS TO DETERMINE THE COEFFICIENT OF FRICTION AT THE BASE OF WALL PANELS

K1 Introduction
K2 Specimens
   K2.1 Concrete specimen
   K2.2 Dry-pack specimens
K3 Test arrangement
K4 Summary test results
K5 Friction values given in codes of practice for design

INDEX
LIST OF FIGURES AND PLATES

FIGURES
Figure 1: Bison LPS dwelling block
Figure 2: Reema Conclad LPS dwelling block
Figure 3: General configuration of the wall and floor panel construction forming an LPS block
Figure 4: Schematic diagram of the various joints between flank wall and floor panels in a typical LPS dwelling block
Figure 5: Damage to Ronan Point (1968)
Figure 6: Schematic showing potential low-level and high-level trigger sites adjacent to the flank wall in a typical LPS dwelling block
Figure 7: Schematic showing some possible progressive collapse failure modes of the flank wall in a typical LPS dwelling block for a low-level trigger site
Figure 8: Schematic showing a possible progressive collapse failure mode of the flank wall involving wall–wall and floor-wall joints for a low- to medium-level trigger site
Figure 9: Schematic showing a possible progressive collapse failure mode of the flank wall and floor slab involving wall–wall and floor–wall joints (and potentially floor–floor) for a low- to medium-level trigger site
Figure 10: Schematic showing some possible progressive collapse failure modes in a typical LPS dwelling block for a high-level trigger site
Figure 11: Schematic illustrating the concepts of probabilistic service life
Figure 12: Schematic illustrating notional changes in failure rate through service life: the so called ‘bath-tub’ performance curve
Figure 13: Some potential sources of hazard to an LPS dwelling block
Figure 14: The tolerability of risk (TOR) framework
Figure 15: Concrete strength parameters – LPS dwelling block floor slabs
Figure 16: Concrete strength parameters – LPS dwelling block wall panels
Figure 17: Estimated characteristic concrete compressive strength versus mean concrete compressive strength – all LPS dwelling block data
Figure 18: Concrete strength parameters – range of values for floors slabs and wall panels
Figure 19: Illustrative section through joint between base of wall panel and floor slab – Flank wall detail for pre-Ronan Point Bison Wallframe LPS block
Figure 20: Illustrative section through joint between base of wall panel and floor slab – Cross wall detail for pre-Ronan Point Bison Wallframe LPS block
Figure 21: Illustrative section through joint between base of wall panel and floor slab – Flank wall detail for post-Ronan Point Bison Wallframe LPS block
Figure 22: Illustrative section through joint between base of wall panel and floor slab – Cross wall detail for post-Ronan Point Bison Wallframe LPS block
Figure 23: Reinforcement corrosion in an LPS dwelling block floor due to cast-in chloride
Figure 24: Localised pitting corrosion of reinforcing bar due to ingressed chloride
Figure 25: Diagrammatic view of steel protected from (carbonation induced) corrosion in uncarbonated concrete
Figure 26: Diagrammatic view of steel corroding in carbonated concrete
Figure 27: Carbonation depth versus time and implications of different concrete cover depths upon the notional service life of a concrete structure
Figure 28: Comparison of estimated depth of cover values with measured depth of carbonation values and prognoses of future behaviour as the depth of carbonation increases with time
Figure 29: Estimated risk of steel reinforcement corrosion associated with carbonation, cast-in chloride content and environmental conditions
Figure 30: Analysis of chloride ion content of LPS dwelling block concrete components
Figure 31: Overview of main process steps for inspection, assessment, management and making an intervention upon a concrete structure to extend its useful service life
Figure 32: Interpretation of risk of corrosion of steel reinforcement and possible response actions (after BRE Digest 444: Part 2)
Appendices

Figure A1: Diagrammatic representation of historical changes in LPS structural assessment requirements versus building height, with contemporary Building Regulation requirements (Approved Document A) for multiple occupancy buildings also shown.

Figure A2: Area at risk of collapse in the event of an accident.

Figure A3: Strategies for accidental design situations (after BS EN 1991-1-7:2006).

Figure B1: A moderate piped gas explosion in a modern cavity wall construction house.

Figure B2: A severe piped gas explosion in a reinforced concrete framed building.

Figure B3: A severe piped gas explosion in a block of flats of brick masonry construction.

Figure B4: A moderate cylinder gas explosion in a modern house.

Figure B5: A severe cylinder gas explosion, followed by fire, in a modern house.

Figure B6: A very severe piped gas explosion – damage to six-storey LPS dwelling block at Hulme, Manchester.

Figure B7: A very severe piped gas explosion – damage to six-storey LPS dwelling block at Hulme, Manchester (Viewed from opposite side of LPS dwelling block to Figure B6).

Figure B8: Aerial view showing damage to the Dutch apartment block after the impact by a Boeing 747-200 cargo plane.

Figure B9: Damage and fire that developed in the 10-storey tower block following impact by aircraft.

Figure B10: View of damage to Pirelli Building, Milan, following light aircraft impact.

Figure B11: Aerial view of train carriages following derailment and collision with apartment building.

Figure C1: Schematic illustrating the components of the risk management process.

Figure C2: Overview of risk analysis (after BS EN 1991-1-7:2006; Figure B1).

Figure C3: \( F-N \) curves, where \( F(n) = P(N_d > n) \) in one year, and the ALARP region: collapse of a single stack of dwellings.

Figure C4: \( F-N \) curves showing the \( F(n) = P(N_d > n) \) in one year: requirement for one year: collapse of a single stack of dwellings.

Figure C5: \( F-N \) curves showing the \( F(n) = P(N_d > n) \) in one year: collapse of an entire LPS dwelling block.

Figure C6: Individual risk and the cost of preventing a fatality for various transport safety measures (reproduction of Figure 9 from reference[30]).

Figure D1: Approximate number of blocks by system type: four storeys or less.

Figure D2: Approximate number of blocks by system type: five storeys or more.

Figure E1: Upward deflection of ceiling slab in Leeds LPS 4th floor lounge floor load test, Sensor Group 1.

Figure E2: Upward deflection of ceiling slab in Leeds LPS 4th floor lounge floor load test, Sensor Group 2.

Figure E3: Downward deflection of floor slab in Leeds LPS 4th floor lounge floor load test, Sensor Group 1.

Figure E4: Downward deflection of floor slab in Leeds LPS 4th floor lounge floor load test, Sensor Group 2.

Figure E5: Plan of ceiling slab showing location of upward deflection monitoring transducers (Floor T+1): Leeds LPS 4th floor lounge floor load test.

Figure E6: Plan of floor slab showing location of downward deflection monitoring transducers (Floor T): Leeds LPS 4th floor lounge floor load test.

Figure E7: Upward deflection of ceiling slab in Leeds LPS 8th floor bedroom: Room load test, Sensor Group 1.

Figure E8: Upward deflection of ceiling slab in Leeds LPS 8th floor bedroom: Room load test, Sensor Group 2.

Figure E9: Downward deflection of floor slab in Leeds LPS 8th floor bedroom: Room load test, Sensor Group 1.

Figure E10: Downward deflection of floor slab in Leeds LPS 8th floor bedroom: Room load test, Sensor Group 2.

Figure E11: Plan of ceiling slab showing location of upward deflection monitoring transducers (Floor T+1): Leeds LPS 8th floor bedroom: Room load test.

Figure E12: Plan of floor slab showing location of downward deflection monitoring transducers (Floor T): Leeds LPS 8th floor bedroom: Room load test.

Figure E13: Lateral deflection of flank wall in Leeds LPS 8th floor bedroom: Room load test.

Figure E14: Lateral deflection of cross wall in Leeds LPS 8th floor bedroom: Room load test.

Figure E15: Elevation of flank wall showing location of displacement monitoring transducers: Leeds LPS 8th floor bedroom: Room load test.

Figure E16: Elevation of cross wall showing location of displacement monitoring transducers: Leeds LPS 8th floor bedroom: Room load test.

Figure E17: Upward deflection of ceiling slab adjacent kitchen (Floor T+1) in Sandwell Bison LPS 19th floor lounge: Room load test.

Figure E18: Upward deflection of ceiling slab adjacent elevation wall/window panel (Floor T+1).
in Sandwell Bison LPS 19th floor lounge: Room load test

Figure F19: Downward deflection of floor slab adjacent kitchen (Floor T) in Sandwell Bison LPS 19th floor lounge: Room load test

Figure F20: Downward deflection of floor slab adjacent elevation wall/window panel (Floor T) in Sandwell Bison LPS 19th floor lounge: Room load test

Figure F21: Downward mid-span deflection of adjacent floor slabs (Floor T) in Sandwell Bison LPS 19th floor lounge: Room load test

Figure F22: Plan of floor and ceiling slabs showing location of downward and upward deflection monitoring transducers (Floors T and T+1) in Sandwell Bison LPS 19th floor lounge: Room load test

Figure F23: Lateral deflection of flank wall adjacent elevation wall/window panel in Sandwell Bison LPS 19th floor lounge: Room load test

Figure F24: Lateral deflection of flank wall adjacent kitchen in Sandwell Bison LPS 19th floor lounge: Room load test

Figure F25: Lateral deflection of cross wall adjacent kitchen in Sandwell Bison LPS 19th floor lounge: Room load test

Figure F26: Elevations of flank and cross walls showing location of displacement monitoring transducers: Sandwell Bison LPS 19th floor lounge: Room load test

Figure F27: Lateral deflection of bedroom/kitchen cross wall in Sandwell Bison LPS 19th floor mid-flat kitchen: Room load test

Figure F28: Lateral deflection of lounge/kitchen cross wall in Sandwell Bison LPS 19th floor mid-flat kitchen: Room load test

Figure F29: Elevations of cross walls showing location of displacement monitoring transducers: Sandwell Bison LPS 19th floor mid-flat kitchen: Room load test

Figure G1: Two Liverpool Bison Wallframe dwelling blocks

Figure G2: Schematic elevation/vertical section showing locations where the static overpressure load testing (full room test) and the element removal operation (spine wall test) were undertaken in the Liverpool Bison Wallframe LPS block

Figure G3: Location of the combined wall and floor overpressure tests (Full room test)

Figure G4: Location of the element removal operation (Spine wall test)

Figure G5: Incorrectly installed reinforcement

Figure G6: Correctly installed reinforcement

Figure G7: Schematic of overpressure created by an internal gas explosion applied simultaneously to the walls and floors

Figure G8: Schematic of hydraulic loading rig for room overpressure tests, where the test loads are applied simultaneously to the walls and floors as bending and shear loads

Figure G9: Schematic of hydraulic loading rig used to apply test loads to the wall and floor slabs in a single room (NB. The set-up shown applies test loading to two adjacent pairs of wall panels)

Figure G10: Hydraulic loading rig for room overpressure tests, where the test loads are applied simultaneously to the walls and the floor and ceiling slabs

Figure G11: Schematic representation of location of movement transducers used to monitor behaviour of key structural components during room overpressure load test

Figure G12: Liverpool Bison Wallframe combined wall and floor test: floor panel displacements: Test 1

Figure G13: Liverpool Bison Wallframe combined wall and floor test: wall panel displacements: Test 1

Figure G14: Liverpool Bison Wallframe combined wall and floor test: floor panel displacements: Test 2

Figure G15: Liverpool Bison Wallframe combined wall and floor test: wall panel displacements: Test 2

Figure G16: Correctly formed welded tie between lounge and kitchen floor slabs (in doorway)

Figure G17: General view showing a section of the safety loading shores and instrumentation frame

Figure G18: Close-up of primary jack (two jacks each side of wall) and safety loading shore (left) arrangement prior to removal of head of spine wall

Figure H1: Effect of load distribution on the predicted maximum deflection at mid-span of the flank wall flat lounge floor slab – at the joint between the two floor slabs

Figure H2: Effect of support conditions on the predicted maximum deflection at mid-span of the flank wall flat lounge floor slab – at the joint between the two floor slabs

Figure H3: Effect of concrete tensile strength on the predicted maximum deflection at mid-span of the flank wall flat lounge floor slab – at the joint between the two floor slabs

Figure H1: The F(n) = P(N_d > n) < An + g16 requirement for one year

Figure K1: Estimated circa 15–20 N/mm² site mix concrete (representative of that used as a bed for wall panels, for example in the Fram Russell system)

Figure K2: Dry pack – introduced from top of mould – lightly hand compacted using timber tamper, finished level with steel trowel, textured using stiff brush (based upon the approach employed for the Bison Wallframe LPS form of construction)

Figure K3: Dry pack (as for A09/9197/1) – introduced from top of mould – lightly hand
compacted using timber tamper, minimal finished with steel trowel (based upon the approach employed for the Bison Wallframe LPS form of construction)

Figure K4:  Dry pack (as for A09/9197/1) mix prepared by firm side ramming in increments by small section hand held timber compactor. This is considered to represent a ‘reasonable’ level of workmanship (again based upon the approach employed for the Bison Wallframe LPS form of construction). The dry pack mix was allowed to air dry for 30 minutes prior to use

Figure K5:  Dry pack (as for A09/9197/1) mix prepared by light/inconsistent side ramming in large increments by small section hand held timber compactor. This is considered to represent a ‘poor’ level of workmanship (based upon the approach employed for the Bison Wallframe LPS form of construction).

Figure K6:  Variation in surface finish between samples (Low surface roughness (far left) to pronounced surface roughness (far right))

Figure K7:  General view of static friction test set-up

PLATES

Plate 1:  Direct stress contours in Leeds LPS 4th floor lounge floor slab for an applied uniformly distributed overpressure load (1 kN/m²): as associated with an internal gas explosion

Plate 2:  Direct stress contours in Leeds LPS lounge floor slab from applied test load

Plate 3:  Linear finite element model (only lower five storeys of block shown) showing predicted deflections in the vertical (z) direction (kitchen–lounge spine wall present)

Plate 4:  Linear finite element model (only lower five storeys of block shown) showing predicted deflections in the vertical (z) direction (kitchen–lounge spine wall absent)

Plate 5:  Linear finite element model (only lower three storeys of block shown) showing predicted deflections in the vertical (z) direction (kitchen–lounge spine wall present)

Plate 6:  Linear finite element model (only lower three storeys of block shown) showing predicted deflections in the vertical (z) direction (kitchen–lounge spine wall absent)

Plate 7:  Solid element model of half of a lounge test floor slab (symmetry assumed about mid-span of test floor)

Plate 8:  General view of finite element model showing cast-in voids in the precast concrete lounge floor slabs

Plate 9:  Type and distribution of steel reinforcement in finite element model of the precast concrete lounge floor slabs

Plate 10:  Vertical displacement pattern predicted by the numerical analysis for the flank wall flat lounge floor slab (symmetry assumed about mid-span of test floor)

Plate 11:  Stress distribution (von Mises) as predicted by numerical analysis at concrete face immediately below floating screed for the flank wall flat lounge floor slab (symmetry assumed about mid-span of test floor)

Plate 12:  Development of cracks in the soffit of the flank wall flat lounge floor slab (symmetry assumed about mid-span of test floor) – low load intensity: Partition present

Plate 13:  Development of cracks in the soffit of the flank wall flat lounge floor slab (symmetry assumed about mid-span of test floor) – high load intensity: Partition present

Plate 14:  Development of cracks in the soffit of the flank wall flat lounge floor slab (symmetry assumed about mid-span of test floor) – low load intensity: Partition absent

Plate 15:  Development of cracks in soffit of floor slabs – high load intensity: Partition absent

Figure K4:  Dry pack (as for A09/9197/1) mix prepared by firm side ramming in increments by small section hand held timber compactor. This is considered to represent a ‘reasonable’ level of workmanship (again based upon the approach employed for the Bison Wallframe LPS form of construction).

Figure K5:  Dry pack (as for A09/9197/1) mix prepared by light/inconsistent side ramming in large increments by small section hand held timber compactor. This is considered to represent a ‘poor’ level of workmanship (based upon the approach employed for the Bison Wallframe LPS form of construction).

Figure K6:  Variation in surface finish between samples (Low surface roughness (far left) to pronounced surface roughness (far right)).

Figure K7:  General view of static friction test set-up
LIST OF TABLES

EXECUTIVE SUMMARY

Table ES1: Mapping of LPS dwelling block structural assessment activities – Main steps

TEXT

Table 1A: Target ultimate limit state design life-time reliability index ($\beta$-values)

Table 1B: Postulated relationship between the ultimate limit state design life-time reliability index ($\beta$-values) and the probability of occurrence (failure), $P_f$

Table 2: Overview of the various full-scale structural tests undertaken by BRE within three LPS dwelling blocks

Table 3: Concrete strength parameters – range of values for floors slabs and wall panels

Table 4: Assessment Level 1: Deterministic linear elastic analysis (eg spreadsheet based)

Table 5: Assessment Level 2: A deterministic non-linear finite element analysis

Table 6: Assessment Level 3: Probability-based calculations (structural reliability evaluation)

APPENDICES

Table B1: Reported and significant explosions in the UK building stock: all causes

Table B2: Reported and significant explosions in the UK building stock: piped gas

Table B3: Reported and significant explosions in the UK building stock: other gaseous substances, including cylinder gas

Table B4: Reported and significant explosions in the UK building stock: 1984–1994

Table B5: Yearly probability of explosions in UK dwellings: 1984–1994

Table B6: Target ultimate limit state design life-time reliability index ($\beta$-values) for a structure and the associated life-time probability of occurrence (failure)

Table B7: Annual probability of death of an individual for various United Kingdom age groups based upon deaths in 1999 (after Table 2 of reference [26])

Table B8: Annual probability of death of an individual from various causes averaged over the entire population (after Table 2 of reference [26])

Table B9: Annual probability of death from industrial accidents to employees for various industry sectors (based upon Health and Safety Commission figures 2001)

Table C1: Severity/Consequence of hazard

Table C2: Frequency or probability of occurrence of hazard

Table C3: Risk assessment – template for risk quantification and profiling

Table C4: Simplified estimate of the notional cost of preventing a fatality in an LPS dwelling block considering a remedial works strategy (Approach 1) and the replacement of the block (Approach 2) with a new building complying with contemporary Building Regulation requirements (see Note at foot of Table)

Table E1: Summary details of static load tests conducted in LPS dwelling blocks by BRE undertaken before 2000

Table F1: ‘Equivalent UDL’ values for the Reema Conclad LPS dwelling block, Leeds 4th floor lounge: Floor overpressure test

Table F2: ‘Equivalent UDL’ values for the Reema Conclad LPS dwelling block, Leeds 8th floor bedroom: Combined wall and floor overpressure test: Floor slab

Table F3: ‘Equivalent UDL’ values for the Reema Conclad LPS dwelling block, Leeds 8th floor bedroom: Combined wall and floor overpressure test: Flank wall

Table F4: ‘Equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 19th floor lounge: Combined wall and floor overpressure test: Floor slab

Table F5: ‘Equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 19th floor lounge: Combined wall and floor overpressure test: Flank wall – Kitchen end

Table F6: ‘Equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 19th floor lounge: Combined wall and floor overpressure test: Flank wall – Window end

Table F7: ‘Equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 19th floor mid-kitchen: Combined wall and floor overpressure test: Cross walls
Table G1: Overview of floor slab mid-span deflections and recovery values for two room overpressure tests (Test 1 and Test 2)

Table H1: The range of material properties used and assumptions made – modelling of removal of kitchen–lounge spine wall element

Table H2: The range of material properties used and assumptions made – modelling of lounge floor slabs for room overpressure test

Table I1: Overview of a number of potential strengthening options

Table K1: Test results – Measured static friction angle at which slippage occurred
**KEYWORDS**
Accidental actions, accidental loading, assessment, collapse resistance, deflagrations, disproportionate damage, gaseous explosions, Large Panel System (LPS) dwelling blocks, precast concrete structures, progressive collapse, robustness, Ronan Point.

**ABBREVIATIONS**
Abbreviations employed in this document include:

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>ALARP</td>
<td>'As low as reasonably practicable': also see SFARP</td>
</tr>
<tr>
<td>ASELB</td>
<td>Association for Structural Engineers of London Boroughs</td>
</tr>
<tr>
<td>BRE</td>
<td>Building Research Establishment Ltd</td>
</tr>
<tr>
<td>BS</td>
<td>British Standard</td>
</tr>
<tr>
<td>BSI</td>
<td>British Standards Institution</td>
</tr>
<tr>
<td>CBDG</td>
<td>Concrete Bridge Development Group</td>
</tr>
<tr>
<td>CEB</td>
<td>Comité Euro-International du Béton</td>
</tr>
<tr>
<td>CEN</td>
<td>Comité Européen de Normalisation (European Committee for Standardisation)</td>
</tr>
<tr>
<td>CIB</td>
<td>Conseil International du Bâtiment (International Council for Research and Innovation in Building and Construction)</td>
</tr>
<tr>
<td>CROSS</td>
<td>Confidential Reporting on Structural Safety</td>
</tr>
<tr>
<td>DCLG</td>
<td>Department of Communities and Local Government</td>
</tr>
<tr>
<td>DETR</td>
<td>Department of the Environment, Transport and the Regions</td>
</tr>
<tr>
<td>DTI</td>
<td>Department of Trade and Industry</td>
</tr>
<tr>
<td>EN</td>
<td>European Standard (European Norm)</td>
</tr>
<tr>
<td>ENV</td>
<td>European pre-standard of the Eurocodes</td>
</tr>
<tr>
<td>ERIC</td>
<td>Eliminate – Reduce – Inform about – Control</td>
</tr>
<tr>
<td>FEA</td>
<td>Finite element analysis</td>
</tr>
<tr>
<td>FEM</td>
<td>Finite element modelling</td>
</tr>
<tr>
<td>fib</td>
<td>Fédération Internationale du Béton (International Federation for Structural Concrete)</td>
</tr>
<tr>
<td>FRP</td>
<td>Fibre reinforced polymer</td>
</tr>
<tr>
<td>HSE</td>
<td>Health and Safety Executive</td>
</tr>
<tr>
<td>ISE</td>
<td>Institution of Structural Engineers</td>
</tr>
<tr>
<td>ISO</td>
<td>International Standardization Organisation</td>
</tr>
<tr>
<td>LPG</td>
<td>Liquefied petroleum gas, commonly referred to as bottled gas or cylinder gas</td>
</tr>
<tr>
<td>LPS</td>
<td>Large Panel System</td>
</tr>
<tr>
<td>NDE</td>
<td>Non-destructive evaluation</td>
</tr>
<tr>
<td>NDT</td>
<td>Non-destructive testing</td>
</tr>
<tr>
<td>PII</td>
<td>Partners in Innovation scheme</td>
</tr>
<tr>
<td>prEN</td>
<td>pre-normative European Standard (a provisional standard issued for comment)</td>
</tr>
<tr>
<td>QA</td>
<td>Quality assurance</td>
</tr>
<tr>
<td>QC</td>
<td>Quality control</td>
</tr>
<tr>
<td>SCOSS</td>
<td>Standing Committee on Structural Safety</td>
</tr>
<tr>
<td>SFARP</td>
<td>'So far as is reasonably practicable': also see ALARP</td>
</tr>
<tr>
<td>WLC</td>
<td>Whole life cost</td>
</tr>
</tbody>
</table>
This section provides definitions for a range of terms, keywords and phrases used within this and other documents concerned with topics such as inspection, assessment, evaluation, repair, maintenance and management of concrete structures.

This list includes various words and terms, which are commonly employed, sometimes in an almost interchangeable but confusing manner, by many different users. These difficulties seem to arise because different users appear to have quite separate understandings of the meaning and usage of these particular terms. It is hoped that these definitions will facilitate a more unified and consistent approach to the structural assessment of LPS dwelling blocks.

**Accidental design situation**
A design situation that considers exceptional conditions, which may act upon the structure or relate to its exposure or environmental circumstances. These considerations may include fire, explosion, impact or local failure scenarios.

**Accidental loads and actions**
The BSI Structural Eurocode BS EN 1990: 2002: *Basis of structural design* (London, BSI, 2002) defines an ‘accidental action’ as an ‘action, usually of short duration but of significant magnitude, that is unlikely to occur on a given structure during the design working life’. This definition is qualified by two notes, which state that:
1. An accidental action can be expected in many cases to cause severe consequences unless appropriate measures are taken.
2. Impact, snow, wind and seismic actions may be variable or accidental actions, depending upon the available information upon statistical distributions.

In the case of the assessment of LPS dwelling blocks within the UK for accidental loads and actions, consideration is given to exceptional conditions associated with fire, internal gas explosion, impacts by various forms of vehicle and the consequences of localised structural failure.

Seismic actions/earthquakes are taken to be an accidental action. However, because of the low probability of their occurrence and the expected small magnitude of events of this nature within the UK, no specific consideration has been given to seismic actions in respect of the assessment of existing LPS dwelling blocks within the UK. Reference may be made to BSI PD 6698:2009: *Recommendations for the design of structures for earthquake resistance to BS EN 1998* (London, BSI, 2009) for further information. (NB. Seismic actions are outside the scope of this document.)

**Action**
There are two considerations:
a) A set of forces (loads) applied to the structure (direct action).
b) A set of imposed deformations or accelerations caused for example, by temperature changes, moisture variation, uneven settlement or earthquakes (indirect action).

There is a series of associated conditions:

**Permanent action**
An action that is likely to act throughout a given reference period and for which the variation in magnitude with time is negligible, or for which the variation is always in the same direction (monotonic) until the action attains a certain limit value.

**Variable action**
An action for which the variation in magnitude with time is neither negligible nor monotonic.

**Seismic action**
An action that arises due to earthquake ground motions. (NB Seismic actions are outside the scope of this document.)

**ALARP**
‘As low as reasonably practicable’: this is a legal term employed in the UK to imply that the responsible individual has a duty to eliminate or reduce all risks, unless the cost of doing to do so is grossly disproportionate to the benefit. Also see SFARP.

**Assessment**
A process of gathering and evaluating information about the form and current condition of a structure or its components, its service environment and general circumstances, whereby its adequacy for future service may be established against specified performance.
requirements, loadings, durability or other criteria. This activity may include prognoses about future condition and potential performance.

**Basement**
In the Building Regulations, Approved Document B – *Fire Safety – Volume 1: Dwelling Houses* (London, TSO, 2006 edition) a basement is defined as ‘A storey with a floor, which at some point is more than 1.2 metres below the highest level of ground adjacent to the outside walls’.

The National House-Building Council (NHBC) Technical Guidance Note 13: The Building Regulations 2004 Edition – England and Wales Requirement A3 – Disproportionate Collapse (Milton Keynes, NHBC) concerning disproportionate collapse states that ‘To qualify as a basement storey, the distance between external ground level and the top surface of the basement floor should be at least 1.2 m for a minimum of 50% of the plan area of the building’.

For the purposes of this guide, the NHBC Technical Guidance Note 13 definition of a basement is used.

**Collapse**
Catastrophic physical disruption, giving-way or breakdown of the elements or components forming part or all of a structure, such that the structure is unable to perform its intended function and may have largely lost its original form and shape. Collapses may be sudden occurrences, giving little warning of the impending calamity. Once initiated, a collapse event is likely to be largely complete in a few seconds. A large structure could experience a sequence of collapse events, depending upon its form and the nature of the damage or deterioration which has occurred.

**Collapse resistance**
Collapse resistance is a measure of the ability of a structural system to resist the effects of defined accidental loads or actions occurring at or below a defined threshold.

Collapse resistance provides a measure of the sensitivity of a structural system to specified accidental loads or actions and the ability of the structural system to survive them. Collapse resistance is a combined property of the structural system and the loading/action configuration applied to it. It is influenced by numerous factors including the strength, form and ductility of the structural system; which have a bearing upon the possible causes of initial local failure. In circumstances where the accidental loads or actions result in impulsive loads being applied to the structure, collapse resistance will be greatly influenced by the strength of the elements involved and the ability of the structural system concerned to absorbed energy. Collapse resistance issues may apply at both local and global structural system levels.

**Connections**
Refer to the entry for ‘joints’.

**Cross wall**
Internal loadbearing walls orientated perpendicular to main longitudinal axis of the LPS dwelling block. These walls provide stability against lateral loads acting perpendicularly to the main longitudinal axis of the LPS dwelling block (ie acting on the main elevations of the building).

**Damage**
Physical disruption or change in the condition of a structure or its components, brought about by external actions and influences, such that some aspect of either the current or future functionality of the structure or its components will be impaired.

**Deflagration**
The process of combustion, involving the rapid chemical combination of a flammable gas with oxygen, where the rate of propagation of the combustion front occurs at a velocity which is less than the speed of sound in the unreacted mixture of gas and air.

**Deficiency**
A specific deficiency or inadequacy in the structure or its component parts, which materially affects their ability to perform some aspect of their intended function, either now or at some future time.

**Deflection**
This is a lack of something, possibly arising as a result of an error in design, specification or construction, which affects the ability of the structure to perform some aspect of its intended function, either now or in the future. Often concerned with specific issues, such as strength or ductility, but may be more general in nature and concern matters such as durability.

**Design life**
The intended period of useful service of the structure under the expected conditions of use and with the implementation of the maintenance, preventive and/or remedial measures envisaged at the time of design.

**Deterioration**
This is a worsening of condition with time, or a progressive reduction in the ability of a structure or its components to perform some aspect of their intended function. Typically these changes will be driven by chemical, mechanical or physical processes or agents, or a combination thereof.

**Detonation**
This is a process of extremely rapid combustion, involving the extremely rapid exothermic chemical reactions, where the rate of propagation of the combustion front occurs at a velocity which is greater than the speed of sound in the unreacted medium. The term is commonly used to describe the type of explosion created by a bomb or other form of military/high explosive material.

**Diagnosis**
Identification of the cause or explanation of the mechanism(s) by which a phenomenon affects the behaviour or condition of a structure or its components.
based upon an investigation of the signs and indications exhibited therein. The term is typically applied to forms of deterioration and degradation or other mechanisms causing an alteration in the expected or desired behaviour of the structure or its components.

Disintegration
Severe physical damage and disruption of a structure or its components, which results in its localised or more general break-up into fragments, with an associated gross impairment of their functional capability.

Disproportionate damage
A collapse in which the failure front moves progressively away from the initial trigger point of local damage to envelope portions of the structure significantly larger than the part directly damaged by the initial incident such that the overall degree of damage is out of proportion with the magnitude of the initiating event.

A quantitative judgement on the issue of ‘proportionality/disproportionality’ could potentially be developed by reference to the design objectives for a structure if these defined the nature and magnitude of initiating events, which were to be considered along with the tolerable extent of the associated damage.

Diversity
Structural diversity is the concept that there can be a number of alternative ways of achieving specified structural performance requirements, which are not dependent upon the same form of structural behaviour. The term also acts as an indicator of the extent that this concept may be applicable to a particular structural system. For example, if a building has a high degree of structural diversity, this would indicate that its sensitivity to the effects of local damage is reduced, which accordingly would improve the resilience of the structural system. An example of this concept is the ability of a framed structure to develop catenary action following local damage, which impairs/disables the original load-carrying mechanisms, such as the loss of a column member. Ideally the structurally behaviours should be independent involving different principles and mechanisms of behaviour.

Ductility
Ductility is the ability of a structural material to accommodate a large degree of plastic deformation without breaking or experiencing a significant reduction in its toughness.

Ductility is also the ability of a structural system to accommodate large displacements or rotations without catastrophic failure, and/or to absorb or dissipate large amounts of energy while ensuring that specified overall displacements or rotations remain within acceptable specified bounds.

Effect of action
This is the effect of actions (or action effect) on:
- structural members in terms of the mobilising of phenomena such as internal forces, moments, stress, strain etc. or
- on the whole structure in terms of outcomes such as deflection, rotation, etc.

Environment
The characteristics of the atmosphere or surroundings to the structure affecting and influencing its future durability and performance. In the context of this report, these are taken to be factors influencing durability (rather than say structural loadings associated with wind or other effects). These factors need to be taken into account during the review of service life/estimating the residual life of a particular parts of an LPS dwelling block. Environmental factors may need to be considered at different scales ranging from macro level (affecting the overall structure), meso level (affecting, say, an individual element or component) down to micro level (localised influences affecting a small part of a structure, e.g. a particular joint).

Equivalent uniform static pressure
A statically applied internal pressure applied to the bounding elements of an enclosed volume representing the dynamic overpressure arising from a piped or a non-piped gas explosion. This concept is also potentially applicable to some other forms of explosion, but its applicability would need to be considered knowing the specifics of the envisaged situation and the nature of the hazard.

Explosion
Is an incident whereby rapid combustion of particular substances creates an overpressure, which, when applied to structural and non-structural building components, typically causes an intense short duration increase in the imposed loading applied to such elements.

In the context of LPS dwelling blocks, consideration is restricted to gaseous explosions (deflagrations) and excludes detonations (bombs). Gaseous explosions involve the rapid combustion of flammable gases. In dwellings, gaseous explosions typically involve gas from either a piped source or a portable storage cylinder. Past experience of gaseous explosions indicates that those involving gas from a piped source have the potential to be more severe (larger and more damaging) than those where the gas did not originate from a piped source.

Piped gas explosion
A gaseous explosion involving a source of piped gas.

Non-piped gas explosion
A gaseous explosion not involving a source of piped gas.

Flank wall
These are loadbearing cross walls orientated perpendicular to the main longitudinal axis of the LPS dwelling block, which occur primarily at the ends of the LPS dwelling block forming part of the external perimeter of the LPS dwelling block. Flank walls are also found in re-entrant zones in the external perimeter of the LPS dwelling block (as viewed on plan). These walls provide...
stability against lateral loads acting perpendicularly to the main longitudinal axis of the LPS dwelling block (ie acting on the main elevations of the building).

**Hazard**
A hazard is a situation or physical substance with the potential to cause harm through the occurrence of an undesired event, which releases that potential. Typically physical hazards involve stored energy; examples of which are include potential energy arising from gravity, kinetic energy, chemical energy, electrical energy, etc.

**Global hazard**
A hazard affecting the whole structure.

**Local hazard**
A hazard affecting only part of the structure or arising from a particular local feature, such as the possibility debris or loose concrete falling from a building due to cracking or spalling of concrete on a cladding panel. Another example of a local hazard is the possibility of the displacement of a damaged or defective façade component.

In the above example, the local hazard poses a risk to public safety by virtue of the potential consequences that might arise from the falling debris striking and injuring/ killing a person. See also the definition of ‘risk’.

**Ingress**
The entry of substances into structural and/or non-structural components forming the fabric of a building. Often the term ‘ingress’ is associated with the entry of substances, which cause deterioration (eg chlorides into reinforced or pre-stressed concrete, sulfates and carbon dioxide (CO₂) into concretes).

**Inspection**
A primarily visual examination, often at close range, of a structure or its components with the objective of gathering information about their form, current condition, service environment and general circumstances.

**Intervention**
This is a general term relating to an activity or series of activities taken to modify or preserve the future performance of a structure during service, usually in the context of works to improve its durability and extend its anticipated service life. Interventions may be planned or unplanned, but in the former situation the activity would tend to be classified as a maintenance intervention. Thus intervention activities might be instigated for the purposes of preventive treatment, repair, and so on, of the structure concerned. They may also be undertaken as a pro-active intervention (applying some form of treatment / taking action prior to damage becoming visible) or as a reactive intervention (taking action after damage has become visible, eg cracking or spalling of concrete).

**Investigation**
The process of inquiry into the cause or mechanism associated with some form of deterioration or degradation of the structure and the evaluation of its significance in terms of their current and future structural functionality. The term may also be employed during the assessment of defects and deficiencies. The process of inquiry might employ sampling, testing and various other means of interrogating/gathering information about the structure, as well as theoretical studies to evaluate the importance of the findings in terms of the functionality/performance of the structure.

**Joint**
This is the zone in which precast concrete components are brought together and connected in various ways to form part of the overall building. Accordingly joints are made in situ, typically using a combination of in-situ concrete or mortar and reinforcement or metallic devices to create the required physical connection required to transmit forces. The term ‘connection’ is reserved for the physical/mechanical devices or mechanisms which act within the joints.

There are various types of joint associated with the connection of wall panels and floor slabs. At a generic level these include wall–wall joints, wall–floor joints and floor–floor joints. However, an extended series of more specific joint types need to be differentiated during a structural assessment of an LPS dwelling block. The types recognised typically include cross wall–floor slab joints, spine wall–floor slab joints, flank wall–floor slab joints, flank wall–spine wall joints, flank wall–flank wall joints, cross wall–elevation wall joints, and so on.

**Key structural elements/components**
In the case of an LPS dwelling block, a key structural element/component is one upon which the stability of the adjacent parts/remainder of the structure depends.

Almost all structural elements in an LPS dwelling block are load bearing or contribute to the stability of the loadbearing elements of LPS dwelling block – thus all load bearing elements and stabilising elements (eg floor slabs) may be considered to be ‘key elements’ in the sense that this term is defined in the 2004 edition of The Building Regulations, Approved Document A – Structure (London, NBS, 2004).

**Large panel system (LPS)**
Large panel system (LPS) dwelling blocks typically comprise large precast reinforced concrete floor and roof units spanning on to large structural precast concrete wall panels. The precast storey-height and room-width wall and floor panels form the structural system for the building. Vertical loads are carried to the ground through the structural wall panels, which commonly contain very little steel reinforcement and are treated as being plain (unreinforced) concrete for the purposes of structural design or assessment. Stability against lateral loads is also provided by the structural wall panels, with those orientated across the short dimension of the building being referred to as cross-walls, or flank walls if they are the exterior walls located at the ends of the building. Some structural walls are orientated along the long dimension of the building, providing lateral stability in
that direction. Such walls are often referred to as spine-walls.

**Local collapse, damage and/or failure**
Damage, collapse or severe impairment of a localised part of the structure (eg an individual load bearing structural component) such that it is unable to carry its intended load or serve its intended structural function.

Note: In order to maintain the stability of the overall building in these circumstances it is necessary for the load originally carried by the affected structural component(s) to be transferred to adjacent structural components or to be carried via secondary structural actions. See entry for ‘redundancy’.

**Linear finite element analysis**
Linear models use simple parameters and assume that the material is not plastically deformed or cracked. For example, if the applied forces are doubled, the displacements increase in a linear manner and they also double. Behaviours that differ from this response are termed non-linear.

**Maintenance**
A planned (usually periodic) activity intended to either prevent or correct the effects of minor deterioration, degradation or mechanical wear of the structure or its components in order to keep their future structural functionality at the level anticipated by the designer.

**Malicious act or attack**
A deliberate act carried out with the intention of doing harm. Generally a premeditated act characterised by malice. The potential forms of malicious attack are wide ranging. The intended consequences of the act may be to cause significant loss of life and injuries and/or economic loss, eg causing severe damage and possible collapse of buildings or other structures, with or without a warning being given. Malicious attacks have included internal and external explosions that were deliberately caused and vehicle impacts, as well as direct mechanical damage to vulnerable parts of structures (eg bridge stay cables).

(NB. Malicious acts or attacks are outside the scope of this document).

**Overpressure**
A uniform pressure applied to the surfaces of an enclosed volume and arising from a deflagration/gaseous explosion notionally occurring within that volume. Explosions/ deflagrations generally result from the ignition of an explosive mixture of gas and air within enclosed volume, with the associated increase in pressure in the enclosed volume, creating a short duration accidental/abnormal load. As such the overpressure is assumed to be applied in addition to the dead load and a proportion of the design imposed (live) load.

**Progressive collapse**
A collapse in which the failure front moves progressively away from the initial trigger point of local damage to envelope portions of the structure significantly larger than the part directly damaged by the initial incident.

Note: The extent of a progressive collapse may be proportionate to the nature and/or magnitude of the initial incident.

**Protection**
An activity or series of activities undertaken to seek to defend a structure from the effects of further or future
deterioration by providing a physical or chemical barrier to aggressive species (e.g., chloride ions) or other deleterious environmental agents upon the in-service performance and durability of a structure. This may be provided in a number of ways such as by surface coatings, impregnation treatments, electro-chemical treatments, enclosure of the surface, etc. applied to the concrete structure, elements or parts thereof.

**Recalculation**
A process of analytical examination using mathematical models or simplified representations of a structure or elements thereof to make an estimate of their structural functionality. Typically this is concerned with in-service performance assessment and structural load capacity in particular. The process may utilise similar steps and procedures to design but fundamentally differs from this by seeking to take into account the actual form and condition of the structure as found, including deterioration. This will often include a more realistic consideration of the actual loading regimes, rather than the idealised values used in design. The recalculation process may be used to predict future structural performance taking into account the influence of ongoing deterioration processes and any preventative or remedial intervention activities.

**Redundancy**
The provision within a structural system of additional components able to provide alternative load paths in the event of there being a collapse or severe impairment of a localised part of the structure such that it is unable to carry its intended load or serve its intended structural function.

*Note:* In order to maintain the stability of the overall building in these circumstances it is necessary for the load originally carried by the affected structural component(s) to be transferred to adjacent structural components or to be carried via secondary structural actions.

**Repair**
Generally an activity undertaken to reinstate to an acceptable level the current functionality of a structure or its components which are either defective, deteriorated, degraded or damaged in some way and without restriction upon the materials or methods employed. The action may not be intended to bring the structure or its components so treated back to its original level of structural functionality or durability. The work may sometimes be intended simply to reduce the rate of deterioration or degradation, without significantly enhancing the current level of structural functionality.

**Resilience**
Resilience is the ability of a building or structural system to withstand an accidental or exceptional loading incident without experiencing an undue degree of damage, such that progressive collapse or disproportionate damage occurs.

**Risk**
Risk is a quantity that is measured in units of the undesired consequences of failure per unit time (e.g., expected number of fatalities, expected number of serious injuries, expected financial losses, etc.) and is thus a vector quantity. The word ‘expected’ in this context means the average value of the losses per unit time taken over a long period of time, or many repetitions of the same scenario. Risk can therefore be thought of as the magnitude of undesired consequences of failure multiplied by the probability that the failure event will occur in a specified period of time, e.g., a year.

In engineering situations the concept of risk is typically expressed by an equation in the following format:

\[
\text{Risk (consequence/unit time)} = \text{Frequency (event/unit time)} \times \text{Magnitude (consequence/event)}
\]

**Acceptable risk**
This is a risk whose magnitude is considered to be sufficiently small and adequately controlled such that it may be borne without modification.

**Tolerable risk**
This is a risk whose magnitude is sufficiently small that it is considered to be broadly acceptable but which must be reduced as low as reasonably practicable (ALARP) by appropriate risk reduction measures.

**Robustness**
Robustness is the ability of a building subject to accidental or exceptional loading or other action to sustain damage or local failure without experiencing a disproportionate degree of overall distress or collapse.

**Qualifying observations**
In buildings ‘robustness’ is commonly taken to be the ability of the structural system to mobilise alternative load paths around an area of local damage, which is typically a function of the degree of redundancy within the structural system together with the strength and ductility of the elements and joints between them.

Whilst this approach is satisfactory when the extent of local damage is relatively small, perhaps involving one or two vertical load bearing elements, mobilising alternative load paths within the structural system tends to have the effect of increasing the overall extent of damage caused across the building, although typically the severity and consequences of that damage is reduced. It should be recognised that the above scenario implies that there are differing probabilities of different degrees of damage occurring in various parts of the structure.

However, when the initial damage to the building could be more extensive than the local failure described above, the overall extent of collapse and damage may be better controlled by segmenting the building into zones by the introduction of joints or discontinuities which are intended to limit the propagation of collapse and damage.
across the building. Thus it would be intended that a collapse in one such zone should not propagate across the boundary joints or discontinuities into the adjacent zones of the building.

Thus robustness is related not only to the strength, form and ductility of the structural system, but may also be influenced by the division of a building into zones by joints or discontinuities. Robustness is a quality of the structural system alone and is independent of the cause of the damage and/or the probability of initial local failure.

However the Structural Eurocode BS EN 1991-1-7: Eurocode 1 – Actions on structures: Part 1-7: General actions – Accidental actions (including National Annex) (London, BSI, 2006) gives a broader definition which does include possible causes of the initial failure. The definition given in Clause 1.5.14 of BS EN 1991-1-7 is: ‘Robustness is the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause’.

SFARP
‘So far as is reasonably practicable’: This is a legal term used in the UK in the Construction (Design and Management) Regulations 2007 (London, HMSO) with respect to the Duties of Designers. Also see ALARP.

Spine wall
Internal loadbearing walls orientated parallel to main longitudinal axis of the LPS dwelling block. These walls provide stability against lateral loads acting along the longitudinal axis of the block.

Stability
This is the state of being stable, which implies that the performance or functionality of a structural component or structural system will remain largely unchanged with time and is unlikely to be disturbed or diminished by an anticipated external trigger event or set of circumstances/actions such that its ability to carry its intended load and to serve its intended structural function would be significantly diminished.

Strengthening
An activity or series of activities undertaken to increase the strength of a structure or its component parts, to improve structural stability, the robustness and/or the collapse resistance of the overall building or parts thereof.

Structural functionality
The ability of the structure to perform the structural function/purpose intended by the designer. This may be evaluated under various headings and consideration would normally be given to a number of issues affecting either the whole structure, or parts thereof. The issues would typically include the ultimate load case(s) and possibly also various other serviceability limit state cases (eg deflection, vibration, thermal movements, etc.).

Structural integrity
The ability of structural components to act together as a competent single structural entity.

Survey
The process, often involving visual examination but which may utilise various forms of sampling and testing, whereby information is gathered about the form and current condition of a structure or its components. The term may be applied to the inspection of a number of similar structures/components to obtain an overview. The term is also used to describe the formal record of inspections, measurements and other related information which describes the form and current condition of a structure and its components.

Testing
Processes or procedures whereby data or some form of information is obtained about the current condition or performance of the structure or its components. Various types of testing are recognised, classification being primarily on the basis of the amount of damage or interference caused to the structure. The main divisions are:

- Non-invasive testing: where no damage is caused to the structure by the test procedure (such as covermeter, radar, etc.) as there is no physical penetration into the structure.
- Non-destructive testing (NDT): which utilises testing methods which may cause a small degree of superficial damage or marking of the surface finishes (such as pull-out tests, ultrasonic pulse velocity, material sampling, load testing in elastic range, etc.).
- Structural load and response testing.

The combined use of several of the above methods may be termed non-destructive evaluation (NDE).

Venting panel
A non-structural part of the bounding elements of an enclosed space/room with limited resistance to laterally applied load which is intended to relieve developing pressure from a deflagration in order to reduce the overpressure loading applied to structural parts of the building. In the case of an LPS dwelling block almost all wall and floor components are load bearing or contribute to the stability of the loadbearing elements of the LPS dwelling block.

Vulnerability
The susceptibility of a structural component or structural system to damage and potential collapse.
EXECUTIVE SUMMARY

Background – Ronan Point and the evolution in technical requirements. Since the progressive collapse of part of Ronan Point following an internal gas explosion in 1968, LPS dwelling blocks have effectively been treated as a special class of UK building. Requirements for their structural assessment for normal and accidental loads have been given in Ministry of Housing and Local Government (MHLG) Circulars 62/68 and 71/68, both of which were titled Flats constructed with precast concrete panels. Appraisal and strengthening of existing blocks: Design of new blocks. Later BRE Report 107: Part 2 gave additional guidance on the structural adequacy and durability of LPS dwelling blocks. MHLG Circulars 62/68 and 71/68 published in 1968, along with various other related guidance from that era, were never withdrawn and notionally remain in force today. However, that guidance has become outdated by subsequent technical developments in structural assessment procedures, and by changes in the philosophy behind regulatory requirements and how this has been embodied in new codes of practice, such as the Eurocodes. This Handbook provides revised technical performance requirements (in section 2) and associated structural assessment guidance.

Reasons for undertaking the programme of LPS dwelling block work and producing revised guidance for structural assessment. In the early and mid-1990s LPS dwelling blocks were being appraised, some after more than 30 years in service. BRE found that extensive strengthening works were often being recommended. In a number of instances these appeared to offer only marginal improvement to the existing level of safety. In other instances, viable communities within LPS dwelling blocks were inadvertently destroyed when the decision was made to demolish a block because of the cost of the remedial and related works that were being recommended. Such outcomes, whilst following the guidance available at the time, were judged to offer not only poor value for money but were resulting in what seemed to be unjustified outcomes. Thus there was a need to better-establish the actual performance of LPS dwelling blocks under accidental loads, as well as improving and updating the guidance for the structural assessment of this particular class of buildings.

Types of accidental loading, sources of hazard and benchmarking of risks. A review was made of the hazards to which LPS dwelling blocks are exposed and the associated risks. These hazards include internal and external gas explosions and fire, as well as potential vehicular impacts arising from road vehicles, trains and aircraft. The annual probability of occurrence of these extreme events was found to be very small; details are given in section 6. The review also examined the maximum overpressure likely to be generated during an internal gas explosion. It was found that some explosions involving a piped gas supply had the potential to be significantly more devastating than those not involving a piped gas supply. It was established that for situations where a piped gas supply was not involved in an internal gas explosion, an overpressure of 17 kN/m² formed a reasonable assessment criterion. Conversely, where a piped-gas supply is present in any part of a building, or where it contains a basement or other poorly-ventilated zone where gas from an external source could accumulate, the assessment overpressure criterion should be 34 kN/m².

BRE programme of load testing existing LPS dwelling blocks. This established that, for the LPS dwelling blocks tested, member capacities were compatible with the loads applicable to circumstances where a piped-gas supply is not present in any part of the building (ie the wall and floor panels in the LPS blocks tested were able to resist an overpressure in excess of 17 kN/m²). In the LPS dwelling blocks tested by BRE, the load bearing wall panels generally failed by translation (lateral shear) at the bottom of the wall panels (ie the wall panels performed satisfactorily in bending). The floor slabs failed in bending under the applied load (ie they performed satisfactorily in shear).

Requirements against which LPS dwelling blocks are to be evaluated. An LPS dwelling block exceeding four storeys in height (ie five storeys and higher) will be considered to satisfy Requirement A3 of Approved Document A – Structure if it meets one of the following:

LPS Criterion 1: There is adequate provision of horizontal and vertical ties to comply with the current requirements for Class 2B buildings as set down in the codes and standards quoted in Approved Document A – Structure as meeting the requirements of The Building Regulations.
**LPS Criterion 2:** An adequate ‘collapse resistance’ can be demonstrated for the foreseeable accidental loads and actions as defined below.

**LPS Criterion 3:** Alternative paths of support can be mobilised to carry the load, assuming the removal of a critical section of the load bearing wall in the manner defined for Class 2B buildings in Approved Document A – Structure\(^4\) or alternatively assuming the removal of adjacent floor slabs (taking the floor slabs bearing on one side of the wall at a time) providing lateral stability to the adjacent floor slabs (taking the floor slabs bearing on one side of the wall at a time) providing lateral stability to the critical section of the load bearing wall being considered.

**Collapse resistance** of LPS dwelling block structural elements and the associated load transmitting joints between the structural elements should be evaluated for the forces associated with accidental overpressure values of 17 kN/m\(^2\) or 34 kN/m\(^2\) for the circumstances as defined below\(^1\).

A. An LPS dwelling block with a piped gas supply within or to any part of the building: an assessment overpressure of 34 kN/m\(^2\) should be used generally throughout the building. The overpressure should be applied simultaneously to all surfaces of the single room/bounding enclosure within which the explosion (deflagration) is considered to occur. Alternatively, the approach used for the design of ‘key elements’ in new buildings may be adopted\(^2\).

B. An LPS dwelling block with a basement: an assessment overpressure of 34 kN/m\(^2\) should be used in the basement and in any other zone where an explosive mixture of gas might accumulate (potentially from an external source). The overpressure should be applied simultaneously to all surfaces of the single room/bounding enclosure within which the explosion (deflagration) is considered to occur. Alternatively, the approach used for the design of ‘key elements’ in new buildings may be adopted\(^2\).

C. An LPS dwelling block without a basement and with a piped gas supply to any part of the building: an assessment overpressure of 17 kN/m\(^2\) should be used. The overpressure should be applied simultaneously to all surfaces of the single room/bounding enclosure within which the explosion (deflagration) is considered to occur. Alternatively, the approach used for the design of ‘key elements’ in new buildings may be adopted\(^2\). These requirements can be applied to the part of the dwelling block above the zone defined in Category B above.

**Collapse resistance** is defined as ‘a measure of the ability of a structural system to resist the effects of defined accidental loads or actions occurring at or below a defined threshold’.

---

\(1\) However, for reliability-based assessments it is necessary to recognise and take into account that the 17 kN/m\(^2\) (non-piped gas situations) and 34 kN/m\(^2\) (piped gas situations) assessment criteria values may be exceeded – indeed it is necessary to estimate the probability distribution function of the complete range of possible overpressures for the respective situations.

\(2\) Refer Clauses 5.1d and 5.3 of Approved Document A – Structure\(^4\)

**Structural assessment methodology for LPS dwelling blocks.** The LPS dwelling block structural assessment methodology considers both normal and accidental loads and actions, seeking to identify the plausible hazards. It makes a general evaluation of the risks arising from the various types of accidental loads and actions that such blocks may be exposed to. These hazards include internal and external gas explosions, fire, as well as potential vehicular impacts arising from road vehicles, trains and aircraft. The structural assessment methodology proposed evaluates the collapse resistance of the LPS dwelling block, drawing upon experience gained from full scale structural tests in existing LPS dwelling blocks and historic data gathered about previous gas explosions in the UK building stock.

This demonstrated that the structure should resist the overpressure loads which might plausibly be imposed. The historic data also indicate that the probability of structurally significant vehicular impacts to be very low, such that the associated risks might be considered to be ‘regarded as insignificant and adequately controlled’.

There are four main stages in the LPS dwelling block structural assessment methodology:

- **Assessment Stage 1** – Review of existing technical information.
- **Assessment Stage 2** – Collection of new technical information.
- **Assessment Stage 3** – Assessment of block for normal loading.
- **Assessment Stage 4** – Assessment of block for accidental loading.

The hierarchical approach to the structural assessment of LPS dwelling blocks can potentially involve the following steps, which utilise increasingly sophisticated forms of structural assessment calculation.

- **Assessment Level 1** – A deterministic linear elastic analysis (eg spreadsheet)
- **Assessment Level 2** – A deterministic non-linear finite element (or alternative) analysis
- **Assessment Level 3** – Probabilistic based calculations (structural reliability evaluation)

These and the other main steps in the overall LPS dwelling block structural assessment process are shown as a mapping in Table ES1.

**Implications.** In circumstances where the assessment overpressure criterion to be used is 34 kN/m\(^2\) (eg where a piped-gas supply is present), the load bearing wall panels tend to fail the structural assessment both in bending, at the mid-height of the wall panel, and in shear at the bottom of the wall panel. This would require a considerable amount of strengthening works to be carried out, resulting in a situation which is generally considered to be not economically viable. Under an overpressure criterion of 34 kN/m\(^2\) the floor slabs would be expected to fail the assessment both in bending and shear, again this would require an unrealistically large amount of expensive strengthening works.
Thus the general expectation is that, unless the LPS dwelling block concerned was designed and built to accommodate the overpressure criterion associated with a piped gas supply (ie 34 kN/m²), the LPS dwelling blocks in question will not have a piped gas supply and that they will therefore be assessed for an overpressure criterion of 17 kN/m².

The underlying assessment concept is to seek to verify that structural failure in an LPS dwelling block is unlikely to be initiated under accidental loads and action effects. As a result the structural assessment methodology does not consider post-failure behaviour, such as the ability to mobilise an alternative load path.

However, it is important to recognise that the ability to resist the foreseeable accidental load from an internal gas explosion (deflagration) – that is having an adequate collapse resistance – does not remove the small risk of progressive collapse in an LPS dwelling block should a load bearing component (a ‘key element’) fail or should some form of sufficiently devastating incident occur which initiates failure. In such circumstances, currently it is not entirely clear whether such a progressive collapse would be considered to be ‘proportionate’ or ‘disproportionate’ to the magnitude of the initiating event, bearing in mind the expected very low probability of its occurrence.

Whilst the historically observed performance of LPS dwelling blocks has in general been satisfactory and the results of the BRE load tests performed to the ultimate condition on three (uninhabited) LPS dwelling blocks (two Bison Wallframe and one Reema Conclad) were reassuring, the potential vulnerability of this form of construction has been highlighted by the recent progressive collapse of parts of a number of LPS dwelling blocks during demolition[5]. The LPS dwelling blocks concerned were pre-Ronan Point Fram Russell LPS. Limited investigation of one of the collapses by BRE suggested that the particular LPS dwelling blocks are likely to have been constructed to a poor standard. Particular points were the quality of the tying at the floor slab/cross wall panel junctions, where there was a significant lack of connection (mechanical tying) between floor panels and the cross walls forming a bay, and the lack of end bearing to the floor slabs. BRE is aware that similar LPS dwelling blocks located in another part of the UK have been deconstructed without incident.

However, these incidents do highlight the need to be cautious when making a structural assessment and the critical need for adequate invasive investigation of the nature and quality of construction achieved in individual LPS dwelling blocks before embarking upon structural calculations and other aspects of the structural assessment process.

Future circumstances. The observations in this report are made on the basis of historic data. It is necessary to look to the future and consider the potential implications of higher performance standards being applied to LPS dwelling blocks and what impact issues such as less ‘leaky buildings’ with their lower air leakage/air change rates might have in terms of the likelihood of the occurrence of internal gas explosions and the resulting overpressures which may be generated by them.

<table>
<thead>
<tr>
<th>Activity</th>
<th>Observation</th>
<th>Doc. section [Reference]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Define scope of work</td>
<td>Agree brief and requirements for structural assessment</td>
<td>2, 3 [6]</td>
</tr>
<tr>
<td>Hazard identification</td>
<td>Review potential sources of hazard</td>
<td>5, 11</td>
</tr>
<tr>
<td>Risk evaluation</td>
<td>Influences decision making; use ALARP &amp; SFARP principles</td>
<td>6, 11</td>
</tr>
<tr>
<td>Structural assessment procedures and factors influencing structural behaviour</td>
<td></td>
<td>8, 9, 12</td>
</tr>
<tr>
<td>Assessment Stage 1:</td>
<td>Review of existing technical information</td>
<td>12.5</td>
</tr>
<tr>
<td>Assessment Stage 2:</td>
<td>Collection of new technical information</td>
<td>12.6</td>
</tr>
<tr>
<td>Assessment Stage 3:</td>
<td>Assessment of block for normal loading</td>
<td>12.7</td>
</tr>
<tr>
<td>Assessment Stage 4:</td>
<td>Assessment of block for accidental loading or action</td>
<td>12.8</td>
</tr>
<tr>
<td>Overpressure loadings associated with an internal deflagration</td>
<td></td>
<td>Steps 7</td>
</tr>
<tr>
<td>• Assessment Level 1: Deterministic linear elastic analysis</td>
<td>1 to 21</td>
<td>Table 4</td>
</tr>
<tr>
<td>• Assessment Level 2: Deterministic non-linear finite element analysis</td>
<td>22</td>
<td>Table 5</td>
</tr>
<tr>
<td>• Assessment Level 3: Probabilistic based/structural reliability calculations</td>
<td>23</td>
<td>Table 6</td>
</tr>
<tr>
<td>Post-fire evaluation</td>
<td>Evaluation of structural effects and remedial actions</td>
<td>10.4</td>
</tr>
<tr>
<td>Interventions to enhance durability and strength: Through-life care, etc.</td>
<td></td>
<td>10, 13</td>
</tr>
<tr>
<td>• Maintenance works</td>
<td>Ongoing planned periodic activities to maintain functionality</td>
<td>Glossary</td>
</tr>
<tr>
<td>• Durability</td>
<td>Preventive and remedial durability works options</td>
<td>10.2, 10.3</td>
</tr>
<tr>
<td>• Strengthening</td>
<td>Interventions to enhance strength and damage tolerance</td>
<td>13, Table 34</td>
</tr>
<tr>
<td>Through-life management, monitoring and care</td>
<td></td>
<td>12.9</td>
</tr>
<tr>
<td>• Periodic inspection, monitoring and structural assessment</td>
<td>5, 6, 12.10</td>
<td>Table 35</td>
</tr>
<tr>
<td>• Hazard identification, risk reduction and management measures; use ALARP and SFARP principles</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Demolition/end of life</td>
<td>End of life: Demolition and progressive collapse issues</td>
<td>10.5</td>
</tr>
<tr>
<td>Reporting of findings</td>
<td>Provision of report on findings and recommendations</td>
<td>[6]</td>
</tr>
</tbody>
</table>
REFERENCES


1 INTRODUCTION AND SCOPE OF THE HANDBOOK

1.1 BACKGROUND
Essentially, large panel system (LPS) dwelling blocks are gravity structures, as are traditional masonry constructed buildings. LPS dwelling blocks typically comprise precast reinforced concrete floor and roof components spanning onto storey-height structural precast concrete wall panels. The precast concrete components are connected by various forms of joints made on site. Vertical loads are carried to the ground through the structural wall panels, which also provide stability against lateral loads. Walls orientated across the short dimension of the building are usually called cross-walls, or flank walls if they are the exterior walls located at the ends of the building or in re-entrant zones. The structural walls orientated along the long dimension of the building are often referred to as spine-walls.

Figures 1 and 2 illustrate two common types of LPS dwelling blocks originating from the 1960s or thereabouts. However, few people realise that large panel precast concrete construction was pioneered at about the turn of the 20th century. The Eldon Street flats in Liverpool were built using the system in 1905, with the structural form being remarkably similar to the LPS dwelling block system of building ‘invented’ in the 1950s. A series of three photographs showing: the transportation of the large storey-height wall panels from the precasting yard to site on a trailer towed by a coal-fired traction engine; the erection of the panels on site; and the finished three-storey building, is included in *Historic concrete – background to appraisal*, edited by Sutherland, Humm and Chrimes.

Figure 3 illustrates the general arrangement of the wall and floor panel construction forming a notionally typical LPS dwelling block. In reality there are significant differences in the details of the individual forms of LPS construction, which were proprietary products produced by the various manufacturers of the day. It is also common for there to be differences in aspects of the structural details employed by a particular manufacturer for buildings at different locations, reflecting specific requirements of the building owner or commissioning authority.

---

Figure 1: Bison LPS dwelling block (courtesy Sandwell Metropolitan Borough Council)
Figure 2: Reema Conclad LPS dwelling block (courtesy Leeds City Council)
The structural precast concrete wall panels in LPS blocks designed before 1968 commonly contain very little steel reinforcement\(^1\). Typically the wall panels contain only small amounts of peripheral ‘handling steel’ to accommodate stresses arising during manufacture, transport and erection. The floor components are commonly treated as spanning in one-direction between the cross-walls/flank walls. However, on some occasions floor components act as two-way spanning slabs (ie in Fram Russell LPS dwelling blocks). Although LPS dwelling blocks are understood to be constructed from reinforced concrete, it is possible (but believed to be unlikely) that prestressed concrete floor components could have been used in some LPS dwelling blocks.

In some LPS dwelling blocks the ground floor structure is formed entirely of in-situ concrete or is of precast/in-situ reinforced concrete hybrid construction.

The site-placed steel reinforcement present in the in-situ horizontal joints between wall and floor panels (see Figure 4) has no significant role in the primary structural function of transmitting vertical loads to the ground under normal loading conditions (ie gravity and wind loads).

The overall structural behaviour of LPS dwelling blocks under normal loads (eg gravity and wind, including self-weight, imposed and snow loads), thermal and ground movements, together with accidental loads, is outlined in BRE Report 107: Part 2\(^2\).

Thus, in addition to normal loads, LPS dwelling blocks could be subject to accidental loads and actions arising from incidents such as internal or external gaseous explosions (deflagrations), fire or vehicle impact. In these circumstances the steel reinforcement within the in-situ joints may mobilise continuity forces to avoid the initiation of local failure within the LPS dwelling block.

---

\(^1\) Wall panels in post-1968 LPS dwelling blocks tended to contain significantly more reinforcement compared to wall panels in similar blocks designed/built prior to 1968.

---

\(^2\) If the accidental loads and actions are of a sufficiently large magnitude, local damage could be caused to load bearing components and/or to the in-situ joints between them. In these circumstances it would be necessary to mobilise alternative load paths to maintain the overall stability of the LPS block. Where local failure has occurred, the steel reinforcement within the in-situ joints and panels would need to have sufficient ductility and
strength, plus appropriate detailing, to accommodate the potentially significant deformations and distortions created as alternative load paths are mobilised.

Whilst it also has to be recognised that LPS dwelling blocks could potentially be subject to malicious acts or attacks, such a matter is outside the scope of this document. Perhaps bizarrely, such events could be inadvertent; recent experience has shown that flats within some LPS dwelling blocks have been used by those involved in the manufacture of improvised explosive devices (IEDs). Accordingly, it may be of interest that the Home Office, through the Centre for Protection of National Infrastructure and other UK security agencies, makes available guidance on how to create safer places and buildings that are less vulnerable to terrorist attack[9].

The existing guidance used for the structural assessment of LPS dwelling blocks for accidental loading is the MHLG Circulants 62/68[10] and 71/68[11], which were produced shortly after the partial collapse of Ronan Point in 1968. It is necessary to update this guidance not only to account for subsequent BRE work involving full-scale load testing in three LPS dwelling blocks and the general development of assessment methodologies, but also to make the guidance consistent with the approach to accidental actions employed in the structural Eurocodes and the general philosophical approach employed in contemporary risk management.

It is also important to recognise that overall, the historic structural performance of LPS dwelling blocks has been satisfactory, with most of the blocks which still exist having been in service for over 40 years. In instances where internal (non-piped) gas explosions have occurred in LPS dwelling blocks, structural damage has been limited in extent, and no occupied UK LPS dwelling blocks have experienced progressive collapse or a disproportionate degree of damage in-service as a result of a gaseous explosion or a vehicle impact since the partial collapse of Ronan Point in 1968.

However, the potential vulnerability of this form of construction has been highlighted by the recent progressive collapse of parts of a number of LPS dwelling blocks during demolition[12]. The LPS dwelling blocks concerned were pre-Ronan Point Fram Russell LPS. Limited investigation of one of the collapses by BRE suggested that the LPS dwelling blocks concerned could have been constructed to a relatively poor standard. Particular points were the quality of the tying at the floor slab/cross wall panel junctions, where there was a significant lack of connection (mechanical tying) between floor panels and the cross walls forming a bay, and the lack of end bearing to the floor slabs. BRE is aware that similar LPS dwelling blocks located in another part of the UK have been successfully deconstructed without incident. However, these collapses do highlight the need to be cautious when making a structural assessment, and the critical need for adequate invasive investigation of the nature and quality of construction achieved in individual LPS dwelling blocks before embarking upon extensive structural calculations in the structural assessment process.

Similarly, progressive collapse could also occur should a sufficiently large explosion or devastating incident occur which created forces which exceed the collapse resistance of the LPS dwelling block. In-service deterioration could adversely affect collapse resistance.

However, full-scale structural load tests by BRE to the ultimate load condition in three LPS dwelling blocks have demonstrated that this form of construction is stronger than the traditional ‘simplified’ structural calculations used to assess their response to accidental loading could demonstrate. Thus the LPS dwelling blocks tested by BRE have proved able to resist the forces due to (severe) internal gas explosions, thereby avoiding the initiation of element and structure failure which might have lead to progressive collapse or a disproportionate degree of damage. The condition of these three LPS dwelling blocks was judged to be ‘reasonable’ following detailed invasive forensic investigations of their construction.

These investigations revealed that the three LPS dwelling blocks did contain a variety of construction ‘faults’, typical of those encountered with this form of construction. Thus the three LPS dwelling blocks tested were considered to be reasonably representative of those found in the overall population of LPS dwelling blocks.

In addition, assessments undertaken following the existing guidance[4,5] have sometimes produced unexpected and inconsistent results. For example, some engineering consultants chose to ignore certain facets of structural behaviour which other consultants would take into account and, if incorporated into the assessment process, would in many instances have been able to demonstrate adequate reserves of strength against the specified accidental loading.

This work takes advantage of the outcomes of previous DTI PII funded research project involving full-scale structural load tests to the ultimate load condition in three LPS dwelling blocks.

It is estimated that there are in excess of 700 individual high rise LPS dwelling blocks (circa 50,000 individual dwellings), plus over 1000 low- and medium-rise LPS dwelling blocks in the UK.

Whilst this Handbook is essentially concerned with LPS dwelling blocks, the outcomes and the assessment methodologies described may be applicable to other forms of LPS buildings.

It should be noted that this Handbook has been written primarily from the perspective of The Building Regulations 2010[7] for England and Wales. However, it should be recognised that Scotland and Northern Ireland are governed by separate legislation. Whilst the general objectives and requirements are similar, there are subtle differences. In spite of this the principles involved will be generally applicable.

1.2 AIMS AND OBJECTIVES OF THE CURRENT PROJECT

The overall aim of this project has been to produce new guidance for the structural assessment of existing LPS dwelling blocks, focusing primarily upon their resistance to accidental loading associated with gaseous explosions (deflagrations) occurring within buildings –
1.3 RELATED BRE RESEARCH STUDIES

This Handbook draws on the final output report of a DTI part-funded Partners in Innovation research project. The aim of that research project was to improve the management of ageing assets by the use of advanced techniques to undertake structural assessment of existing multi-storey LPS dwelling blocks for accidental loading associated with what have been termed non-piped gas explosions.

This Handbook draws on work undertaken in several research projects. The most recent work involved full-scale structural load tests up to failure within a Bison LPS dwelling block situated in Liverpool, which was carried out under the (previous) PII scheme via a project entitled ‘Improving the management of ageing assets by advanced techniques for assessing existing multi-storey LPS blocks’. Reference is also made to the results of an earlier research project. The most recent work involved full-scale structural load tests up to failure within a Bison LPS dwelling block situated in Sandwell (a Bison LPS dwelling block) and Leeds (a Reema Conclad LPS dwelling block).

The final outputs from the previous DTI project were:

- An outline historical summary of key events that have occurred within the UK with respect to large panel structures (LPS) including the partial collapse of Ronan Point in 1968, together with the changes that have taken place in the technical requirements with respect to LPS dwelling blocks following the Ronan Point Inquiry.
- An overview of the risk environment that LPS dwelling blocks may be subjected to and a brief summary of the mechanisms associated with piped and non-piped gas explosions.
- A summary of the results of a series of a limited number of gas explosion tests and full-scale static tests undertaken on joint ‘mock-ups’ and, more recently, complete buildings.

The work highlighted a number of inconsistencies in assumptions and approaches used in current structural assessments which previously have resulted in widely differing conclusions about the suitability of existing LPS dwelling blocks for continued service and in the associated management strategies adopted. This earlier work also confirmed that there was a need to incorporate the findings of the DTI project and that of related engineering assessments and studies of LPS dwelling blocks into current guidance documents covering the assessment of LPS dwelling blocks under accidental loading.

The programme of work described in this Handbook was undertaken to resolve these issues.

1.4 PREVIOUS BRE WORK ON LPS DWELLING BLOCKS

In the mid-1980s BRE undertook a programme of investigations and other work relating to LPS dwelling block systems. This resulted in a series of BRE reports (see bibliography) of which BRE Report 107: The structural adequacy and durability of large panel system dwellings is most pertinent to the current considerations. This reported upon site investigations of construction carried out upon examples of eleven LPS dwelling block systems (in Part 1) and provides general guidance on assessment of LPS dwelling blocks (in Part 2). These reports complement earlier work concerning Taylor Woodrow-Anglian (TWA) LPS dwelling blocks of which Ronan Point was an example.

BRE Report 107: Part 1 - Investigations of construction describes the findings of visual inspections and the physical opening-up of joints at some 70 locations in the various blocks examined; together with some 400 determinations of carbonation depths and chloride contents. Additional information was obtained from investigative reports produced by consultants. Useful details provided concern:

- Reinforcement provision and condition within joints between precast concrete components.
- Reinforcement provision and condition within precast components and in-situ concrete joints.
- The nature and condition of the concrete and dry-pack mortar within in-situ joints.
- The security of parapet panels.

BRE Report 107: Part 2 - Guidance on appraisal provides recommendations for sampling and inspection of joints and structural connections, as well as structural assessment for normal and accidental loads.

---

1 Refer to the requirements described in section 2.
2 A non-piped gas explosion refers to an accidental deflagration which arises from the ignition of a build-up of flammable gas in a building or other enclosed space due to leakage from a source other than a piped gas supply. This implies that the build-up of flammable gas which can occur in a poorly ventilated enclosure is limited and is unlikely to result in a deflagration more violent than that associated with a ‘severe’ explosion. These matters are discussed in section 5.
2 REQUIREMENTS FOR STRUCTURAL ASSESSMENT OF LPS DWELLING BLOCKS FOR ACCIDENTAL LOADING

A large panel system (LPS) built dwelling block exceeding four storeys in height, (ie five storeys and higher) will be considered to satisfy Requirement A3 of Approved Document A9 if it meets one of the following:

**LPS Criterion 1:** There is adequate provision of horizontal and vertical ties to comply with the current requirements for Class 2B buildings as set down in the codes and standards quoted in Approved Document A – Structure10 as meeting the requirement set down in The Building Regulations11.

**LPS Criterion 2:** An adequate collapse resistance can be demonstrated for the foreseeable accidental loads and actions as defined below.

**LPS Criterion 3:** Alternative paths of support can be mobilised to carry the load, assuming the removal of a critical section of the load bearing wall in the manner defined for Class 2B buildings in Approved Document A – Structure12 or alternatively assuming the removal of adjacent floor slabs (taking the floor slabs bearing on one side of the wall at a time) providing lateral stability to the critical section of the load bearing wall being considered.

Collapse resistance of LPS dwelling block structural elements and the associated load transmitting joints between the structural elements should be evaluated for the forces associated with accidental overpressure values of 17 kN/m² or 34 kN/m² for the circumstances as defined below6.

A. An LPS dwelling block with a piped gas supply within or to any part of the building: an assessment overpressure of 34 kN/m² should be used generally throughout the building. The overpressure should be applied simultaneously to all surfaces of the single room/bounding enclosure within which the explosion (deflagration) is considered to occur. Alternatively, the approach used for the design of ‘key elements’ in new buildings may be adopted8.

B. An LPS dwelling block with a basement: an assessment overpressure of 34 kN/m² should be used in the basement and in any other zone where an explosive mixture of gas might accumulate (potentially from an external source). The overpressure should be applied simultaneously to all surfaces of the single room/bounding enclosure within which the explosion (deflagration) is considered to occur. Alternatively, the approach used for the design of ‘key elements’ in new buildings may be adopted8.

C. An LPS dwelling block without a basement and without a piped gas supply to any part of the building: an assessment overpressure of 17 kN/m² should be used. The overpressure should be applied

---

1 A large panel system (LPS) dwelling blocks typically comprise large precast reinforced concrete floor and roof components spanning onto large storey-height structural precast concrete wall panels. The precast storey-height and often room-width wall and floor panels form the structural system for the building. Vertical loads are carried to the ground through the structural wall panels, which for pre-1968 LPS dwelling blocks commonly contain very little steel reinforcement and are treated as being plain (unreinforced) concrete for the purposes of structural design or assessment. Stability against lateral loads is also provided by the structural wall panels, with those orientated across the short dimension of the building being referred to as cross-walls, or flank walls if they are the exterior walls located at the ends of the building. Some structural walls are orientated along the long dimension of the building, providing lateral stability in that direction. Such walls are often referred to as spine-walls.

LPS dwelling blocks are basically gravity structures, in the same way that traditional masonry constructed buildings are.

A key feature of LPS dwelling blocks is the jointing between the precast concrete panels, particularly the joints between the reinforced concrete floor panels and the plain concrete structural wall panels in pre-1968 LPS dwelling blocks, and the degree of overall structural continuity which is thereby achieved. LPS dwelling blocks with limited continuity reinforcement may be prone to progressive collapse under accidental loads and actions which exceed the collapse resistance of the block. The absence of effective vertical ties is a problem in the majority of existing pre-1968 LPS dwelling blocks, although the provision of vertical ties varies depending upon the (proprietary) system of construction used.

In determining the number of storeys in an LPS dwelling block, basement and podium storeys constructed utilising integral in-situ concrete construction (not formed using precast concrete components) may be excluded if such storeys fulfill the robustness requirements of Class 2B buildings.

The nominal length of a loadbearing reinforced concrete wall is defined as the distance between lateral supports subject to a maximum length not exceeding 2.25H, where H is the storey height in metres. In the case of an external masonry wall, the nominal length is the distance between vertical lateral supports.

6 However, for reliability based assessments it is necessary to recognise and take into account that the 17 kN/m² (non-piped gas situations) and 34 kN/m² (pipied gas situations) assessment criteria values may be exceeded – indeed it is necessary to estimate the probability distribution function of the complete range of possible overpressures for the respective situations.

9 See Clauses 5.1d and 5.3 of Approved Document A – Structure10.
simultaneously to all surfaces of the single room / bounding enclosure within which the explosion (deflagration) is considered to occur. Alternatively, the approach used for the design of ‘key elements’ in new buildings may be adopted. These requirements can be applied to the part of the dwelling block above the zone defined in Category B above.

Collapse resistance is defined as follows:

Collapse resistance is a measure of the ability of a structural system to resist the effects of defined accidental loads or actions occurring at or below a defined threshold. It provides a measure of the sensitivity of a structural system to specified accidental loads or actions and the ability of the structural system to survive them. Collapse resistance is a combined property of the structural system and the loading/action configuration applied to it. It is influenced by numerous factors including the strength, form and ductility of the structural system; which have a bearing upon the possible causes of initial local failure. In circumstances where the accidental loads or actions result in impulsive loads being applied to the structure, collapse resistance will be greatly influenced by the ability of the structural system concerned to absorbed energy. Collapse resistance issues may apply at both local and global structural system levels.

The structural assessment will need to take account of any potentially adverse implications of:

- Structural deterioration and/or deformation effects upon panels and the joints between panels and their effect on current and/or future structural performance.
- Previous and proposed structural repairs and related interventions undertaken to minimise the effects of existing or future deterioration of structural panels and in the joints between panels.

All LPS dwelling blocks should be managed using a systematic risk assessment methodology to guide through-life management activities and associated structure related actions, with the goal of:

- eliminating hazards where practicable, and
- reducing hazards and controlling risks to the structure of these buildings as far as is practicable,

consistent with ALARP/SFARP principles, taking account that risks associated with structural damage and collapse are generally at very low levels/acceptable levels.

The risk assessment methodology should consider other potential accidental actions such as fire effects, the possibility of an external explosion (deflagration) and impact by some form of vehicle.

Whilst these activities seek to evaluate and reduce the probability of the occurrence of the foreseeable accidental loads and actions causing structurally significant damage to an LPS dwelling block, they do not remove the risk of progressive collapse in an LPS dwelling block should a sufficiently large explosion or devastating incident occur creating forces which exceed the collapse resistance of the structure.

2.1 ALTERNATIVE APPROACH

Clause 5.4 of Approved Document A – Structure[9] introduces the concept of an alternative approach to satisfying the requirements of the Building Regulations for the design and construction of new structures. In the context of the structural assessment of existing LPS dwelling blocks, it would seem reasonable that such an approach might also be adopted. It could permit the adoption of a probabilistic-based assessment methodology for estimating internal gas explosion loads and the associated structural response in order to check whether an acceptably low probability of failure was likely to be achieved. Undertaking such an evaluation would require specialist expertise and experience, together with appropriate data.

It is perhaps interesting to note that Clause 2 of the Appendix to MHLG Circular 62/68[4] indicates that ‘where residual risks are lessened by the control of the incidence of an explosion in magnitude or frequency, a corresponding reduction may be made in the pressure’. The pressure referred to here was the static overpressure of 5 lb/in² (34 kN/m²) defined for the design of new LPS dwelling blocks and for their assessment. It is suggested that this might be interpreted as being consistent with the above approach to structural assessment of existing LPS dwelling blocks.

The reliability/probabilistic approach would require that the overpressure loading was defined by a probability density function, as would the strength/response function (based upon a probability density function for material/member strength). This information would be used to estimate the probability of failure. This would then be judged as being satisfactory or not depending upon the resulting level of the probability of structural failure (it would be necessary to choose an appropriate structural reliability β-value consistent with the design code value). Different structural reliability β-values could perhaps be adopted for different circumstances (see section B7, Appendix B). Annexes B and C of BS EN 1990[11] are relevant to such considerations.
3 BUILDING REGULATION AND LPS ASSESSMENT REQUIREMENTS, PROGRESSIVE COLLAPSE AND RELATED MATTERS

3.1 INTRODUCTION
This section reviews the origin of the special structural issues associated with LPS dwelling blocks and the related UK guidance for their structural assessment dating from the progressive collapse of part of Ronan Point in 1968 and more recent developments. The evolution of UK Building Regulations and contemporary requirements for reducing the sensitivity of new buildings to disproportionate damage, as given in Approved Document A – Structure,[9] are described. The phenomena of progressive collapse and disproportionate damage are discussed, along with the concepts of robustness and collapse resistance. These matters are put into the wider perspective of structural safety and risk assessment considerations, with linkages to building regulation and related requirements being discussed in the context of contemporary expectations for the safety of new buildings.

A more detailed review of the development of UK technical requirements for the structural assessment of LPS dwelling blocks and the evolution of contemporary requirements for reducing the sensitivity of new buildings to disproportionate damage is presented in Appendix A. Risk issues are examined in Appendix C.

3.2 THE STARTING POINT – THE PARTIAL COLLAPSE OF RONAN POINT
At approximately 5.45am on Thursday 16 May 1968, Ronan Point, a 23-storey large panel system (LPS) block[10] built by Taylor Woodrow-Anglian Ltd, suffered a vertical progressive collapse of its south-east corner (Figure 5).

Four people died immediately as a result of the collapse, suffering from multiple-crushing injuries. At the time of the explosion these people had been situated in flats located in the south-east corner of the block at the seventeenth and twenty-second floors levels. A further seventeen people were injured and were taken to hospital. Fortunately fourteen of them were only slightly injured and they were soon discharged after treatment. Of the people taken to hospital, one subsequently died a fortnight later, but it is reported[12] that the death of the 82 year old lady was not directly related to the accident. It is understood that the other two people hospitalised made a satisfactory recovery.

Figure 5: Damage to Ronan Point (1968)

The south-east corner of Ronan Point contained the lounges/living rooms of the single-bedroom flats situated in this part of the building. Fortunately, at 5.45am most people who lived in these flats were in their bedrooms, which were situated next to the lounges/living rooms. If the explosion had occurred somewhat later, it seems quite likely that many more people would have been in their lounges/living rooms, all of which were swept away in the collapse.

A Tribunal was subsequently established under Section 318 of The Public Health Act 1936 and Section 290 of The Local Government Act 1933, to investigate the cause or causes of the collapse, to consider the implications of the findings and to make appropriate recommendations. The Tribunal’s report[12] was published towards the end of 1968.

Tests and detailed investigations were carried out upon certain elements of the structure and the collapse debris to resolve a number of questions relating to aspects

---

[9] Ronan Point was constructed using the Larsen Nielsen system, which was conceived in Denmark in 1948.
such as the cause of the explosion, likely strength of key components and associated joints, explosion pressures, etc. One of the key questions that the Tribunal needed to answer was 'what lateral pressure was required to displace a flank wall panel'. The investigators estimated that an overpressure of approximately 5 lb/in² (≈ 34 kN/m²) would have been required to produce sliding failure at the base of the flank wall. This and related matters are discussed in section 7 below.

The Tribunal made a number of recommendations affecting system-built (ie LPS) blocks of over six storeys (ie seven storeys and above) in height. These included:

- Measures for the strengthening of Ronan Point.
- A requirement to appraise and, if needed, to strengthen other existing LPS dwelling blocks.
- Factors to be taken into account when designing new LPS dwelling blocks.

### 3.3 MINISTRY OF HOUSING AND LOCAL GOVERNMENT CIRCULARS 62/68 AND 71/68

Following on from the recommendations of the Ronan Point Tribunal, in late 1968 the then Ministry of Housing and Local Government (MHLG) issued advice to Local Authorities via MHLG Circulars 62/68 and 71/68. The advice given to building owners at that time was to appraise all their LPS dwelling blocks of seven storeys and above for their susceptibility to progressive collapse. Owners were required to consider whether strengthening was necessary.

MHLG Circulars 62/68 and 71/68 were both titled *Flats constructed with precast concrete panels. Appraisal and strengthening of existing blocks: Design of new blocks.* MHLG Circular 62/68 was issued in November 1968 and provided advice to local authorities on matters raised in the Ronan Point Tribunal report. In particular MHLG Circular 62/68 drew attention to the recommendation that all blocks over six storeys in height should be appraised by a structural engineer who should consider whether they were susceptible to progressive collapse; whether they were designed to resist adequately the maximum wind loadings which they may experience; and their potential behaviour in the event of fire.

MHLG Circular 62/68 identified two basic methods of avoiding progressive collapse, namely:

**Method A.** By providing alternative paths of support to carry the load, assuming the removal of a critical section of the loadbearing walls.

**Method B.** By providing a form of construction of such stiffness and continuity so as to ensure the stability of the building against forces liable to damage the load supporting members (ie. to be sufficiently strong to resist the accidental loads).

In MHLG Circular 62/68 it was also indicated that for these purposes, the forces should be assumed as being equivalent to a standard static overpressure of 5 lb/in² (34.5 kN/m²). MHLG Circular 62/68 also advised that where residual risks were lessened by the control of the incidence of an explosion in magnitude or frequency, a corresponding reduction might be made in the magnitude of the overpressure employed in the structural assessment or design.

MHLG Circular 62/68 defined the critical section as follows:

> 'In load-bearing walls the critical section is represented by the distance between substantial walls at right angles to it or between a substantial wall and a return end, or the length of one precast wall panel whichever is the greater.'

The Institution of Structural Engineers in its Report RP68/02 dated 1968 published 'Notes for guidance which may assist in the interpretation of Appendix 1 to MHLG Circular 62/68'.

MHLG Circular 62/68 indicates that a tensile resistance of at least 44 kN/m width should be provided continuously across the length and breadth of floor and roof slabs. It also indicated that steel connections should be provided between adjacent floor or roof slabs over internal supports and between the slabs and the supporting external walls, spaced at no greater distance than 600 mm. It was stated that in the design of these connections, stresses appropriate to mild steel should be used. It was advised that connectors transverse to the span in only one slab may be concentrated in the joints along the line of the supporting walls, and may be in high tensile or mild steel. It was indicated that in suitable cases this steel may provide both (a) horizontal continuity between adjacent wall panels and (b) the necessary reinforcement to develop beam or cantilever action.

MHLG Circular 62/68 indicated that the tensile resistance may be developed between panels (a) by welding together projecting reinforcement or (b) by closely overlapping projecting loop bars locked together by longitudinal dowel reinforcement. MHLG Circular 62/68 suggested that lapped bars within in-situ concrete jointing should not in general be used as a means of achieving continuity in tension or compression. However, lapping could be used for connections transverse to the span and for beam and cantilever action in walls. Where continuity was developed by bond in the in-situ concrete, the width of joint should be sufficient to ensure adequate compaction of the concrete and be not less than 75 mm.

The Institution of Structural Engineers (ISE) produced Report RP68/02 dated 1968 entitled *Notes for guidance which may assist in the interpretation of Appendix 1 to MHLG Circular 62/68* which provided further clarification of the requirements given in MHLG Circular 62/68 and ways in which these might be satisfied.

Notable points in this ISE document include the following observations and recommendations:

- Where piped gas was absent, the overpressure criterion for LPS dwelling block assessment could be halved to 2.5 lb/in² (17 kN/m²).
- Structural design and assessment Method B permitted structural engineers to develop an alternative to Method A design/assessment procedure supported by tests or other practical experience as may be necessary.
- An assumed tensile strength of structural concrete in flexure not exceeding 2.0 N/mm².

---
MHLG Circular 71/68[3] was issued in December 1968 and was official endorsement of the advice given by the Institution of Structural Engineers in its report RP68/02[13]. The document was sent to all local authorities that owned LPS buildings and their professional advisors.

**Note:** The concept defined in Method B, above, is of particular note in regard to the full-scale structural overpressure load tests and ancillary studies undertaken in recent years by BRE, which are described in Appendices E to H. BRE was able to demonstrate by full-scale static overpressure testing that the form of construction of the LPS dwelling blocks concerned was capable of resisting the accidental forces liable to damage the load supporting members. Thus, the particular examples of LPS construction tested were shown to be sufficiently strong to resist the accidental loads which might be expected to be imposed by a non-piped gas explosion.

However, it should also be recognised that from a probabilistic point of view this outcome is to be expected. Failure is only likely to occur if an internal gas explosion occurs with a chance magnitude somewhat greater than usual in a dwelling that by chance has much lower than average strength properties. Thus the full scale structural tests, although re-assuring, cannot therefore be taken to guarantee that all LPS dwelling blocks will perform satisfactorily in such circumstances therefore are ‘safe’.

### 3.4 CHANGES TO THE BUILDING REGULATIONS AND LPS ASSESSMENT REQUIREMENTS WITH TIME

In 1970 provisions to resist progressive collapse based on the 5 lb/in² (≈ 34 kN/m²) accidental loading criterion were introduced in Section D17 of the Building Regulations[14] for blocks of five storeys and above. It was at this time that the building height threshold for considering progressive collapse and accidental loading issues in new construction changed by two storeys. That is from being buildings of seven storeys or more high, to being buildings of five storeys or more high.

It is understood that this change was not reflected in the available guidance for the structural assessment of LPS dwelling blocks. MHLG Circulars 62/68[4] and 71/68[5], published in 1968, remained as the contemporary guidance defining the requirements for the structural assessment of LPS dwelling blocks.

As a result it appears that there could have been some confusion as to whether structural assessments for accidental loads were required for LPS dwelling blocks over four storeys tall or for blocks over six storeys tall.

BRE Report 107: Part 2[2], published in 1987, gave further guidance on the assessment of structural adequacy and durability of LPS dwelling blocks. This BRE report defined the requirements for a structural assessment of an LPS dwelling block, offering non-mandatory guidance on the assessment of structural adequacy, sampling and inspection of structural connections, assessment of findings and procedures for evaluation of the future durability of the structural components and connections. Consideration was given to the accidental loads which might be applied to an LPS dwelling block, as noted previously. Amongst the possible forms of progressive collapse and the significance of connections/tying reinforcement between precast structural components. BRE Report 107: Part 2[2] confirmed that structural assessments for accidental loads were required for all LPS dwelling blocks over four storeys tall. BRE Report 107: Part 2[2] also indicated that assessment overpressure criterion could be halved (17 kN/m²) for LPS dwelling blocks where piped gas was absent.

The 1996 second edition of the Institution of Structural Engineers report Appraisal of existing structures[15] repeated the advice that the assessment criterion for key elements could be halved (17 kN/m²) for LPS dwelling blocks where piped gas was absent and, importantly, that it was certain that the building would remain without a piped gas supply in the future. This document also made reference to the requirement to appraise the robustness of buildings over four storeys tall (ie five storeys and higher).

Thus there was uncertainty as to whether structural assessments were carried out for accidental loads for all five and six storey LPS dwelling blocks during the early 1970s, or subsequently. BRE understands that some five and six storey LPS dwelling blocks with a piped gas supply were not assessed for accidental loads until the early 2000s.

Figure A1 (see Appendix A) presents the changes that have taken place since 1968 in relation to the requirements for structural assessment relative to the height of LPS dwelling blocks in diagrammatic form. For completeness, Figure A1 also shows the contemporary requirements (interpreted in the context of their applicable to LPS dwelling blocks) as defined in Approved Document A – Structure[9].

### 3.5 OTHER GUIDANCE ON ASSESSMENT OF LPS DWELLING BLOCKS

In September 1984 The Institution of Structural Engineers, Scottish Branch, published a guidance note[16] in response to growing concerns over the potential risk of detachment of the outer leaf to sandwich panels on LPS dwelling blocks. This report, whilst not dealing with the issue of structural robustness, documented a national scare in which the outer leaf to a few LPS cladding panels had become detached. It is understood that there were no fatalities as a result of these incidences. In April 1989, the Institution of Structural Engineers published a guidance note on the security of cladding on large panel construction[17], which formalised the above-mentioned earlier guidance for LPS dwelling blocks.

In October 1984 the Ministry for Housing and Construction announced in a press release that the BRE was to undertake a programme of investigations of LPS dwelling blocks. This programme of work covered a range of aspects with respect to this form of construction including structural adequacy and durability, habitability, overcladding and weathertightness.

BRE subsequently published a number of reports covering each of the areas listed above (see section 16). Of particular relevance to this discussion was BRE Report 107: Part 2[2] on the structural adequacy and durability of LPS dwelling blocks, as noted previously.
other recommendations made was that a full structural assessment of a complete LPS dwelling block should be undertaken every 20 years; with supporting visual inspections of the external envelope of the building at intervals of about 5 years, together with intrusive investigations every 10 years to check upon the condition of the reinforcement within in-situ joints which might experience rain penetration (eg at flank wall and roof locations).

It is also known that some consultants have undertaken assessments on the basis of contemporary structural design codes. In most instances these are not appropriate to the assessment of existing structures – for reasons, see the discussion relating to Table B6 (Appendix B) and the 2010 third edition of the Institution of Structural Engineers report Appraisal of existing structures\(^{[15]}\). The outcomes of this type of assessment when undertaken by different engineers/consulting practices apparently came to distinctly different interpretations. In some instances different engineers/consulting practices had been reasonably consistent. Typically few problems are identified in respect of the requirements for normal loads (eg gravity and wind, including self-weight, imposed and snow loads).

However, guidance on the assessment for accidental loading as provided by MHLG Circulars 62/68 and 71/68\(^{[4,5]}\) has perhaps been more open to different loading as provided by MHLG Circulars 62/68 and 71/68\(^{[4,5]}\) including self-weight, imposed and snow loads). The likelihood (ie. probability) of occurrence of various forms of accidental action which might affect an LPS dwelling block are examined in section 5 and Appendix C of this Handbook. The hazards considered are as follows:

- Accidental internal and external gas explosions (deflagrations).
- Accidental impacts by road vehicles and other types of vehicle, which, in section 5 and Appendix C of this document, focuses upon impacts by aircraft.

Section 5 of this Handbook also examines the potential consequences of the various forms of accidental action considered in order to establish what response, if any, is appropriate to the identified hazard.

### 3.6.2 Progressive collapse

BRE Report 107: Part 2\(^{[2]}\) on the structural adequacy and durability of LPS dwellings discusses potential modes of progressive collapse in LPS dwelling blocks. Paraphrasing, this document suggests that in this context progressive collapse should be taken to be ‘a collapse in which the failure front moves progressively away from the initial trigger point of local damage to envelope portions of the structure significantly larger than the part directly damaged by the initial incident’. BRE Report 107: Part 2\(^{[2]}\) goes on to observe that the direction and extent of a progressive collapse will depend upon the structural form involved; that is the disposition of structural materials in the building, the pattern of relatively weak and strong zones/joints and the facility with which the potential energy of the building can be released allowing the failure front to progress.

BRE Report 107: Part 2\(^{[2]}\) notes that in (high-rise) LPS dwelling blocks the number of physical mechanisms of collapse is limited, with the critical consideration being the progression of the failure front. BRE Report 107: Part 2\(^{[2]}\) differentiates mechanisms which could initiate progressive collapse into those which could be generated by (a) a trigger at low level in an LPS dwelling block and (b) those which could be triggered near the top of an LPS dwelling block (see Figure 6).

- **Low-level trigger-induced failure mechanisms.** There are various forms of progressive failure which might be initiated from a low-level trigger site, including those identified in Figure 7 for the section of an LPS dwelling block adjacent the flank wall.

Figure 7 illustrates three possible failure mechanisms which hypothetically would allow the end bay of an LPS dwelling block to experience vertical progressive collapse. The mechanisms illustrated are; (a) shear
failure through door/window openings, (b) shear failure through vertical joints between wall panels and floor slabs, and (c) where each wall panel and associated floor slab above the trigger point could act independently and fall off sequentially, with the failure front propagating up the building from somewhere near the bottom rather like a ‘zip fastener being undone’.

Failure mode (c) might occur in circumstances where the wall and floor joints involved were weak or lacked continuity. The floors might then fold down or drop off, as happened in the Ronan Point failure (see Figure 5).

• **High-level trigger-induced failure mechanisms.**

Figure 10 illustrates two forms of progressive failure which might be initiated from a high-level trigger site. The ‘pancake’ type shear floor failure shown in Figure 10a could notionally occur at any location in an LPS dwelling block. This form of failure might notionally be caused by debris loading impacting on the floor below the trigger point, with the failure front propagating rapidly down the building. In this situation the walls might not be affected and could thus remain in place, but would be expected to be in a
de-stabilised condition. Conversely, the mechanism of progressive failure shown in Figure 10b would occur in the section of an LPS dwelling block adjacent the flank wall. This mechanism might occur when the flank wall had strong vertical connections between the individual flank wall panels, but relatively weak connections to the adjacent floor slabs. It is hypothetically possible that a sufficiently large internal gas explosion might result in the flank wall peeling off of the building rather like a banana skin being removed. The failure front would then progress down the building. It might be possible for the floor slabs to either span diagonally in some way or perhaps they would collapse progressively, falling off the building and following the flank wall to the ground.

**Mid-height trigger-induced failure mechanisms.** Of course, there is also the possibility that there could be mid-height trigger points (see Figures 8 and 9), which result in progressive collapse of part of an LPS dwelling block. These might trigger collapse mechanisms in which the failure front propagated both upward and downward in the LPS dwelling block. Generally the mid-height type triggered failure is considered to be a less likely cause than either a high-level trigger-induced failure mechanism (which is probably most likely for a Ronan Point type LPS dwelling block assuming inadequate connectivity between the loadbearing panels) or a low-level trigger-induced failure mechanism, where the greater vertical (compressive) loads in the loadbearing wall elements helps them resist accidental lateral loads.

It should be borne in mind that the trigger for the local damage/initial failure will generally be expected to be some form of external (to the structural system) energy source. However, the secondary failure mechanisms that are associated with progressive collapse are driven by the conversion of the potential energy, which is contained in the structure, into kinetic energy. The amount of potential/kinetic energy mobilised in a progressive collapse is likely to be very large.

BRE Report 107: Part 2 notes that LPS dwelling blocks, along with other buildings, have a significant degree of three-dimensional structural continuity. It is generally true that the higher up in a building that local damage occurs, the fewer the number of potential alternative paths there are to redistribute loads. Accordingly, damage high up in a building close to a corner is likely to be most critical in terms of the possible number of alternative load paths which might be mobilised. However, because the mass of the structure above this level will be limited, the magnitude of the potential/kinetic energy which will be mobilised during a local failure will be limited. Damage a few storeys from the top is likely to be more critical because the magnitude of the potential/kinetic energy which could be mobilised during a local failure will be much larger.

Starossek also recognises that various forms of progressive collapse can occur which may or may not result in disproportionate damage of part or even all of a building or a structural system. For example, the various forms of progressive collapse identified by Starossek include the following:

- **The pancake form of progressive collapse in a structural system.** In buildings this is where a vertical progressive collapse in the affected building results in the floors ending up stacked upon each other after the destruction of the vertical load bearing wall and column elements. The extent of the failure can involve part or all of a building, depending upon the nature of the triggering incident as well as upon the type and strength of the connections between structural elements in the affected zone.

The ‘pancake form of progressive collapse’ is like a brittle parallel system in which there is progressive failure following the failure of an initial structural element. In this situation the total vertical load is transferred to the vertical load bearing elements of the floor below upon the failure of the first floor, and as more floors fail it becomes increasingly likely that each floor below will also fail in turn.

This form of progressive collapse can also propagate in the horizontal direction when the series of structural elements act horizontally, such as is the case with a multi-span viaduct arch structure. In these circumstances the horizontal component of the arch thrust from one arch is balanced by that from the adjacent arches. Collapse of one arch eliminates the horizontal component of the stabilising arch thrust applied to the adjacent arches. Depending upon the size and strength of the piers between adjacent arches, the unbalanced horizontal thrust may destabilise the structural system and result in a progressive collapse which propagates in the horizontal direction. This type of failure sequence may be referred to as a ‘domino form of progressive collapse’.

The ‘pancake form of collapse’ is relevant to the current considerations about LPS dwelling blocks and was one of the modes of failure purposely initiated in the laboratory experiments undertaken by BRE in the 1970s on quarter-scale LPS dwelling block structural test models.

- **The progressive failure of the load bearing elements in a parallel structural system.** This is commonly referred to as the ‘zipper form of progressive collapse’, where the sequence of the progressive collapse propagates in the horizontal direction from one vertical load bearing element to the next. In many instances the failure will propagate until the collapse failure front encounters some form of discontinuity in the structure or the entire structural system is affected. The extent of the affected zone will depend upon the type and strength of the connections between the structural elements. The collapse of the Tacoma Narrows Bridge in Washington State, USA in the 1940s is cited as an example of this type of failure. More commonly, the collapse of heavy plaster and other forms of ceilings within buildings supported by hanger systems is another example of this type of failure. In the context of LPS dwelling blocks, the partial progressive collapse in the south-east corner of Ronan Point exhibited this type of failure characteristic. In the context of LPS dwelling blocks, the partial progressive collapse in the south-east corner of Ronan Point exhibited this type of failure characteristic.
Figure 8: Schematic showing a possible progressive collapse failure mode of the flank wall involving wall-wall and floor-wall joints for low-to medium-level trigger sites.

Figure 9: Schematic showing a possible progressive collapse failure mode of the flank wall and floor slab involving wall-wall, floor-floor (and potentially wall-wall) joints for low-to medium-level trigger sites.

Figure 10: Schematic showing some possible progressive collapse failure modes in a typical LPS dwelling block for a high-level trigger site.
Starossek[20] also recognises that progressive collapse can be controlled either by:

- ensuring that the structure has the ability to mobilise alternative load paths around a zone of local damage, or by
- introducing discontinuities into the structure (i.e., joints) which prevent the collapse failure front propagating beyond a certain point.

The latter technique is referred to as segmentation of the structure. This approach seeks to divide the structure into a series of zones which isolate a collapsing section from the remainder of the structure. Segmentation can be introduced either horizontally, to limit the horizontal propagation of collapse or damage, or vertically, to limit the vertical propagation of collapse or damage. Clearly, the related design and potential load transfer considerations have to be taken into account for such an approach to work successfully.

### 3.6.3 Robustness

The term ‘robustness’ is referred to but not currently defined in Approved Document A – Structure[21]. However, in the technical literature, there are a number of existing definitions of this characteristic of a structure or building. Some of these definitions are reproduced below by way of information, before a working definition is proposed for use in this Handbook with respect to the assessment of LPS dwelling blocks.

**BS EN 1991-1-7**

The definition given in Clause 1.5.14 of BS EN 1991-1-7: Eurocode 1 – Actions on structures: Part 1-7: General actions – Accidental actions[20] is reproduced below: *Robustness is the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause.*

**SCOSS (Standing Committee on Structural Safety)**

SCOSS suggests that robustness may be defined as: *An encompassing term used to describe a building’s ability to perform safely and efficiently under the anticipated conditions over its expected working life. These conditions may relate to loads, the effects of climate change or of the environment.*

**fib Bulletin 44**

The definition given in ‘Appendix A: Keywords’ of fib Bulletin 44, Concrete structure management – Guide to ownership & good practice[21], is reproduced below: *The ability of a structure subject to an accidental or exceptional loading to sustain local damage to some structural components without experiencing a disproportionate degree of overall distress or collapse. Robustness is an indication of the ability of a structural system to mobilise alternative load paths around an area of local damage. It is related to the strength and form of the structural system, particularly the degree of redundancy within the structural system.*

**Starossek definition and discussion of issues**

The following observations, presented in a paraphrased form, are given in a paper by Uwe Starossek entitled Disproportionate collapse: A pragmatic approach which is contained in the Institution of Civil Engineers Journal, Structures and Building, Vol 160, Issue SB6, December 2006[19]:

*That robustness is defined as insensitivity to local failure, where ‘insensitivity’ and ‘local failure’ are quantified by the design objectives (which are part of the design objectives).*

According to this definition, robustness is a property of the structure alone and independent of the cause and probability of initial failure. This is in contrast to a broader definition of robustness (as is given in, for instance in EN 1-1-7) which does include possible causes of initial failure. Such a broader definition is close to the term ‘collapse resistance’.

It is believed that clarity is served by distinguishing these two properties (which could be named differently if no consensus on a re-definition of the term robustness is reached).

It has been suggested that in addition to considerations of the degree and extent of damage, there should also be some restriction upon the amount of deformation or displacement of structural elements needed to mobilise alternative load paths. For example, these considerations could potentially involve placing limits upon the magnitude of the vertical deflections associated with mobilising catenary behaviour in framed structures or via the provision of internal structural ties.

It has also been proposed by some workers that robustness should be related to the particular design objectives / design criteria of the structure.

The important contribution that connections/fixings between structural elements make to the robustness of a structure should be recognised.

BRE propose that the following should be adopted as a working definition of robustness, with qualifying observations, in respect of LPS dwelling blocks:

**Definition:** Robustness is the ability of a building subject to accidental or exceptional loading or other action(s) to sustain damage or local failure without experiencing a disproportionate degree of overall distress or collapse.

**Qualifying observations:** In buildings, robustness is commonly taken to be the ability of the structural system to mobilise alternative load paths around an area of local damage, which is typically a function of the degree of redundancy within the structural system together with the strength and ductility of the elements and joints between them.

Whilst this approach is satisfactory when the extent of local damage is relatively small, perhaps involving one or two vertical load bearing elements, mobilising alternative load paths within the structural system tends to have the effect of increasing the overall extent of damage caused across the building, although typically the severity and consequences of that damage is reduced. It should be
recognised that the above scenario implies that there are differing probabilities of different degrees of damage occurring in various parts of the structure.

However, when the initial damage to the building could be more extensive than the local failure described above, the overall extent of collapse and damage may be better controlled by segmenting the building into zones by the introduction of joints or discontinuities which are intended to limit the propagation of collapse and damage across the building. Thus it would be intended that a collapse in one such zone should not propagate across the boundary joints or discontinuities into the adjacent zones of the building.

Thus robustness is related not only to the strength, form and ductility of the structural system, but may also be influenced by the division of a building into zones by joints or discontinuities. Robustness is a quality of the structural system alone and is independent of the cause of the damage and/or the probability of initial local failure.

However BS EN 1991-1-7: Eurocode 1 – Actions on structures: Part 1-7: General actions – Accidental actions[20] gives a broader definition which does include possible causes of the initial failure. The definition given in Clause 1.5.14 of BS EN 1991-1-7 is reproduced below:

‘Robustness is the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause.’

3.6.4 Collapse resistance
This term is not mentioned or defined in Approved Document A – Structure[9]. The term is, however, the subject of discussion in the technical literature. A definition of the term is reproduced below by way of information, before a working definition is proposed for use in this Handbook with respect to the assessment of LPS dwelling blocks.

Starossek definition and discussion of issues
The following observations, presented in a paraphrased form, are given in a paper by Uwe Starossek entitled Disproportionate collapse: A pragmatic approach which is contained in the Institution of Civil Engineers journal, Structures and Building, Vol 160, Issue SB6, December 2006[18].

That collapse resistance is defined as insensitivity to accidental circumstances, comprising low-probability events and unforeseeable incidents, which are quantified by the design objectives.

Collapse resistance is a property that is influenced by numerous conditions including both structural features and the possible causes of initial local failure. The structural system is of particular importance. It would intolerably limit the range of design choices, however, if only those structural systems were permitted that are clearly robust. Nor is such a limitation necessary because a non-robust structure, the system of which tends to promote collapse progression, can possibly be made collapse resistant by other measures such as a particularly safe design of key elements.

BRE propose that the following should be adopted as a working definition of collapse resistance, with qualifying observations, in respect of LPS dwelling blocks:

Definition: Collapse resistance is a measure of the ability of a structural system to resist the effects of specified accidental loads or actions occurring at or below a defined threshold.

Qualifying observations: Collapse resistance provides a measure of the sensitivity of a structural system to specified accidental loads or actions and the ability of the structural system to survive them. Collapse resistance is a combined property of the structural system and the loading/action configuration applied to it. It is influenced by numerous factors including the strength, form and ductility of the structural system; which have a bearing upon the possible causes of initial local failure. In circumstances where the accidental loads or actions result in impulsive loads being applied to the structure, collapse resistance will be greatly influenced by the ability of the structural system concerned to absorbed energy. Collapse resistance issues may apply at both local and global structural system levels.

The important contribution that connections/fixings between structural elements make to the collapse resistance of a structure should be recognised.

3.7 CONTEMPORARY BUILDING REGULATIONS AND RELATED GUIDANCE
Over the period since the Ronan Point incident there has been ongoing development in the requirements for the design of new buildings to reduce their sensitivity to disproportionate damage in the event of an accident. Some aspects of these developments are closely aligned with those discussed earlier in this section in respect of the requirements for the structural assessment of existing LPS dwelling blocks. However, it is important to consider some aspects of the concepts now being applied in respect of new buildings.

A key aspect in Building Regulations[7,9,14,22] over the intervening period is that they have all included a requirement that ‘the building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause’. This has been embodied as Requirement A3 in Approved Document A – Structure[9].

Contemporary guidance on the potential extent of acceptable damage is given under Requirement A3 of Approved Document A – Structure[9]. Table 11 in this document defines a series of classes based upon factors such the number of storeys in the building, floor areas and the potential number of occupants who might be at risk.

Further changes are anticipated in the Building Regulations with the introduction of the Structural Eurocodes, which since 2008 have been designated by DCLG[7] as an acceptable way of satisfying Approved Document A – Structure[9].
The structural Eurocodes formally replaced the UK national standards for structural design in April 2010. At that time the British Standards Institution (BSI) was obliged to withdraw conflicting UK national structural design standards, some of which are referenced in the Building Regulations Approved Documents, particularly Approved Document A – Structure. The British Standards withdrawn on 31 March 2010 remain available from BSI. However, BSI committees had previously stopped updated those standards, so in the medium and long term they are expected to become less suitable for aspects of structural design.

Guidance on the requirements for the design of new structures in accordance with the Structural Eurocodes is given in Section 2 of BS EN 1990: ‘Eurocode – Basis of structural design’[11].

Annex A of BS EN 1991-1-7: 2006[20] offers guidance on one of the two[13] ‘recommended strategies’ which ‘should provide a building with an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse’ – see Figure A3 (Appendix A). In the context of the structural assessment of LPS dwelling blocks, it is of interest to note that the concept of designing the structure to sustain the accidental action is given as one approach for addressing the accidental design situations.

Annex B of BS EN 1991-1-7: 2006[20] offers guidance on risk assessment describing procedures for qualitative and quantitative risk analysis, together with risk acceptance and mitigation measures. Annex B of BS EN 1991-1-7: 2006[20] indicates that the ALARP principle is to be used to guide risk acceptance and mitigation measures, with various approaches and criteria being outlined. Annex B indicates that risk acceptance levels should be specified and that they will usually be formulated on the basis of:

- Individual risk – with the risks to an individual being expressed as a fatal accident rate, an annual fatality probability or as the probability per unit time of a single fatality when involved in a specific activity.
- Socially acceptable risk to human life – often presented as an F–N curve, indicating a maximum yearly probability F of having an accident with more than N causalities.

Annex B of BS EN 1991-1-7: 2006[20] indicates that alternatively, concepts such as ‘value of a prevented fatality’ (VPF) or a ‘quality of life index’ might be used. These matters are discussed in detail in Appendix A3 of this document.

In the context of the structural assessment of LPS dwelling blocks, particularly pertinent considerations embodied in contemporary Building Regulations and related guidance are:

- The adoption of consequences classes.
- The need to ‘undertake a systematic risk assessment of the building taking into account all the normal hazards that may reasonably be foreseen, together with any abnormal hazards’.
- The use of risk assessment procedures for qualitative and quantitative risk analysis for both (a) risk of death to individuals and (b) socially acceptable risk to human life where there might be multiple fatalities in any one accident.

The recent publication by The Institution of Structural Engineers of guidance concerning the practical implementation of measures to ensure structural robustness and reduce the sensitivity to disproportionate collapse in buildings complements the existing literature on these topics for new construction[23].

---


[20] The two recommended strategies are defined in Figure 3.1 of BS EN 1991-1-7: 2006[20] entitled Strategies for accidental design situations. The two strategies given are:
1. Strategies based on identified accidental actions, eg explosions and impact.
2. Strategies based on limiting the extent of localised failure.
4 THROUGH-LIFE PERFORMANCE AND ASSESSMENT ISSUES

4.1 INTRODUCTION
This section considers some aspects of the through-life performance of buildings generally. It examines, at a conceptual level, issues associated with deterioration of the structure with time and the potential implications of this and other factors upon the through-life failure rate/reliability of the structure. The benefits of a proactive approach, as opposed to a reactive approach, to the through-life management of LPS dwelling blocks are outlined. Assumptions about the behaviour of LPS dwelling blocks under accidental loading associated with an internal gas explosion are described, as these form the basis for the LPS dwelling block structural assessment methodology developed by BRE, which is described in section 12 of this Handbook.

4.2 THROUGH-LIFE PERFORMANCE OF BUILDINGS
There are a number of phases in the development and use of a building. Typically the sequence of events through the life of a building is as follows:
- **Concept phase** – where the owner’s basic requirements and needs are established.
- **Design** – usually involving preliminary and detailed design phases.
- **Construction** – the process whereby the building is created.
- **Operation and use** – through life performance and maintenance of its functionality.
- **Disposal** – the process by which the building is demolished and removed.

Whilst most concrete buildings provide satisfactory performance over an acceptably long service life period, deterioration processes affect all building materials to varying degrees. As a result through-life care and management is needed to maximise the service life achieved by a structure.

Deterioration of buildings may arise for a number of reasons including:
- Poor design, specification, detailing or execution during construction.
- Poor planning or implementation of maintenance operations.
- Lack of funds for routine maintenance.
- Past underestimation of the role of proper and timely maintenance.
- Ageing processes or environmental aggressiveness and actions upon the building.
- Other factors, such as increased loading causing damage.
- Accidental events such as fires and explosions.

As the chemical reactions associated with deterioration processes typically require moisture, deterioration will usually progress at a much slower rate when little moisture is available.

Extending the required service life of a building significantly beyond that originally planned may also require intervention works to slow any ongoing deterioration processes. These requirements should perhaps not be thought of as a failure, but more in terms of a successful extension of the useful service life of the building.

Deterioration processes will tend to reduce the achieved levels of structural functionality, safety, serviceability and such like of buildings and structures with time. Figure 11 is a schematic representation of the effects of deterioration processes over time on structural condition. This is set out in terms of two probability distributions; one for the resistance (strength) function $R(t)$ and the other for the loading function $S(t)$. Over time the overlap between the two probability distributions increases.

Strictly, the area of overlap between $R(t)$ and $S(t)$ and marked as $P_f$ on the upper part of Figure 11 is not the failure probability – in spite of the fact that this statement occurs a lot in the literature. The correct failure probability at any time, $t$, involves a more complicated expression, which does not correspond to the area of overlap. In the lower part of the diagram (upper part of Figure 11) the failure probability, $P_f$, is represented by the area under the service life density curve up to time $t$. The increase in the probability of failure with time is illustrated schematically in the lower part of Figure 11 by a cumulative failure probability curve.

Generally damage to a concrete structure is the result of a deterioration process acting over a period of time, probably over years and possibly over decades. Only some causes of physical damage, such as explosion, impact or overloading may result in the sudden appearance of severe damage.

Most deterioration processes (eg corrosion of reinforcement, alkali–silica reaction (ASR), etc.) result in the gradual development of cracking of concrete.
Typically there is then a period when the failure rate is more or less constant (Stage B); with the failures which do occur are caused primarily by accidents or some form of calamity, such as those arising from fires, explosions, etc.

Finally, towards the end of the service life of the building/structure, there is period when the failure rate is observed to increase with time (Stage C). The increasing failure rate is associated with ageing or deterioration of the buildings/structures. The increase in the probability of failure occurs in spite of the previous long period of satisfactory through-life performance of the buildings/structures concerned.

The ‘bath-tub’ curve is often simplified to a constant failure rate with time. In the context of Figure 12, such behaviour would be portrayed by a horizontal line. Whilst such an approach would make any calculations simpler, this simplification ignores and under-estimates issues associated with teething problems in the early stages of service life or effects due to ageing/deterioration in the later stages of service life.
involve undertaking a programme of investigations and/or re-bolting of the external leaf of the cladding panels perceived to have a higher risk of detachment.

Adopting proactive management of assets, rather than a reactive approach, has been demonstrated to minimise whole life costs, whilst potentially reducing the risk of injury to the public and building users, by seeking to avert the premature deterioration or failure of a building or its components. Thus it is desirable that building owners take a proactive approach to the management of the remaining life of LPS dwelling blocks.

The investigation and assessment processes adopted need to give LPS dwelling block owners an understanding of the need and urgency of any interventions required upon the building, particularly if these are essential for its structural safety.

4.3 APPROACHES TO THE THROUGH-LIFE MANAGEMENT OF LPS DWELLING BLOCKS

There are two main approaches that can be adopted for the management of an existing LPS dwelling block through its life. There is the reactive approach to through-life management. In this case maintenance or remedial works are typically triggered by the occurrence of readily observable damage to a part of the structure, such as the existence of cracking or spalling of concrete caused by the corrosion of embedded reinforcing steel. Remedial works are then initiated in order to slow the rate of deterioration and extend the service life of the structure.

The second is the proactive approach, which involves taking action at an earlier stage. In the case of reinforcement corrosion, the goal would be to delay the start of corrosion of the embedded reinforcing steel by undertaking a preventative intervention. In the case of corrosion this might involve the early application of a coating to the surface of the concrete or perhaps the installation of a cathodic protection system. Thus the proactive approach involves taking appropriate action before a critical situation or event occurs.

In the particular context of LPS dwelling blocks, the external leaf of some types of cladding panels have been found to have a higher risk of detachment. A proactive approach to managing and controlling this situation would involve undertaking a programme of investigations and/or re-bolting of the external leaf of the cladding panels perceived to have a higher risk of detachment.

Adopting proactive management of assets, rather than a reactive approach, has been demonstrated to minimise whole life costs, whilst potentially reducing the risk of injury to the public and building users, by seeking to avert the premature deterioration or failure of a building or its components. Thus it is desirable that building owners take a proactive approach to the management of the remaining life of LPS dwelling blocks.

The investigation and assessment processes adopted need to give LPS dwelling block owners an understanding of the need and urgency of any interventions required upon the building, particularly if these are essential for its structural safety.

4.4 UNDERLYING PREMISE OF ASSESSMENT METHODOLOGY DEVELOPED BY BRE

On the basis of evidence available from various sources including the historic load testing of LPS components, assemblies and some full-scale testing which was carried out in the aftermath of the partial collapse of Ronan Point (see Appendix D); BRE postulated that some forms of LPS dwelling block would be sufficiently strong to accommodate the forces associated with the overpressure generated during a structurally significant internal gas explosion in a building without a piped-gas supply.

The absence of a piped-gas supply implies that an internal gas explosion in an LPS dwelling block is unlikely to be more onerous than the ‘severe’ explosion category (see Appendix B). As such this suggests that the overpressure from an internal gas explosion is unlikely to exceed the specified structural assessment overpressure criterion of 17 kN/m² for a building without a piped gas supply.

Full-scale tests were then performed by BRE on three LPS dwelling blocks during two research projects to confirm that the postulated behaviour was reasonably valid and supported by physical behaviours directly observed by BRE. The load testing was supported by associated finite element modelling and calibration exercises. This work is described in Appendices E to H.
This work led to the development and refinement of a hierarchical structural assessment procedure for LPS dwelling blocks subject to accidental internal overpressure loads. The LPS dwelling block structural assessment methodology was developed primarily with the specified overpressure criterion of 17 kN/m² in mind. At this magnitude of internal overpressure the majority of wall panel and floor slab components at most floor levels are not expected to fail.

However, the applied loads generated would generally be expected to cause the failure in the joints between wall and floor components, in particular at the base of wall panels which would result in their sideways translation under the applied lateral loads. That is away from the site of an internal gas explosion. Thus, for example, the flank walls would be pushed in an outwards direction. This problem can be solved by local strengthening at the base of the wall panels. Flank wall panels in the upper levels of an LPS dwelling block are particularly prone to this form of failure under accidental internal overpressure (gas explosion) loads.

Experience suggests that, in general, it is impractical to attempt to justify LPS dwelling blocks for loads associated with an accidental internal overpressure of 34 kN/m² (unless the LPS dwelling block was specifically designed for the loadings associated with a piped gas supply). This is because the wall and floor components are expected to fail in flexure and in base shear, which would necessitate extensive strengthening measures. The extent of such strengthening measures would be much greater and technically complex than those required to stabilise the base of the wall panels, as described above. Accordingly, it is generally most practicable to remove the piped gas supply.

The hierarchical approach to the structural assessment of LPS dwelling blocks can potentially involve the following steps which utilise increasingly sophisticated forms of structural assessment calculation:

**Assessment Level 1.** A deterministic linear elastic spreadsheet based analysis of the wall panel and floor slab components, together with an empirical evaluation of the likely performance of the associated joints between components based upon experience gained from full-scale testing of LPS dwelling blocks containing similar joints.

**Assessment Level 2.** A deterministic non-linear elastic finite element (or alternative) analysis of selected wall panel and floor slab components which failed to satisfy the structural Assessment Level 1 procedure, as may be appropriate.

**Assessment Level 3.** Probabilistic calculations of selected wall panel and floor slab components which failed to satisfy the structural Assessment Level 2 procedure, as may be appropriate.

With this hierarchical approach to structural assessment, it is assumed that the assessment process will be stopped as soon as is practicable. Thus, if it is possible to justify the behaviour of all the wall and floor slab components in an LPS dwelling block using structural assessment calculations performed to Assessment Level 1, then the process would stop.

If it was possible to justify the behaviour of only some of the wall and floor slab components of the LPS dwelling block at Assessment Level 1, the process would stop for those components. The remaining wall and floor slab components would then proceed to Assessment Level 2, and so on.

The general expectation is that, unless the LPS dwelling block concerned was designed and built to accommodate the overpressure criterion associated with a piped gas supply (ie 34 kN/m²), that LPS dwelling blocks will not have a piped gas supply and that they will, therefore, be assessed for an overpressure criterion of 17 kN/m².

The underlying assessment concept is to seek to verify that structural failure in an LPS dwelling block is unlikely to be initiated under accidental loads and action effects. As a result the structural assessment methodology does not consider post-failure behaviour, such as the ability to mobilise alternative load paths.

If the behaviour of the LPS dwelling block cannot be justified by the structural assessment calculations and the proposed risk reduction/management measures cannot reduce the risks due to accidental loads and actions to the point that they might be ‘regarded as insignificant and adequately controlled’; the respective wall panel and floor slab components and/or associated joints that failed the structural assessment would need to be strengthened.

The strengthening provided would need to either prevent initiation of structural failure under the applied accidental loads or would need to control the post-failure behaviour of the LPS dwelling block.

In the case of LPS dwelling blocks designed after 1968, their post failure behaviour should be controlled by the additional horizontal and vertical tying required by guidance introduced soon after the Ronan Point incident.
5 HAZARD IDENTIFICATION

5.1 INTRODUCTION
In developing an appropriate assessment methodology for existing LPS dwelling blocks, it has been necessary to examine a number of factors that could have a bearing on their behaviour. These include material and structural behaviours, material properties as well as the hazards that these types of buildings are likely to be subjected to during their service life.

There clearly are numerous aspects to be considered when seeking to define the hazards that a building might be subjected to. A process for making such an assessment might include:

- Establishing the nature of the hazards involved and considering the potential outcomes.
- Establishing what hazard scenarios should be considered.
- Evaluating the risks involved.
- Clarifying whether the hazards/risks are likely to change with time.
- Establishing which risks can be carried and what criteria should be used in this process.
- Identifying which risks need to be controlled or reduced and how can this be achieved.

The hazards and risks which need to be taken into account for an existing LPS dwelling block should be discussed and agreed with the client/owner of the LPS dwelling block and any relevant bodies/authorities. In this consideration should also be given to possible future effects from climate change.

5.2 SOURCES OF HAZARDS
Sources of hazards (ie sources of potential accidental loads and actions) may include:

- Fire within the building – perhaps especially during any refurbishment or remedial works.
- Accidental internal or external gas explosions (deflagrations).
- Accidental impact by various forms of vehicle, including road vehicles, aeroplanes, trains, etc.
- Hazards due to human errors during the design and construction of a structure or during its operation (service life), or due to the lack of proper maintenance during its service life.
- Unauthorised/inadequately planned structural modifications.
- Environmental hazards such as exceptionally strong wind or heavy snow on the roof – these are normal load effects which could have either local or global influences upon behaviour.
- Hazards due to misuse such as overloading of a floor slab.
- Land slip associated with nearby deep excavations, cuttings or changes in ground level.
- Malicious attacks, such as internal or external explosions that are deliberately caused.
(NB. Malicious acts or attacks are outside the scope of this document.)

A number of these potential hazards are illustrated pictorially in Figure 13.

5.3 ACCIDENTAL LOADS AND ACTIONS ON LPS DWELLING BLOCKS
Notionally, LPS dwelling blocks are susceptible to three principal forms of accidental loading, namely:

- Gas explosions.
  - Internal gas explosions.
  - External gas explosions.
- Vehicle impacts involving:
  - Road vehicles – whilst this is the most probable type of vehicle impact involving UK buildings, historical data indicate that there are no recorded road vehicle impacts which have caused severe or very severe damage to a UK building with a height of five storeys and over – see Appendix B, section B4.
  - Aircraft – whilst there is a very low incidence of aircraft hitting buildings, a number of impacts by both small and large aircraft have occurred around the world. Impacts by large aircraft, with the associated fires resulting from their large fuel load, have caused severe structural damage and, in some cases, complete collapse of all or some parts of a small number of multi-storey buildings outside the UK – see Appendix B, section B5.
  - Trains – statistically these appear to pose a very low risk of causing severe damage to buildings, but instances of such damage have occurred outside of the UK – see Appendix B, section B5.
- Fires also pose a potential hazard to LPS dwelling blocks by virtue of the thermal heating and expansion effects, coupled with the potential damage caused to structural members.
As accidental impacts by vehicles causing severe or very severe damage to multi-storey buildings in the UK appear to be very rare, these hazards do not appear to merit separate consideration.

An overview of the perceived relative risks associated with the hazards listed above is given in the following sections. (Note: Malicious acts or attacks are outside the scope of this document.)

5.4 ACCIDENTAL INTERNAL GAS EXPLOSIONS

A gas explosion (deflagration) may occur if an ignition source is introduced to a mixture of gas and air when the gas concentration is within a certain range; that is between the lower- and upper-explosion limits for the particular flammable gas. The combustion of the gas–air mixture produces a rise in pressure which, for a stoichiometric gas–air mixture in a room, could reach a theoretical overpressure of some 700–1000 kN/m² if the walls and floors of the enclosing room were strong enough to sustain such pressures.

In most cases the rising pressure will result in failure of the weakest component of the surfaces bordering the room containing the explosion, such as a window or a lightweight partition, thus venting the explosion and limiting the peak pressure thereby created. However, experiments have shown that even though venting occurs, the overpressure experienced by the remaining components forming the boundary of the room still continues to rise for a short period. Thus whilst a window may fail at an overpressure of say 6–8 kN/m², the walls and floors that remain in place will typically experience an overpressure somewhat in excess of this limiting value.

Whilst the presence of double glazing of windows is expected to increase the overpressure at which windows fail to some degree, it is not expected to substantially increase the overpressure at which failure occurs. However, it has to be recognised that there is a lack of experimental data on this matter. It appears that performance tests which are undertaken are limited to establishing that windows are able to sustain, without failure, a certain magnitude of pressure loading (such as that due to wind) as defined in the appropriate design standards. However, such testing regimes have no requirement to establish the actual pressure load at which windows/window systems fail.

Appendix B, section B2 presents further information about accidental internal gas explosions.

5.4.1 Classification of explosion incidents

Gas explosions may be classed by their severity and effects; ranging from minor / insignificant effects to those causing structurally significant damage. It is the latter type of explosions, which cause damage over and above the displacement or loss of windows and window frames, which are of concern in the assessment of LPS dwelling blocks. These are classified in accordance with the degree of damage caused. The definitions of damage used are as follows:

Moderate explosion – destroys relatively weak structures up to and including the typical small bungalow, or in the case of more substantial and larger structures, destroys light cladding and windows, damages stud partitioning and similar elements and blows doors off hinges.

Severe explosion – destroys or damages heavier claddings, infills and partitions (ie blockwork and brickwork) and destroys or damages load bearing elements such as the gable wall and first floor of the typical post-1919 two-storey house.

Very severe explosion – destroys or seriously damages stronger load bearing elements (reinforced concrete) or causes very substantial damage over several storeys or to several adjacent buildings of less substantial construction.

Two different forms of internal gas explosion are described in the following sections.
5.4.2 Single-room explosions
Available information indicates that the magnitude of the pressures and forces generated during gaseous explosions vary widely and are influenced by many factors. These include the nature and quality of the explosive substances, air turbulence; as well as the type and nature of the building concerned in which the explosion occurred.

The simplest explosions occur in a single enclosed space/room. In these circumstances the peak pressure is typically limited by the strength of the structure, or vents, rather than by the potential energy available in the gas. This type of explosion probably accounts for over 93%[25] of gas explosions in all types of dwellings in the UK. It can vary in magnitude from minor, in which only superficial damage to windows and fittings occurs, to structurally significant, in which damage to the fabric of the structure of the building concerned occurs.

A limited number of test explosions of this type have been undertaken by BRE in low-rise dwellings[24,25]. Such explosions generated in a single room resulted in a maximum recorded overpressure of 13 kN/m². However, in the BRE tests undertaken in a Sheffield maisonette involving small aerosol canisters, overpressures of between 2.6 and 9 kN/m² were recorded[24].

A study by Ellis and Currie[25] of the overall explosion data set and ancillary information suggests that the 17 kN/m² assessment criterion currently adopted for LPS dwelling blocks without a piped gas supply is considered to be a reasonable higher limit estimate of the overpressure likely to be generated during non-piped gas explosions for use in deterministic assessment calculation procedures. However, for reliability based assessments it is necessary to recognise and take into account that the 17 kN/m² assessment criterion value may be exceeded – indeed it is necessary to estimate the probability distribution function of the complete range of possible overpressures.

Further details about the origin of the current national overpressure assessment criterion for LPS dwelling blocks without a piped gas supply are given in section 7.2, with additional information being presented in Appendix B, section B2.

5.4.3 Multi-room explosions
Pressures generated during the more complex explosions associated with the larger volume of gas available from a piped gas supply can be much greater and involve more than one room. The type of explosion, which propagates from one room to the next, is termed a ‘cascade explosion’. Explosions of this type are mostly within the ‘severe’ or ‘very severe’ categories. Although the peak overpressure created in a ‘very severe’ explosion has not been measured, it may exceed the overpressure assessment criterion of 34 kN/m² which is used for LPS dwelling blocks with a piped gas supply. Research previously undertaken by the then Fire Research Station (now part of BRE Global) in a purpose-built multi-compartment vented steel chamber has shown that cascade type explosions can generate overpressures in the order of 90 kN/m².

5.5 ACCIDENTAL EXTERNAL GAS EXPLOSIONS
Appendix B2 reviews the circumstances relating to major accidental external gas explosions. It is clear that this form of incident can create a substantial overpressure which could cause severe or very severe damage to multi-storey buildings. Fortunately, such explosions appear to be very rare.

In the context of LPS dwelling blocks accidental external gas explosions are probably sufficiently rare events not to require specific consideration. However, it should be recognised that the magnitude of the damage potentially caused by such incidents could lead to overall collapse of the zone of the affected building, or possibly the whole of an LPS dwelling block which was potentially susceptible to progressive collapse. In these circumstances such damage may not be disproportionate.

5.6 ACCIDENTAL VEHICLE IMPACTS
5.6.1 Road vehicle impacts
A review of the occurrence of damage to buildings within the United Kingdom caused by road vehicle impacts over the periods 1971–1981 and 1985–2000 shows that there were no incidents reported causing severe or very severe damage to a UK building with a height of five storeys or more. Overall road vehicle impacts appear to pose a vanishingly small risk of causing structural damage which might lead to progressive collapse or disproportionate damage in an LPS dwelling block.

Nonetheless, it is possible to envisage situations where an LPS dwelling block might conceivably be at risk of a road vehicle impact, such as where an LPS dwelling block is located at the bottom of a hill or steeply sloping access road. In such cases it would be sensible to evaluate the risk of impact by a heavy vehicle and the strength of the section of the block in the anticipated impact zone. The risk of significant structural damage would be reduced where the LPS dwelling block is constructed on an in-situ concrete podium, which would be expected to be more damage tolerant than construction formed using large storey height (effectively unreinforced) concrete wall panels.

Perhaps the greatest risk associated with the impact of a heavy vehicle might not arise from the kinetic energy associated with the impact, but from a large subsequent fire involving a flammable cargo.

The preferred approach would be to eliminate the possibility of a road vehicle impact by installing appropriately designed earth banks, crash barriers or some other form of ‘fender’. A further option could be to re-configure the road layout to direct ‘uncontrolled’ vehicles way from the base of the LPS dwelling block.
Whatever the situation it would be necessary to review the potential risks. However, the review would be undertaken in the knowledge that past experience suggests that there would be only an extremely remote possibility of a vehicular impact occurring which might cause structurally significant damage.

BS EN 1991-1-7\[20\] provides an approach for considering the accidental actions associated with road vehicle impacts.

5.6.2 Aircraft impacts
Accidental impacts by aircraft causing severe or very severe damage to multi-storey buildings appear to be very rare. The literature does contain occasional reports of aircraft impacts with multi-storey buildings, potentially causing considerable damage and casualties, often from the fire associated with the fuel carried by the vehicles. It is reported that the chemical energy of the fuel in an aircraft may be 1000 times its kinetic energy\[26,27\]. The progressive collapse of the steel framed World Trade Center Towers in New York City on 11 September 2001 highlighted the potential implications of aircraft impact, albeit, in circumstances associated with a terrorist incident. Brief details of three accidental aircraft impacts are presented in Appendix B5, giving some insight into the nature of these incidents. Figures B8–B10 illustrate the nature of the damage caused in these three incidents.

Whilst such incidents are rare, it should be recognised that:
- An impact by an aircraft could lead to the partial or total collapse of a LPS dwelling block (or other type of building) depending on circumstances.
- The damage resulting from such an incident might be considered to be proportionate to the initiating event.

It is concluded in Appendix C that the risks associated with the impact of an aircraft on an LPS dwelling block can be ‘regarded as insignificant and adequately controlled’.

5.6.3 Train impacts
Instances of buildings being impacted by trains are also rare, although such an event did occur in Japan in 2004. A train derailed near Amagasaki, 400 km west of Tokyo. The first carriage slid into the first floor parking garage of a high-rise apartment block situated immediately adjacent to the raised rail track bed. The derailed carriages after impact with the building are shown in Figure B11 in Appendix B.

Whilst the force of the impact resulted in significant localised damage, the overall extent of damage was considered to be proportional to the magnitude of the impact. As the impact occurred in Japan, it is assumed that the high-rise apartment block would have been designed for earthquake loading. The seismic design requirements applied would undoubtedly have ensured that the Japanese apartment block had a high degree of robustness and it seems likely that it would have been more tolerant of local damage than an equivalent UK apartment block. It was reported that 106 passengers and the driver were killed. A further 549 others were injured.

BS EN 1991-1-7\[20\] provides an approach for considering the accidental actions associated with derailed rail traffic under or adjacent to structures. This approach provides a basis for the classification of buildings depending on their location relative to the railway tracks.

5.6.4 Vehicle impacts – Summary
On the basis of the information available it seems reasonable to conclude that the risks associated with the impact of any form of vehicle on an LPS dwelling block can be ‘regarded as insignificant and adequately controlled’. Consequently, in most instances it will only be necessary to consider accidental loading associated with either piped or non-piped gas internal explosions when undertaking a structural assessment of an LPS dwelling block for accidental loads and actions.

There may, however, be particular circumstances associated with an individual LPS dwelling block which exceptionally would require further consideration of vehicle impact, but such situations would have to be assessed on a case by case basis.

5.7 FIRE
Fires are generally not classified on the basis of the medium, severe and very severe incident categories used for gas explosions and vehicle impacts with buildings which have been described previously in this Handbook. The occurrence of fires in the UK is captured as fire statistics, via a fire and rescue service reporting system which collects information on all fires in buildings, vehicles and outdoor structures and any fires involving casualties or rescues in England, Wales, Scotland and Northern Ireland. The findings are analysed and published every year as the Department for Communities and Local Government Fire Statistics\[28\].

From the above source and the observations made in Appendix B, taking the 2006 fire statistics for United Kingdom as being indicative, it will be seen that:
- The risks to life arising from fires within LPS dwelling blocks are estimated to be significantly larger than those arising from the other accidental hazards discussed earlier in this section. However, in the vast majority of cases, the deaths and injuries which occur during a fire incident are not associated with structural failure of the building.
- There are a total of about 55,800 fires in UK dwellings (2006 figures). Of these about 45,700 are classed as being accidental fires (figures rounded). This corresponds to an annual total fire occurrence of about two fires per 1000 dwellings.
- There are about 360 deaths from fires in dwellings (2006 figures), which corresponds to about seven fatalities per 1000 fires. Alternatively, based on the overall number of UK dwellings\[28\], this result can be broadly expressed as being about 15 fatalities per year due to fire per year per million dwellings, which is significantly larger than the numbers associated with
the other accidental hazards discussed earlier in this section.

- In addition, there are about 11,300 non-fatal injuries due to fires in dwellings (2006 figures), corresponding to 202 injuries per 1000 fires.
- The suggestion is that about 10% of fires in all UK buildings cause severe structural damage.
- The suggestion is that about 3% of fires in all UK buildings result in destruction of the structure.

Historically, the number of injuries and deaths associated with fire incidents have gradually fallen over the years as safety measures and preventative measures have continued to improve. However, they are still significantly larger than the numbers associated with the other accidental hazards discussed in this section.

The data available in terms of fire statistics provide no insight into matters such as the chance of an internal explosion being followed by a large fire or conversely the prospect of a large fire being followed byinitiating an internal explosion after thermal movements have disturbed the joints/structure forming the LPS dwelling block.

Experience of fires in other types of concrete structure suggests that the potential threat to the stability and performance of an LPS dwelling block would be influenced by the effects of the fire on the concrete structure during the heating phase and, potentially perhaps more critically, during the cooling phase and thereafter. The threats to the structural stability of an LPS dwelling block from a fire/associated thermal effects would be expected to relate to the possibility of events such as local floor collapse, thermal expansion/movements causing disturbance to joints between concrete elements, induced displacements forcing the flank wall to be out-of-plumb, compressive loads in floor slabs, redistribution of gravity loads within the structure as a result of thermal expansions and movements.

5.8 STATISTICS ON EXPLOSIONS AND ASSOCIATED RISKS

Although the total number of reported explosion incidents within the United Kingdom has shown a considerable decline over the past two decades or so (see Table B1, Appendix B), in recent years there has been a noticeable increase in the number of explosions causing significant structural damage which are attributed to cylinder gas and aerosol cans (see Table B3, Appendix B). However, the number of structurally significant explosions in all types of buildings remains very low, with there being about 40 incidents per year within the entire UK building stock. In total there are about 10 severe or very severe explosions per year within the entire UK building stock. This includes buildings with and without a piped gas supply. Most fatalities associated with explosions in buildings (all types) occur in circumstances where the structural damage caused by the incident is only minor (ie most fatalities are caused by the direct effects of the explosion, rather than by injury caused by elements of the building collapsing).

The above reported increase over recent years in the number of explosions causing significant structural damage attributed to cylinder gas and aerosol cans demonstrates that the hazard and risk environment is a dynamic one, whose characteristics are ever changing.

Historical data indicate that the risk of people being killed or injured by the structural collapse of a building (all types) is very small, demonstrating the high level of safety achieved in-service within the UK building population. However, it should also be recognised that this does not entirely rule out the possibility of the occurrence of a very low probability but high consequence event, such as the collapse of an entire building causing a large number of casualties.

Appendix B, section B2 reviews the overall hazard environment associated with internal gas explosions. Appendix B, section B7 looks in more detail at the circumstances associated with internal gas explosions in buildings without a piped gas supply. Appendix B, section B7 presents details of the estimated probabilities of occurrence and the risk levels associated with internal non-piped gas explosions. The review and estimates of explosion occurrence were prepared on the basis of statistical analysis and review of historical records.

Summarising the available information about the frequency and severity of piped gas explosions in the entire UK building stock (see Table B2, Appendix B):  
- Moderate explosions – about 20 of these occur per year.
- Severe explosions – about eight of these occur per year.
- Very severe explosions – the frequency of these is about one every two years.

Summarising the available information about the frequency and severity of non-piped gas explosions in the entire UK building stock (see Table B3, Appendix B):  
- Moderate explosions – about 10 of these occur per year.
- Severe explosions – about three of these occur per year.
- Very severe explosions – there is no recorded incident of a ‘very severe’ explosion being caused by cylinder gas or other (non-piped) gaseous substances.

It is primarily ‘severe’ and ‘very severe’ explosions which are of interest to the current considerations about structural damage to LPS dwelling blocks. The annual probability of the occurrence of explosions of this severity in UK dwellings is as follows (refer Table B5, Appendix B):  
- Severe explosions – about $0.5 \times 10^{-6}$ per year.
- Very severe explosions – about $0.02 \times 10^{-4}$ per year.

If consideration is limited to cylinder gas or other (non-piped) gaseous substances, the yearly probability of a ‘severe’ explosion in a dwelling is estimated to be about $0.1 \times 10^{-6}$.

Their frequency of occurrence has been considerably lower than the estimate of annual frequency of explosions able to cause structural damage of $3.5 \times 10^{-4}$ made at the time of the Ronan Point Inquiry.
Appendix B, section B8 benchmarks the above risks with those associated with some other hazards which people are exposed to during their everyday lives. These data demonstrate that the risks associated with gas explosions are relatively low.

From the above and the observations made in previous sections of Appendix B, it will be seen that the risks to life arising from fires within LPS dwelling blocks are significantly larger than those arising from the other accidental hazards discussed earlier in this section. The available data suggest that there are about 15 fatalities per year due to fires in dwellings per million dwellings. The non-fatal injury rate due to fires in dwellings appears to be about 30 times larger.

Summarising, the behaviour of high-rise LPS dwelling blocks over the last 30 years or so suggests that the risk of injury or death to their residents arising from structural failure or damage due to accidental loading, such as caused by internal gas explosions, is not appreciably different to that of the general population of UK buildings. There is the suggestion is that about 10% of fires in all UK buildings cause severe structural damage.

However, it should be noted that changes in human behaviour and other circumstances (e.g., the change from CFCs to butane as the propellant gas in aerosols) might alter the future risk environment. Thus the identification of hazards and the quantification of the probability of their occurrence and their severity will need periodic review.
6 RISK ISSUES

6.1 INTRODUCTION
Risk is a term used in the context of exposure to a situation or some form of activity with a potentially adverse or unpleasant outcome. Typically in human terms this is in relation to the danger of injury or death, but also potentially in relation to some form of economic or other loss.

More specifically risk is a quantity that is measured in units of the undesired consequences of failure per unit time (e.g., expected number of fatalities, expected number of serious injuries, expected financial losses, etc.). The word ‘expected’ in this context means the average value of the losses per unit time taken over a long period of time, or many repetitions of the same scenario. Risk can therefore be thought of as the magnitude of undesired consequences of failure multiplied by the probability that the failure event will occur in a specified period of time, e.g., a year.

Risk is evaluated by the combination of the likelihood of occurrence of a particular hazard, or a failure event arising from the presence of a hazard, and the magnitude of the consequences thereof. In engineering situations the concept of risk is typically expressed by an equation in the following format:

Risk (consequence/unit time) = Frequency (event/unit time) × Magnitude (consequence/event)

Section 5 and Appendix B consider various hazards which might affect LPS dwelling blocks and the residents. Estimates of the likely frequency/probability of occurrence of hazards are given below. Consideration is also given to the potential consequences of the failure events induced by the hazards. Together these factors provide a basis for estimating the relative importance of the potential risks associated with the various types of hazards affecting LPS dwelling blocks.

6.2 THE USE OF RISK ASSESSMENTS IN THE DECISION-MAKING PROCESS
Decision-making is generally based on a process commonly referred to as risk assessment. This typically involves a number of stages including hazard identification, risk analysis and evaluation, option analysis and consideration of practicable risk avoidance measures, before risk management or acceptance is considered.

Guidance on what level of risk may be acceptable is generally derived from regulations, standards and knowledge of what constitutes reasonable and best practice in particular industry sectors and situations. Various factors, including cultural, social and economic aspects, need to be considered; as may the consequence of a failure and the associated public perception of such an occurrence.

Issues of risk evaluation and risk management are considered in Appendix C. The issues are treated at both generic and more detailed levels. Both qualitative and quantitative risk assessment approaches are considered, with attention being given to the various hazards identified in Appendix B which might affect LPS dwelling blocks. The concepts of risk identification, communication and risk management considered here align with the approaches to risk analysis adopted in the Eurocodes[11,30].

When risks are considered to be too large for direct acceptance, it is necessary to look for adequate counter measures. When planning counter measures it is important to recognise possible associated hazards, which might diminish the benefits of the proposed new counter measures. The aim is to identify those events or processes where, with an acceptable degree of effort, a significant benefit can be gained. Possible measures can be technical and/or administrative in nature. There are a number of possible strategies which can be adopted to eliminate, reduce or manage risk. For example, in the case of internal gas explosions due to a piped gas supply, the actions could potentially include:

- Removing the hazard – remove the piped gas supply from a block.
- Minimise the hazard – install gas cut off valves which operate in the event of a leak.
- Controlling the hazard – use suitable alarm systems, vigilance, undertaking a regime of planned inspections, etc.
- Reduce the possibility of failure – provide load-carrying members with an adequate capacity to resist potential overpressure loads due to an accidental internal gas explosion.

6.3 THE ALARP AND SFARP PRINCIPLES
The ALARP principle adopted by the HSE defines that the residual risk should be ‘As Low As Reasonably Practicable’ (ALARP). The approach is now central to UK considerations on health and safety management.
In this Handbook reference is made to managing risks on the basis of ALARP principles. The term ALARP is used widely in the technical literature and in guidance. However, an alternative term that is commonly used in legislation is SFARP, ‘So Far As is Reasonably Practicable’. In this document where reference is made to the term ALARP this should be taken to also include the alternative, SFARP.

The issue of risk levels and probability of occurrence which are considered acceptable to society are discussed in a number of publications including the following:

- The tolerability of risk from nuclear power stations prepared by the Health and Safety Executive in 1992[26].
- Reducing risks, protecting people, prepared by the Health and Safety Executive in 2001[27].

The tolerability of risks is illustrated diagrammatically in Figure 14. This indicates the levels of annual probability of death of an individual in workplace situations at which the risk is considered to be intolerable/unacceptable ($10^{-4}$) and the level at which the risk is considered to be broadly acceptable ($10^{-6}$), with the ALARP/SFARP zone of tolerable risk existing between these two boundaries. Strictly, these probabilities of death apply only to workplace situations and not residential accommodation.

In line with the ALARP/SFARP principle, it is proposed that all LPS dwelling blocks be subject to systematic risk assessment procedures to guide through-life management and associated actions taken, with the goal of:

- Eliminating hazards where practicable.
- Reducing hazards and controlling risks to the structure of these buildings, which are generally at very low levels/acceptable levels, as far as is possible.

This approach to risk reduction and management is sometimes referred to by the acronym ‘ERIC’ which stands for actions to Eliminate, Reduce, Inform about and Control hazards and associated risks.

### 6.4 OVERVIEW OF RISK ISSUES APPLICABLE TO LPS DWELLING BLOCKS

Statistical analysis and historical records of accidental loads or actions described in Appendices B and C indicate that the following risk environment is applicable to LPS dwelling blocks:

- For internal gas explosions (deflagrations) involving cylinder gas or other gaseous substances:
  - Severe explosion – the probability of occurrence during the remaining life of an LPS dwelling block is estimated to be less than one in 1000 (ie $10^{-3}$).
  - The probability of a severe explosion occurring in a single stack (ie flats located immediately above/below each other) of dwellings (assuming a maximum of 25 storeys) is estimated to be less than $2.5 \times 10^{-6}$ per annum.
  - The probability of a severe explosion occurring within the upper five storeys of an LPS dwelling block, where the possibility of initiating a progressive collapse in a single stack of dwellings is greatest, is estimated to be less than $0.5 \times 10^{-6}$ per annum.
  - On the basis of the assumption that 20% of severe explosions might hypothetically create accidental loads that exceed the actual collapse resistance of LPS dwelling block elements; the probability of a progressive collapse in an LPS dwelling block is estimated to be less than $0.1 \times 10^{-6}$ per annum.

- Historically, the level of safety achieved in the overall population of LPS dwelling blocks appears to be comparable with the levels expected for the overall population of buildings within the UK.

---

14 A single stack of dwellings is taken to be the vertical projection of an individual dwelling up the full height of the LPS dwelling block. There are commonly four or six different flats on each storey of a typical UK LPS dwelling block, but sometimes there are significantly more in long ‘slab’ style blocks. For example, the LPS dwelling block floor plan portrayed in Figure 3 shows eight flats per storey. Whilst the incidence of a non-piped gas explosion occurring in a single stack of individual rooms is expected to be lower than the figures quoted in the main text above, the historical explosion statistics record does not provide such detail.
The ‘upper bound’ number of fatalities caused by a progressive collapse in a single stack of dwellings resulting from a severe explosion in a single LPS dwelling is estimated to be about 60 people.

- The notional risk of death to an individual associated with a ‘severe’ explosion involving cylinder gas or other gaseous substances is estimated to be in the order of 0.1 × 10⁻⁶ per annum.

- For internal gas explosions (deflagrations) in blocks with a piped gas supply or a basement the historic evidence suggests that only these circumstances would conceivably result in a ‘very severe’ explosion. The probability of a ‘very severe’ explosion occurring in a single stack of dwellings (assumed a maximum 25 stores) is estimated to be less than 0.5 × 10⁻⁶ per year.

- External gas explosions (deflagrations) – the historic record suggests that these are probably sufficiently rare events not to require specific consideration. However, their magnitude may be such that collapse of part or all of an LPS dwelling block could occur. Such damage may not be disproportionate.

- Road vehicle impact – in the 20 year survey period no road vehicle impact had caused severe or very severe damage to a UK building with a height of five storeys and over.

- Aircraft impact – the annual probability of an aircraft impact on an LPS dwelling block in the London area is estimated to be less than 1 × 10⁻⁶. The magnitude of the damage caused may be such that collapse of part or all of an LPS dwelling block could occur. Such damage may not be disproportionate. While such an impact might conceivably create 350 fatalities in one of the largest LPS dwelling blocks (assuming a maximum of 150 dwellings in the block and that the average number of people living in a UK dwelling is 2.3), the causalities would generally be expected to be less than 100 people.

- Fire – the risks to life arising from fires within LPS dwelling blocks are estimated to be significantly larger than those arising from the other accidental hazards discussed above. However, in the vast majority of cases, the deaths and injuries which occur during a fire incident are not associated with structural failure of the building. In 2006 there were a total of 55,800 fires in UK dwellings, which corresponds to an annual total fire occurrence of about two fires per 1000 dwellings. In 2006 fires in UK dwellings resulted in a total of 363 deaths, which equates to about 15 fatalities per year per million dwellings.

Thus the statistical assessment of the annual probability of death for an individual due to an accidental load or action other than fire (eg an internal gas explosion) is estimated to be below the threshold at which action might be required. On this basis, rationally the risks might again be regarded as insignificant and adequately controlled.

Considering the number of multiple fatalities arising from an incident such as the progressive collapse of a single stack of dwellings in an LPS dwelling block, plotting these on an F–N curve (see Figures C3–C5 in Appendix C) suggests that risk levels are sufficiently low for them to be considered to be socially acceptable. The situation was also evaluated using the cost of preventing a fatality. This approach also indicated that the annual probability of an individual fatality is below the threshold at which action might be required. On this basis, rationally the risks might again be ‘regarded as insignificant and adequately controlled’.

However, in light of the historical aspects of the partial collapse of Ronan Point, wider societal and emotive considerations might bring a modified perspective to this deliberation. It is expected that this would effectively require that a lower level of risk be achieved than the 10⁻⁶ criterion suggested in the above discussions. An aspirational risk acceptance criterion for LPS dwelling blocks would be to match the general level of safety achieved in the overall population of UK buildings.

A review of the statistical data available for the UK building stock by Ellis and Currie[25] and also by CIRIA[32] indicates that the yearly probability of a building collapse is about 0.1 × 10⁻⁶. If the past performance of the overall population of LPS dwelling blocks is compared with that for the general population of UK buildings (ie the above information); this suggests that the level of safety achieved in the two groups is comparable. This performance is thought to result mainly from the following:

- The special measures put in place for the assessment and management of LPS dwelling blocks following the partial collapse of Ronan Point in 1968.

- The fact that LPS dwelling blocks are likely to have a higher collapse resistance than previously acknowledged and indicated by simplified structural assessment calculations.

- General safety improvements that have occurred over the intervening period. The overall number of ‘severe’ and ‘very severe’ accidental explosions within UK buildings has fallen significantly over the years, even though the total number of minor (non-piped) gas explosions has increased noticeably over recent years (believed to be due in part to the change from the use of CFCs to butane as the propellant gas in many aerosols).

The above estimated probabilities of occurrence can be put into perspective by considering the ultimate limit state design lifetime reliability/probability (calibration) requirements defined in ISO 2394[13]. These are shown in Tables 1A and 1B. Table 1A details the target ultimate limit state design life-time reliability index (β-values) recognising the influence of the relative costs of safety measures (high, medium and low) and the potential consequences of failure (small through to great). The target ultimate limit state design life-time reliability index (β-values) are set higher for more severe consequences.
Of course, the concept of ‘acceptable’ risk of human death resulting from structural failure raises very sensitive questions which relate to matters of public perception. These matters are discussed further in Appendix B.

\[15\] It should be noted that these reliability indices do not distinguish between the performance of structural components and structural systems.

---

**Table 1A: Target ultimate limit state design life-time reliability index (β-values)**

<table>
<thead>
<tr>
<th>Relative costs of safety measures</th>
<th>Consequences of failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Small</td>
</tr>
<tr>
<td>High</td>
<td>0</td>
</tr>
<tr>
<td>Medium</td>
<td>1.3</td>
</tr>
<tr>
<td>Low</td>
<td>2.3</td>
</tr>
</tbody>
</table>

**Table 1B: Postulated relationship between the ultimate limit state design life-time reliability index (β-values) and the probability of occurrence (failure), Pf**

<table>
<thead>
<tr>
<th>Pf</th>
<th>$10^{-1}$</th>
<th>$10^{-2}$</th>
<th>$10^{-3}$</th>
<th>$10^{-4}$</th>
<th>$10^{-5}$</th>
<th>$10^{-6}$</th>
<th>$10^{-7}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>β-value</td>
<td>1.3</td>
<td>2.3</td>
<td>3.1</td>
<td>3.7</td>
<td>4.2</td>
<td>4.7</td>
<td>5.2</td>
</tr>
</tbody>
</table>

Note: The information in Tables 1A and 1B is derived from Tables E1 and E2: ISO 2394[33]. The notes given in ISO 2394 indicate that these values were derived assuming lognormal or Weibull models of resistance, Gaussian models for permanent loads and Gumbel extreme value models for time-varying loads and with the design value being established using the first order reliability method (FORM).
7 ORIGIN OF CURRENT NATIONAL ASSESSMENT OVERPRESSURE CRITERIA

7.1 OVERPRESSURE LOADING ASSOCIATED WITH INTERNAL PIPED GAS EXPLOSIONS

As part of the site investigations undertaken within Flat 90 of Ronan Point following the piped-gas explosion and associated collapse, investigators examined quantitative evidence provided by certain objects. The nature and extent of damage sustained by these items provided an indication of the likely maximum overpressures that were generated at various locations throughout the flat.

The Tribunal report\[12\] indicated that the most important item of evidence was the distorted fuse box cover in the hall. Subsequent experiments undertaken upon a similar undamaged cover indicated that the peak overpressure that occurred around the hall fuse box was in the order of 12 lb/in\(^2\) (\(\approx 83\) kN/m\(^2\)). Similar tests were also carried out upon three new biscuits tins that were nominally visually identical to those present in the kitchen of Flat 90. The original three tins were estimated to have experienced overpressures of between 3 and 9 lb/in\(^2\) (\(\approx 21 \text{ to } 62\) kN/m\(^2\)).

The report concluded that since the flat’s occupant did not suffer from ruptured ear drums it is unlikely that the overpressure within the kitchen exceeded 10 lb/in\(^2\) (\(\approx 69\) kN/m\(^2\)); although a maximum overpressure of 12 lb/in\(^2\) (\(\approx 83\) kN/m\(^2\)) is likely to have occurred in the hall of Flat 90.

In considering the behaviour of the structure during the explosion, the investigators engaged by the Tribunal estimated that an overpressure of approximately 5 lb/in\(^2\) (\(\approx 34\) kN/m\(^2\)), but actually 34.5 kN/m\(^2\)) would have produced the observed slight displacement and slight cracking of the party cross wall between the kitchen to Flat 90 and the adjacent rooms in Flat 89. This wall remained in place after the explosion.

Since the flank wall panel bordering the living room and kitchen blew out it was not possible to ascertain the pressure which instigated sliding failure from a simple visual examination. Accordingly, the sliding behaviour of the flank wall panel, which was designed to depend primarily upon friction for its lateral restraint, was investigated at some length by a research team. Whilst there was no question over the ability of the flank wall panels to accommodate either inward or outward pressures due to wind of the order of 0.083 lb/in\(^2\) (0.6 kN/m\(^2\)), the estimates of the overpressure that could instigate sliding failure varied from 0.2 to 5.9 lb/in\(^2\) (1.4–41 kN/m\(^2\)).

To resolve this uncertainty an extensive testing programme was undertaken by the then Building Research Station (now BRE) in collaboration with Dr J C Chapman of Imperial College.

An evaluation of the possible behaviour of the base of the flank wall panel joint included consideration of vertical dead load (including uplift on the ceiling slab), wear and tear of the joint over the life of the block, wind suction acting over more than one storey level and contribution made by the levelling dowels. Taking these factors into consideration the investigators estimated that an overpressure of approximately 5 lb/in\(^2\) (\(\approx 34\) kN/m\(^2\)) would have been required to produce sliding failure at the base of the flank wall\[12\].

However, the geometry of the joint and the in-plane dead loads at the head of the flank wall panel differ from those at the base of this panel. A description of the geometry and predicted plane of failure is provided in the Tribunal report\[12\]. The investigators calculated that for the conditions that existed at the head of the flank wall panels an overpressure of approximately 3 lb/in\(^2\) (\(\approx 19\) kN/m\(^2\)) would have been sufficient to cause a sliding failure.

Laboratory testing of floor panels indicated that the above pressures (\(\approx 3\) lb/in\(^2\) and \(\approx 6\) lb/in\(^2\)); corresponding to \(\approx 19\) kN/m\(^2\) and \(\approx 41\) kN/m\(^2\)) accord reasonably well with those required to break the floor panels in upward (\(\approx 2\) lb/in\(^2\), ie \(\approx 14\) kN/m\(^2\)) and downward (\(\approx 4\) lb/in\(^2\), ie \(\approx 26\) kN/m\(^2\)) loading. However, it was noted in the report\[12\] that the strength (parameter type not defined in the report) of the test floors were above the specified strength. Therefore the strength of the test floors may have been greater than those used in the construction of Ronan Point.

The Tribunal panel concluded that the Ronan Point ceiling slab failed, or partly failed, due to the upward overpressure loading, which meant that the strength of the top joint\[17\] was not fully realised. They also concluded that the floor to the flat did not fail due to the overpressure, but probably failed under impact loading arising from debris falling from above.

In reviewing the results of the load tests and visual inspections of the remaining sections of the structure the Ronan Point review panel concluded that the gas pressure

---

\[12\] That is the horizontal joint between the head of the wall panel bordering the site of the explosion and the end of the floor slab that is bedded on it.

\[17\] That is the horizontal joint between the head of the wall panel.
on the flank wall to the lounge and bedroom probably rose to a peak in a few milliseconds to a maximum of about 5 or 6 lb/in², that is \( \approx 34 - 41 \) kN/m². It was also concluded that the overpressure then fell away over a period of approximately 0.1 seconds, with the wall being subjected to an average overpressure of approximately 3 lb/in² \( (\approx 21 \) kN/m²).

Flexural test undertaken upon a single Ronan Point wall panel suggested that such panels might fail in bending at an applied overpressure of 6.9 lb/in² \( (\approx 48 \) kN/m²).

It has been postulated that the three flank Ronan Point wall panels would have failed effectively simultaneously under an average overpressure of about 3 lb/in² \( (\approx 21 \) kN/m²), with the panels being pushed sideways (ie outwards) away from the face of the block, whilst remaining reasonably parallel to it. It was postulated that there would have inevitably been some differential movement between the panels, which was put down to the lack of horizontal mechanical ties between these elements in the Ronan Point construction. Since the flank wall panels were found at ground floor level fairly close to the base of the flank wall of the block, it was concluded as being consistent with a moderate average overpressure.

The complex nature of explosion front propagations (in space and time) has led to the adoption of an equivalent uniform static pressure as a basis for designing and assessing LPS dwelling blocks. The value of 34 kN/m² \( (\approx 5 \) lb/in²) was chosen as a practical design/assessment criterion in the absence of unambiguous data.

Therefore based on the evidence presented in the Ronan Point Tribunal report it may be argued that the currently adopted overpressure assessment criterion for a building with a piped gas supply of 34 kN/m² was arrived apparently arbitrarily. Indeed this was challenged by Thomas Akroyd who recommended that the strengthening of structural elements should be capable of accommodating a peak pressure of 70 kN/m². However, an independent review of his report by BRE concluded that it was not a reasonable design/assessment criterion. This view was supported by a Court of Appeal judgement.

Ellis and Currie reviewed the information available on UK gas explosion compiled in BRE studies concerned with the incidence of damage to buildings caused by explosions and vehicle impacts. The explosions studies considered two 10-year periods, namely between 1971–1981 and 1984–1994. They concluded that explosions in the ‘severe’ and ‘very severe’ category tend to generate damage in more than a single room (ie these are termed ‘multiple room explosions’). Although the peak pressure in a ‘very severe’ explosion has not been monitored, Ellis and Currie anticipated that it could significantly exceed 34 kN/m². ‘Very severe’ explosions typically require large volumes of gas which practically can only be delivered by some form of piped gas supply. Further details on this subject are given in Appendix B2.

In conclusion, whilst it is recognised that overpressures in excess of 34 kN/m² \( (\approx 5 \) lb/in²) could be generated during a piped gas explosion, the view was that it is reasonable for this value to be adopted when assessing LPS dwelling blocks containing a piped gas supply.

### 7.2 OVERPRESSURE LOADING ASSOCIATED WITH INTERNAL NON-PIPED GAS EXPLOSIONS

Liquid petroleum gas (LPG) is occasionally used by residents for space heating due to its perceived cheapness in comparison with electrical forms of heating, even though the potential risks of using this fuel are widely known. The practice is contrary to warnings that LPG cylinders should not be used or stored in LPS dwelling blocks. It has also been known for residents, perhaps unwittingly, to use LPG in other contexts. For example, the use of gas-fired barbecues within LPS dwelling block flats has resulted in a number of explosions, fires and injuries. Such practices are contrary to the spirit and contractual terms of occupancy/residence applicable to most LPS dwelling blocks.

Small aerosols canisters, of the type used for hair sprays and deodorants, now pose a potential explosion risk due to the change of propellant gas from CFCs to butane. An indication of the magnitude of overpressures generated in explosions involving small aerosols is given in Appendix E3.

As natural (piped) gas is less dense than air, it tends to rise towards the upper part of the room. Thus, in these circumstances the explosive gas–air mixture tends to collect at the top of the enclosing volume (room). Accordingly, the burning front can progress through the explosive gas–air mixture without impinging significantly on furniture or other internal fixtures and fittings. It therefore has a more direct route to the perimeter of the enclosing volume/of the external perimeter of the building. Explosion pressures can be increased by natural air turbulence.

Conversely, LPG which comprises either propane butane, is more dense than air and so will tend to sink towards the floor. Accordingly, LPG tends to accumulate in unventilated voids or areas below the source of the leak. Thus, the flame propagation of LPG can potentially be influenced by furniture which may tend to lead to turbulence and hence potentially higher pressures than would occur, say in an empty room. Since venting of explosion overpressures normally occurs via failure of windows and/or the frames which tend to be in the upper portion of a room, the route to venting is indirect.

Whilst the derivation of the ‘higher’ overpressure criterion of 5 lb/in² \( (34 \) kN/m²) adopted for the assessment of LPS dwelling blocks containing a piped gas supply is relatively well documented (as discussed in section 7.1), the justification for the ‘lower’ overpressure assessment criterion \( (17 \) kN/m²) given in MHLS Circular 71/68 for LPS dwelling blocks without a piped gas supply seems less clear. However, subsequent research undertaken by BRE and the Fire Research Station has shown that this reduction is reasonable.

Ellis and Currie reviewed the information available on UK gas explosion compiled in BRE studies concerned
with the incidence of damage to buildings caused by explosions and vehicle impacts. The explosions studies considered two 10-year periods, namely between 1971–1981 and 1984–1994. Ellis and Currie concluded that an overpressure of 17 kN/m² appeared to form a reasonable upper bound for a single room explosion, which corresponds to the recommended overpressure to be used during structural assessments of LPS dwelling blocks for accidental loads when a piped gas supply is not present. Further details are given in Appendix B2.

Summarising, full scale explosion tests using bottled gas, small aerosols and cylinders have been undertaken both in dedicated explosion cells and in existing buildings. This research indicates that whilst the pressures generated during the explosions involving LPG are often less than the currently accepted criterion for non-piped gas explosions, it is reasonable to take an overpressure of 17 kN/m² as a likely higher limit for this type of event. However, previous experience has indicated that venting via windows and other relatively weak elements of the construction typically reduces the peak pressure generated to less than the structural assessment overpressure criterion of 17 kN/m².

7.3 OVERPRESSURE LOADINGS ASSOCIATED WITH INTERNAL EXPLOSIONS INVOLVING PETROL OR OTHER CHEMICALS

A BRE survey primarily of press cuttings of incidents of explosions reported to have occurred between June 1981 and March 1991, indicated that explosions involving petrol or chemicals were fairly rare during this period. Whilst statistics obtained in this manner are expected to be incomplete, a review of the collected data (made at the time the study was carried out) indicated that it was unlikely that any major structurally significant explosion incidents would have been overlooked.

Of the 2688 explosions that were reported to have occurred during the period of the survey in all types of building, petrol or chemicals were implicated in 254 (9.4%) of these. A total of 170 of these explosions occurred in industrial premises, 81 in dwellings and the remainder in offices, public venues, schools, etc.

Only 16 explosions were known to have involved petrol and, of these, seven were rated as severe. In 254 of the cases (9.4%) the cause of the explosion was unknown.

Significantly all the explosions involving petrol occurred in garages or buildings (of all types) of three storeys or less in height. Thus no explosions of this type were reported to have occurred in any building type of four storeys or more in height.

Accordingly, on the basis of these statistics the risk of a petrol-based explosion occurring in a medium to high-rise LPS dwelling block would appear to be vanishingly small. However, it should be noted that tenancy agreements and landlord policing policy has probably worked to reduce or remove this hazard. Nevertheless, if any relaxation occurs in this policy then the risk of explosion from this source might rise above the current level.

No experimental data are available on the likely range of overpressures that might develop during a petrol fuelled explosion in a building. In the absence of such information and in view of the apparently extremely low risk of such an explosion occurring in an LPS dwelling block, it is suggested that the existing overpressure assessment criterion of 17 kN/m² should be satisfactory for this hazard.
8 STRUCTURAL PERFORMANCE OF LPS DWELLINGS BLOCKS EVALUATED BY RECENT FULL-SCALE LOAD TESTS CARRIED OUT BY BRE

8.1 OVERVIEW OF THE STRUCTURAL PERFORMANCE OF LPS DWELLINGS BLOCKS DURING BRE FULL-SCALE TESTS

BRE has gained experience and knowledge about the behaviour of LPS dwelling blocks from two programmes of full-scale testing to the ultimate load condition within three LPS dwelling blocks, comprising two of Bison Wallframe design and one of a Reema Conclad design. The Bison Wallframe blocks were 15 and 22 storeys high; and the Reema Conclad block was 10 storeys high.

These tests enabled certain facets of structural behaviour to be explored and, in all three cases, to demonstrate that adequate reserves of strength existed in these LPS dwelling blocks for the accidental loading situations that they were likely to be exposed to. Thus the three LPS dwelling blocks concerned were able to resist the specified structural assessment overpressure loading criterion of 17 kN/m² for a building without a piped gas supply.

This evaluation took account of the fact that the quality of the execution of the construction (workmanship) was not perfect and that errors had been made when erecting the buildings (eg some site placed reinforcing bars were not correctly located in joints, erection was not fully compliant with rules for the lapping of bars or for their anchorage, and so on).

The programmes of full-scale testing typically involved the following steps:

• Forensic investigation of the form of construction and of build quality in the test blocks.
• Development of analytical models, including the construction of three-dimensional finite element computer models.
• Full-scale load testing within the test buildings – undertaken using static loading.
• Comparison of analytical models/computer modelling outputs with the load test results.

Table 2 summarises the various full-scale structural tests undertaken by BRE within three LPS dwelling blocks. In the 1997 programme of load tests, an initial series of full-scale structural tests was carried out in the Bison Wallframe block involving separate loading of wall and floor components to assess their strength and performance. These preparatory tests were carried out prior to undertaking the room overpressure tests involving simultaneous (combined) loading of wall and floor elements.

The BRE tests involved detailed forensic investigation of the buildings prior to programmes of structural load tests in which components/joints between components were loaded to the ultimate load condition.

In the room overpressure tests loading was applied by a hydraulic test rig to walls and floors simultaneously, this was to simulate the uniform overpressure that is created in a gaseous explosion. In some locations repeat tests were performed with damage to tie connections being introduced between different tests in the series. Although the load transfer had to be carried out slowly (effectively static loading) to ensure the safety of the test crew, the results are still considered to be valid as strain rate effects would be expected to arise which would enhance the performance of the structure under the short duration loads associated with a gas explosion. Thus the current understanding of LPS performance under a simulated overpressure loading has been built up from tests in three

<table>
<thead>
<tr>
<th>Nature of test</th>
<th>Overpressure floor slab</th>
<th>Overpressure wall panel</th>
<th>Overpressure room: Flank wall flat</th>
<th>Overpressure room: Internal flat</th>
<th>Component removal</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandwell (1997)</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td></td>
</tr>
<tr>
<td>Bison Wallframe</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Leeds (1997)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reema Conclad</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td>Y</td>
<td></td>
</tr>
<tr>
<td>Liverpool (2004)</td>
<td>Y</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bison Wallframe</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
LPS dwelling blocks constructed before the Ronan Point incident in 1968 (subsequently referred to as pre-Ronan Point blocks), two being of Bison Wallframe design and one being a Reema Conclad design.

Structural assessments by BRE of Bison Wallframe LPS dwelling blocks had consistently shown that certain types of loadbearing wall elements (kitchen–lounge spine walls) were expected to fail under the overpressure loads that would be associated with a severe gaseous explosion. However it was anticipated that, because of the location of the element within the structure, alternative load paths would be mobilised. A test was carried out to demonstrate this practically by breaking the load path to the foundations, involving removal of the top section of one of these walls with 14 storeys of the building remaining above the test location.

It was concluded that this work had successfully demonstrated that the three LPS dwelling blocks concerned were able to:

- resist the specified structural assessment overpressure loading criterion of 17 kN/m² for a building without a piped gas supply, and
- mobilise an alternative load path(s) after the removal of a particular type of loadbearing wall panel.

The three LPS dwelling blocks used in the BRE testing were made available by their owners prior to the blocks being demolished as part of the redevelopment of the site/estate they were situated in. At the time of testing the three LPS dwelling blocks were vacant, with the residents having been decanted as part of the planned redevelopment of the site/estate. After completion of the BRE tests, the three LPS dwelling blocks were left in a safe condition for the following demolition contractor.

Overall the three LPS dwelling blocks tested are thought to be reasonably representative of the standard of construction of the wider population of LPS dwelling blocks within the UK. On this basis, their performance should also be indicative of the potential performance of the wider population.

Appendices E to H provide further details of the various full-scale structural tests undertaken by BRE within the three LPS dwelling blocks, together with the associated finite element modelling (FEM) carried out. The FEM undertaken in advance of the load testing was used to predict the potential behaviour, as a guide to the planned structural testing. The FEM undertaken after completion of the combined wall and floor tests (the room tests) was done to estimate the uniformly distributed load (as would be generated by a gas explosion) overpressure equivalent of the applied test loads. All loadings were taken to be statically applied and no account was taken of the beneficial strain rate effects that might be associated with the short duration of an actual internal gas explosion.

The outcomes of these programmes of work were subsequently used in the further development of the procedures for the structural assessment of LPS dwelling blocks for accidental loads associated with internal gas explosions.

### 8.2 SUMMARY – THE STRUCTURAL PERFORMANCE OF LPS DWELLINGS BLOCKS EVALUATED BY RECENT FULL-SCALE LOAD TESTS CARRIED OUT BRE

Research\(^{[25]}\) has indicated that an overpressure of 17 kN/m² is a reasonable value to use for the structural assessment of LPS dwelling blocks and other type of building without a piped gas supply, as this threshold value exceeds the magnitude of the overpressures generated during almost all non-piped gas explosions (deflagrations) that have occurred within UK buildings.

Thus on the basis of the results of the structural test results by carried out by BRE, it is judged that the three LPS dwelling blocks would have been able to resist the forces arising during a severe explosion in a dwelling involving cylinder gas or other gaseous substances. This behaviour is taken as a demonstration that the three LPS dwelling blocks in question had adequate collapse resistance (refer to the definition given in Glossary and Definitions section) for the overpressure loading criterion of 17 kN/m² for a building without a piped gas supply.

The historic statistical data available for explosions occurring in the UK building stock in two 10-year periods (1971–1981 and 1984–1994) showed that explosions in dwellings arising from cylinder gas or other gaseous substances are unlikely to be more onerous than the ‘severe’ explosion category. No explosions in the ‘very severe’ category had been recorded arising from these sources.

The statistical data available on explosions have indicated that the yearly probability of a ‘severe’ explosion in a dwelling involving cylinder gas or other gaseous substances is about 0.1 × 10\(^{-6}\).

The statistical data available for the UK building stock\(^{[25]}\) indicate that the yearly probability of a building collapse is about 0.1 × 10\(^{-4}\).

The full-scale structural testing carried out by BRE also showed that, in some particular circumstances, the Bison Wallframe design had an ability to mobilise an alternative load path – that is after the removal of the loadbearing wall element between the kitchen and the lounge in one of the dwellings.

In the context of LPS dwelling blocks, collapse resistance would be taken to be the ability to avoid the initiation of failure under specified accidental loads in a structural panel system comprising brittle concrete wall panels, floor panels which exhibit ductile failure under downward loads but potentially brittle failure when subject to large magnitude upward loading and the associated joints between the structural elements. Once the collapse resistance is exceeded the structural panel system would be at risk of collapse, which might be progressive in nature depending upon the characteristics and damage tolerance of the particular structural panel system used for LPS dwelling block in question.
Thus it is important to recognise that the ability to resist the foreseeable accidental load from an internal gas explosion (deflagration) – that is, having an adequate collapse resistance – does not remove risk of progressive collapse should a load bearing component fail or initiation of failure occur through some other means.
9 LPS DWELLING BLOCK ASSESSMENT – SOME FACTORS INFLUENCING BEHAVIOUR UNDER ACCIDENTAL LOADS AND ACTIONS

9.1 INTRODUCTION
This section outlines some additional factors which might influence the behaviour of an LPS dwelling block under accidental loads and actions. It also provides information which may be of assistance when undertaking a structural assessment of a LPS dwelling block for these circumstances.

It may be pertinent to consider a number of the situations described below during the preparations for such an assessment or during the evaluation of post-assessment options. Often there will be a range of issues and associated problems to consider. These matters are presented in the following three groups of related topics:

- LPS dwelling block structural parameters, such as building height, structural form, the presence of a ground floor in-situ podium, direction of span of floor slabs and material properties.
- Factors influencing behaviour during an internal gas explosion, such as venting.
- Issues associated with piped gas supplies, such as the routing of mains gas supply pipes, location and perceived risks associated with gas burning appliances, i.e. cookers, water and space heaters and such like.

A risk-based framework for the structural assessment of a LPS dwelling block for accidental loads and actions has been adopted by BRE as this provides the basis for dealing with currently perceived hazards and, perhaps most importantly, the facility to respond to evolving situations where the nature of the hazards or the balance between them may change in the future.

9.2 LPS DWELLING BLOCK STRUCTURAL PARAMETERS
In this section comments are made against the following headings:

- Building height and form.
- Direction of span of floor slabs.
- Concrete strength parameters obtained by sampling existing LPS dwelling blocks.
- Joint friction/adhesion parameters.
- Ability to mobilise alternative load paths.

9.2.1 Building height and form
The majority of LPS dwelling blocks are rectangular in plan and are of nominally identical height across their plan area. There are exceptions such as blocks with stepped height form. In addition, the flank walls and the various loadbearing cross and spine walls at each storey level are generally located directly above one another, thus forming a vertical ‘stack’ of walls that are aligned in two horizontal orthogonal directions (see Figures 3 and 6). In this arrangement the wall panels provide a direct load path, typically down to an in-situ concrete podium at ground level.

There are cases where adjoining LPS dwelling blocks share common ‘party’ cross walls and the height of the LPS dwelling blocks vary across the development. In some instances where this has posed difficulties in defining the number of storeys for the respective blocks and, accordingly, the criteria against which the LPS dwelling blocks should be assessed.

Issues of defining the number of storeys in an LPS dwelling block have arisen where a number of storeys of LPS construction have been built on top of several storeys of monolithic reinforced in-situ concrete podium or similar frame construction. This has proved problematic under the existing guidance (MHLG Circulars 62/68[4] and 71/68[5]), with different engineers coming to different views, where the total height of an LPS dwelling block is five or more storeys when the height of the in-situ construction is included, but less than five storeys when the latter is discounted.

The difficulty of defining the number of storeys in an LPS dwelling block has also been compounded where the block has a basement or a partial basement. The latter situation can arise where the LPS dwelling block is situated on a site where the ground level is significantly higher on one side of the block than the other. This can result in the lower floor levels, typically constructed in in-situ concrete construction, being below ground level and potentially being considered as a basement storey.

Another question that has arisen under the existing guidance[4,5] in respect of LPS dwelling blocks with a large lateral dimension and where different parts are of different heights is whether the height classification is made on the basis of the overall block or on the respective sections of the LPS dwelling block concerned.

It is suggested that this type of issue and the specific ones mentioned above are resolved by following the guidance and interpretation of Approved Document A – Structure[6] and with reference to the guidance produced by The Basement Development Group Basements for dwellings[7] and NHBC Technical Guidance Note 13 concerning disproportionate collapse[8]. In practical terms this means:
• **Basements.** In Table 11 of Approved Document A – Structure\(^{36}\) it is stated that when determining the number of storeys in a building, basement storeys may be excluded provided such basement storeys fulfil the robustness requirements for Class 2B buildings.

For the purposes of this guide, the NHBC Technical Guidance Note 13 definition of a basement is used which states that ‘To qualify as a basement storey, the distance between external ground level and the top surface of the basement floor should be at least 1.2 m for a minimum of 50% of the plan area of the building’.

• **Differentiation of parts of buildings.** In Table 11 of Approved Document A – Structure\(^{36}\) it is indicated that where the use of a building involves more than one class, it should be classified as being in the most onerous class. However, potentially there could also be implications for such a classifications arising from the requirements of the Regulatory Reform (Fire Safety) Order\(^{36}\) concerning building height versus risk and escape requirements.

### 9.2.2 Direction of span of floor slabs

As noted above, the typical arrangement for an LPS dwelling block is for the load bearing walls to be aligned in two horizontal orthogonal directions, such that the walls form a vertical ‘stack’ (see Figures 3 and 6). Thus typically there is no horizontal stagger in the location of the loadbearing wall panels and the wall panels provide a direct load path, typically down to the in-situ concrete podium at ground level. However, BRE is aware of one medium height LPS development in the UK where some of the walls are staggered horizontally, which requires lateral load transfer to lower walls through some floor slabs.

In this situation there are instances where floor slabs are supported by sections of three or more wall panels on the storey immediately below. The circumstances are further complicated by the presence of intermediate walls above and/or below particular floor slabs, which potentially provided support/restraint to a floor under upward or downward accidental loading of an internal gas explosion.

This configuration of wall panels presents difficulties when seeking to calculate the load run-down within the building because load shedding occurs in three-dimensions. The problems arise because of the unknown degree of support provided by each wall and hence the share of the load taken by each wall. Whilst the ‘ideal’ load distribution within each supporting wall arising from a uniformly distributed load can be calculated using simple hand calculations or linear finite element analysis, there still remains a significant degree of uncertainty over the accuracy of the results. This uncertainty arises from the sensitivity of the load shedding behaviour of floor slabs to the relative height, bearing length and position of the supporting walls over a number of storeys. The degree of uncertainty increases when more floor levels are considered in the calculations.

Several alternative analytical approaches can be employed when seeking to calculate the load run-down within an LPS dwelling block of this type. These include:

• Using a deterministic calculation approach to make upper and lower bound estimates of the load run-down within each wall type. These values should ‘bracket’ the actual vertical load distribution up the height of each wall ‘type’ in the LPS dwelling block. However, a limitation of this approach is that the difference between the upper and lower bound estimates of the vertical load run-down within each wall type can soon become very large. When the range in vertical load estimates for a wall is large it implies that there is a correspondingly wide uncertainty in the ability of the wall panel to resist lateral loads associated with an overpressure generated by an internal gas explosion. Thus there can be considerable uncertainty in respect of the ‘safety’ of the wall panels in these circumstances. If only the lower-bound estimates of the vertical load run-down within each wall type are used, the outcome is likely to be excessively conservative. This could result in unnecessary expenditure upon structural strengthening works which in reality would only achieve a marginal increase in the assessed ‘safety’ of the LPS dwelling block concerned.

• An alternative approach could be to undertake a probabilistic analysis of the load run-down within each wall type and to make an estimate of the ability of the individual wall panels to resist the lateral loads associated with an internal gas explosion. An analysis of the structure of this type would provide a probabilistic estimate of the risk of failure of each wall panel type with height up the building.

Any requirements for structural strengthening could be formulated using the results of either approach.

The physical arrangement of walls and floor slabs described above is likely to occur only infrequently in LPS dwelling blocks within the UK.

### 9.2.3 Concrete strength parameters obtained by sampling existing LPS dwelling blocks

This section reports upon concrete compressive strength parameters established by testing cored concrete samples obtained from 25 LPS dwelling blocks, with 17 being pre-Ronan Point LPS dwelling blocks (ie designed or constructed before the Ronan Point incident in 1968) and eight being post-Ronan Point LPS dwelling blocks. The purpose of this section is to give an overview of the range of concrete compressive strength encountered in existing UK LPS dwelling blocks.

Summary information is presented for each LPS dwelling block in terms of the following concrete compressive strength parameters:

- Mean concrete compressive strength (N/mm\(^2\))
- Standard deviation for the concrete compressive strength (N/mm\(^2\))
- Characteristic concrete compressive strength, where 95% of values are higher (N/mm\(^2\))

The standard deviation of the concrete compressive strength results for the individual LPS dwelling blocks was calculated making the simplifying assumption that the
results represented a normal Gaussian distribution. This assumption is unlikely to be valid for some blocks and the number of cored concrete samples obtained. However, the approach adopted is commonly used for LPS dwelling block investigations.

Figure 15 shows the summary information for the concrete strength parameters obtained for floor slabs, with the results being grouped into those for pre-Ronan Point LPS dwelling blocks and those for post-Ronan Point LPS dwelling blocks. The results are for cored concrete samples, the majority of which were obtained from the solid edge or the intermediate webs of floor slabs. The results presented are sorted on the basis of ascending mean compressive strength of the cored concrete samples. The results for the pre-Ronan Point LPS dwelling blocks and those for post-Ronan Point LPS dwelling blocks are sorted separately. It will be seen that the concrete compressive strength parameters were about the same for both pre- and post-Ronan Point blocks:

- Mean concrete compressive strength ranging from about 20 N/mm² to 50 N/mm².
- Estimated characteristic compressive strength ranging from about 15 N/mm² to 40 N/mm².

Figure 16 presents similar summary concrete strength parameter information for the wall panels in pre- and post-Ronan Point LPS dwelling blocks. In this case the highest compressive strengths of the concrete used in the wall panels of pre-Ronan Point LPS dwelling blocks sampled appears to exceed that used for the post-Ronan Point LPS dwelling blocks. Thus the mean concrete compressive strength of the pre-Ronan Point LPS dwelling blocks ranged from about 25 N/mm² to over 55 N/mm², whereas for the post-Ronan Point LPS dwelling blocks sampled the range was from about 25 N/mm² to 40 N/mm².

Figure 17 presents the estimated characteristic concrete compressive strength plotted against the mean concrete compressive strength for all the sample data (for all pre- and post-Ronan Point LPS dwelling blocks, floor slabs and wall panels). Figure 17 shows graphically the scatter in the relationship. Overall the relationship between the two parameters is satisfactory, but the plot highlights that there appear to be significant divergences in the results obtained for particular LPS dwelling blocks.

These differences may be partly accounted for by variations in the number of samples (drilled cores) taken in the different LPS dwelling blocks for which results are available. The number of samples and locations tested ranged from very few (less than five samples/locations) in some cases, to perhaps 80 samples/locations per LPS dwelling block in the most extensively sampled structures. The results of the sampling were treated somewhat simplistically, as they were assumed to be random for the purposes of making an initial evaluation of the mean strength and obtaining an indication in the scatter in the results. Furthermore the sampling procedures could not be truly random because of constraints on the locations from which samples (drilled cores) could be taken for compressive strength testing. This and other factors are likely to have affected the outcomes reported in Figures 15–17. The reported values should primarily be taken as being ‘indicative’.

It will be seen from Figures 15–17 that the reported estimated characteristic concrete compressive strength in the LPS dwelling blocks sampled was almost always greater than 15 N/mm², and generally it was greater than about 20 N/mm².
Table 3 presents information on the range of values for the three concrete strength parameters discussed above (i.e., the mean concrete compressive strength, the standard deviation value, and the characteristic strength values) for floors slabs and wall panels, giving the minimum, mean, and maximum values for these parameters for the four groupings of LPS dwelling blocks (i.e., floor slabs and wall panels in pre- and post-Ronan Point LPS dwelling blocks). Figure 18 presents graphically the same information as that contained in Table 3 for floors slabs and wall panels, perhaps making this information more easily understood. However, it should be noted that this presentation does ignore the fact that the minimum standard deviation is not linked to the minimum mean or characteristic value, etc.
In the mid-1980s BRE undertook site investigations of examples of eleven LPS dwelling block systems, which complemented earlier work relating to Taylor Woodrow–Anglian (TWA) LPS dwelling blocks. This work is reported in BRE Report 107: Part 1 – Investigations of construction. The site investigations revealed that the placing and compaction of dry-pack mortar in horizontal joints was the most poorly executed feature of LPS dwelling blocks. It was noted that in some cases the material had been omitted completely and in other cases it was found to be friable, poorly compacted and severely voided. Conversely, in some LPS dwelling blocks the dry-pack mortar was found to be consistently well compacted and filled an acceptable proportion the joint at the base.

### Table 3: Concrete strength parameters – range of values for floors slabs and wall panels

<table>
<thead>
<tr>
<th></th>
<th>Mean (N/mm²)</th>
<th>Characteristic strength (N/mm²)</th>
<th>Standard deviation (N/mm²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Pre-Ronan Point floors</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min.</td>
<td>22.9</td>
<td>16.0</td>
<td>0.9</td>
</tr>
<tr>
<td>Mean</td>
<td>36.0</td>
<td>25.1</td>
<td>6.2</td>
</tr>
<tr>
<td>Max.</td>
<td>48.5</td>
<td>36.2</td>
<td>12.2</td>
</tr>
<tr>
<td>Pre-Ronan Point walls</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min.</td>
<td>27.0</td>
<td>14.3</td>
<td>2.3</td>
</tr>
<tr>
<td>Mean</td>
<td>37.2</td>
<td>25.1</td>
<td>7.3</td>
</tr>
<tr>
<td>Max.</td>
<td>56.8</td>
<td>44.7</td>
<td>12.7</td>
</tr>
<tr>
<td>Post-Ronan Point floors</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min.</td>
<td>29.0</td>
<td>19.4</td>
<td>3.9</td>
</tr>
<tr>
<td>Mean</td>
<td>38.8</td>
<td>28.5</td>
<td>6.3</td>
</tr>
<tr>
<td>Max.</td>
<td>50.5</td>
<td>42.0</td>
<td>9.0</td>
</tr>
<tr>
<td>Post-Ronan Point walls</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Min.</td>
<td>26.4</td>
<td>18.1</td>
<td>2.7</td>
</tr>
<tr>
<td>Mean</td>
<td>32.0</td>
<td>23.1</td>
<td>5.4</td>
</tr>
<tr>
<td>Max.</td>
<td>39.1</td>
<td>33.1</td>
<td>7.7</td>
</tr>
</tbody>
</table>

Figure 18: Concrete strength parameters – range of values for floors slabs and wall panels

#### 9.2.4 Joints at the base of loadbearing wall panels: joint friction/adhesion parameters

The horizontal joints at the base of the loadbearing wall panels to some forms of LPS construction (eg Taylor Woodrow–Anglian (TWA), Bison Wallframe) typically comprise a thin layer of dry-pack mortar located above a zone of in-situ concrete introduced to form the joint between the top of the precast concrete wall panel below and the adjacent precast concrete floor slabs. The dry-pack mortar was introduced after the wall panels had been positioned and levelled using integral levelling bolts. The horizontal joint at the base of the loadbearing wall panels is perhaps 15 mm to 30 mm high.

In the mid-1980s BRE undertook site investigations of examples of eleven LPS dwelling block systems, which complemented earlier work relating to Taylor Woodrow–Anglian (TWA) LPS dwelling blocks. This work is reported in BRE Report 107: Part 1 – Investigations of construction. The site investigations revealed that the placing and compaction of dry-pack mortar in horizontal joints was the most poorly executed feature of LPS dwelling blocks. It was noted that in some cases the material had been omitted completely and in other cases it was found to be friable, poorly compacted and severely voided. Conversely, in some LPS dwelling blocks the dry-pack mortar was found to be consistently well compacted and filled an acceptable proportion the joint at the base.
of the loadbearing wall panels. Apparently no single type of LPS dwelling block was identified as having a greater prevalence of poorly made joints than any other type. All LPS dwelling block systems examined had examples of poorly made joints.

As a result of these differences in compaction and the degree of filling of these joints, the quality of the horizontal joints was found to be highly variable. Problems were also reported with the compaction of the in-situ concrete in the joints between precast concrete wall panel and floor slab components. Typically these problems occurred where access had been difficult during construction.

Where the dry-pack mortar was voided or missing, the vertical load was being transferred through the levelling bolt assemblies and associated packing pieces, supplemented sporadically by support from the dry-pack mortar. However, no signs of structural distress were observed arising from this situation in the buildings examined by BRE. In BRE Report 107: Part 2 – Guidance on appraisal it is observed that the thin horizontal joints at the base of the loadbearing wall panels are likely to be subject to relatively high triaxial confining stress state, which would allow a dry-pack mortar with relatively low (compressive) cube strength to form a joint which had a substantial and adequate vertical load capacity.

It has been postulated that the behaviour of the horizontal joints at the base of the loadbearing wall panels under lateral load will be strongly influenced by the quality of the dry-pack mortar. In the worst case where the dry-pack mortar is uncompacted and classed as being friable it might include many rounded ‘pebbles’ of high strength mortar, which might have a sand: cement ratio of 3 : 1 or perhaps 2 : 1. It has been hypothesised that such a layer could notionally act like ‘ball-bearings’ upon which the base of the loadbearing wall panel might slide. Such a layer would be expected to have a very low resistance to lateral loading created during an internal gas explosion.

However, where the dry-pack mortar is well compacted and fills most of the joint at the base of the loadbearing wall panels; the lateral resistance mobilised at the base of the wall panel could be considerable. This would be expected to be controlled primarily by the compressive strength of the dry-pack mortar and its adhesion to the precast concrete wall panel and the in-situ concrete of the joint. As a maximum, the lateral resistance might notionally approach the shear capacity of the loadbearing wall panel, but would generally be expected to be somewhat less. It is anticipated that the compressive strength of the concrete forming the precast components and the in-situ concrete in the joint would have a lesser influence.

Figures 19 to 22 present illustrative sections through the flank wall and cross wall joints between the base of the wall panel and the adjacent/supporting floor slab for both a pre- and a post-Ronan Point Bison Wallframe LPS dwelling block.

BRE have typically used a coefficient of friction value of 30 degrees for the assessment of joints at the base of loadbearing wall panels when estimating the resistance to lateral load due to overpressure associated with an internal gas explosion. This approach has provided estimates of lateral load resistance compatible with the behaviours seen during the full-scale combined overpressure load tests carried out by BRE in three existing LPS dwelling blocks.

To gain some further understanding of the potential behaviour of such joints, BRE has undertaken experiments to determine the coefficient of friction between a formed concrete surface (ie friction between two concrete surfaces) and another surface comprising dry-pack (ie friction between concrete and dry-pack surfaces). These trials confirmed and expanded upon the results of a limited series of tests that were carried out by BRE in the early-1990s to determine an approximate angle of friction for concrete surfaces that were in intimate contact.

The recent experimental trials and results are described in Appendix K. Summarising, the results for the static friction value obtained were as follows:

- Minimum friction angle measured, $\theta$: 30.3 degrees ($\tan \theta = 0.58$)
- Average friction angle measured, $\theta$: 34.9 degrees ($\tan \theta = 0.70$)
- Maximum friction angle measured, $\theta$: 37.2 degrees ($\tan \theta = 0.76$)

These results are expected to represent a lower bound of the static friction values which is likely to be mobilised in the joints at the bottom of wall panels in actual LPS dwelling blocks, where some degree of chemical adhesion between the dry-pack mortar and the concrete surfaces of the wall and floor joint may also be present.

Other situations that could potentially arise are the absence of dry pack at the base of the wall panel or the absence of the in-situ concrete fill between the floor element and the top of the adjacent (supporting) wall panel – see Figures 19–22. In these situations the upper wall panel would be supported on the wall panel levelling bolts, which would therefore be carrying all the vertical loads in the wall at that location. The occurrence of these extreme situations is considered to be unlikely, but notionally possible. Should the forensic investigation phase of the structural assessment process (see Stage 2, section 12) discover such severe deficiencies, it would be necessary to make a remedial intervention to rectify the problem.

9.2.5 Ability to mobilise alternative load paths

Section 5 of Approved Document A – Structure states that if it is not feasible to provide effective horizontal and vertical tying, then each ‘supporting member’ should be considered to be notionally removed, one at a time in each storey in turn, to check that, on its removal the area at risk of collapse of the structure within the storey and the immediate adjacent storeys, is less than the specified limiting area. This is currently 15% of the floor area or 70 m², whichever is the smaller, but the area limit is soon to be increased to 100 m² to accord with the Structural Eurocode provisions.
Figure 19: Illustrative section through joint between base of wall panel and floor slab – flank wall detail for pre-Ronan Point Bison Wallframe LPS block

Figure 20: Illustrative section through joint between base of wall panel and floor slab – cross wall detail for pre-Ronan Point Bison Wallframe LPS block
Figure 21: Illustrative section through joint between base of wall panel and floor slab – flank wall detail for post-Ronan Point Bison Wallframe LPS block

Figure 22: Illustrative section through joint between base of wall panel and floor slab – cross wall detail for post-Ronan Point Bison Wallframe LPS block
In the context of this requirement, the nominal length of a loadbearing concrete wall acting as a supporting member is defined as the distance between lateral supports subject to a maximum length not exceeding 2.25H, where H is the storey height in metres. As LPS dwelling blocks typically have a floor to ceiling height of about 2.4 m, the nominal length would be approximately 5.3 m. This would equate to a maximum of two adjacent precast concrete wall panels.

In an LPS dwelling block the loadbearing concrete wall panels are stabilised by the adjacent floor slabs. Thus, should the adjacent floor slabs be removed or be substantially damaged by the overpressure loading associated with an internal gas explosion, there is a risk that the affected concrete wall panels could buckle. The vertical load capacity of these concrete wall panels would be impaired and possibly greatly diminished.

The vertical load capacity of the concrete wall panels could also be impaired by lateral displacements associated with heating of floor slabs or other components during a fire.

The flank wall panels are particularly sensitive to these issues, especially where the adjacent room is a lounge with a long floor span. Typically these floor slabs are also one-way spanning between the flank wall and the adjacent cross wall.

Whilst the requirement to check for the implications of the notional removal of floor panels during the check on robustness is not explicitly covered in Section 5 of Approved Document A – Structure\(^1\), some might argue that it is implied by the stated requirement to check that the building remains stable when mobilising an alternative load path. To clarify this point, an explicit requirement has now been included to check for the implications of the notional removal of floor slabs under accidental loads or actions (see section 2).

In terms of the more conventional provision of support to the loadbearing wall panels, the end of a reinforced concrete floor slab would be expected to provide vertical support in a similar manner to a reinforced concrete beam, but dependent upon the reinforcement provision in the floor slab. On this basis, it might be argued that although floor slabs are not explicitly listed in the element types cited in Section 5 of Approved Document A – Structure\(^1\), they could be considered to be included.

### 9.3 FACTORS INFLUENCING BEHAVIOUR DURING AN INTERNAL GAS EXPLOSION

#### 9.3.1 Introduction

This section introduces a number of issues which have been voiced as factors which potentially could influence the behaviour of LPS dwelling blocks during an internal gas explosion. However, it is not clear if the concepts could be practically and reliably applied to an LPS dwelling block at the current time.

#### 9.3.2 Venting mechanisms

BRE have received reports of structurally significant gas explosions in LPS dwelling blocks where the complete window and window frame assembly to the room in which an explosion occurred was blown out. Whilst failure of the individual glass panes may well have occurred during the explosion, the fact that the window frame fixings failed allowing the complete assembly to be blown outwards, would suggest that in these cases the frame fixings acted as a ‘weak link’ in the building envelope. Thus in these circumstances it appears that the relatively low strength of the fixings of the window frame assembly, and not that of the glazing, instigated the venting of the overpressure from the internal gas explosion. In these instances it appears that none of the wall panels or floor slabs bounding the site of the explosion sustained any significant structural damage.

The contemporary situation may be different to that portrayed by the historic data discussed above. Modern (replacement) double glazed window units might have more/stronger fixings, potentially reducing the likelihood of the complete window frame being lost during an explosion. Of course such a situation would reduce the chance of appreciable and beneficial venting occurring.

The concept of active venting of an explosion is used in a variety of circumstances, such as in the petrochemical industry, to reduce the maximum overpressure generated within an enclosed space/a room. Suitably designed and positioned vents are used as part of an active risk management strategy for the circumstances. Such vents are designed to ‘fail’ at a low overpressure such that the maximum attained overpressure would be less than the loading which would cause failure of the elements bounding the enclosed space.

If these concepts were applied to LPS dwelling blocks the overpressure loading on the wall panels and floor slabs would be calculated from the estimated strength of the vent panels/windows. However, at this stage it is not clear if such an approach can be practically applied in an LPS dwelling block. The configuration of the vents would depend upon the layout of each room within a dwelling, as well as the nature/estimated strength of the external cladding panels.

#### 9.3.3 Magnitude of overpressure used in structural assessments

Structural assessments of LPS dwelling blocks for accidental loads and actions are undertaken on the basis of an internal overpressure of either 17 or 34 kN/m\(^2\), depending on whether a block contains a piped gas supply or not. However, a probabilistic based assessment methodology would conceptually provide a basis for examining the potential behaviour of an LPS dwelling block when subjected to a range of overpressures. Whilst such an approach might theoretically provide a basis for determining the probability of failure for a range of accidental loads generated by a variety of explosion sources, considerable effort would no doubt be required to develop an acceptable methodology.
9.4 ISSUES ASSOCIATED WITH PIPED GAS SUPPLIES

9.4.1 Introduction
This section introduces a number of issues which have been voiced as factors which are or might be associated with piped gas supplies and associated appliances.

9.4.2 Ability to mobilise alternative load paths when piped gas is present
Section 9.2.5 previously discussed the ability to mobilise alternative load paths in relation to the requirements described in Section 5 of Approved Document A – Structure.

When considering the possible behaviour of an LPS dwelling block in the event of a very severe piped gas explosion (adopting a structural assessment overpressure criterion of 34 kN/m²), it is possible that multiple sets of opposing wall panels or floor slabs or some combination of adjacent wall panels and floor slabs could be lost or badly damaged simultaneously. The extent of this damage may be greater than anticipated in Section 5 of Approved Document A – Structure.

There are potential implications for LPS dwelling blocks constructed after 1968. Theoretically these would have been designed/checked on the basis that a nominal blocks constructed after 1968. Theoretically these would have been designed/checked on the basis that a nominal

9.4.3 Safety issues relating to piped gas supplies and associated appliances
Various questions have been posed and issues raised in relation to the presence of a piped gas supply and the use of gas appliances within LPS dwelling blocks. These points raised include:

1. Ways of undertaking qualitative and quantitative evaluations of the degree of risk of an explosion with respect to the routing of mains gas supply pipework.

2. The role of passive and active ventilation of gas appliances and enclosed spaces/volumes in reducing the probability of occurrence of an internal explosion and how this should impinge on the structural assessment process.

3. The criticality of the positioning of gas fired domestic and ‘district’ boilers within an LPS dwelling block and whether the positioning of the ‘appliances’ can be used as a basis to justify the adoption of the 17 kN/m² assessment criterion, as opposed to the 34 kN/m² value.

These issues were often raised in relation to the positioning of the gas apparatus being considered. In the case of a gas boiler serving an entire LPS dwelling block, the propositions put forward included locating it in a separate lightweight building on the roof of the LPS dwelling block (ie notionally not within the LPS dwelling block) or within an in-situ concrete podium or basement structure (ie notionally not within the LPS dwelling block).

In the case of domestic boilers, the propositions put forward included locating them on balconies located outside each flat so that they were physically located outside the envelope of the LPS dwelling block (ie in the atmosphere). Such considerations would need to recognise that in addition to the implications arising from individual items of gas apparatus (eg a boiler) that gas leaks can occur anywhere in a piping run.

4. Whether the strict implementation of regular (commonly annual) boiler/pipework servicing contracts and associated visual checks, in combination with the use of safety cut-off valves used as part of a management strategy involving ‘active’ risk reduction, as opposed to the ‘passive’ actions (ie strengthening of joints and panels, piped gas removal, etc.), is acceptable practice and produces an adequate reduction in the degree of risk to building residents and users.

A comparison might be the provision of passive fire protection around structural columns and beams which, whilst not stopping a fire from developing, is intended to prevent early structural failure allowing people to safely evacuate a building; with active fire prevention through the provision of water sprinklers, which are intended to prevent a significant fire from developing.

In relation to Point 1, for example, a mains gas supply pipe to a single roof mounted boiler providing hot water to all the flats could be routed either (a) externally by fixing the gas pipe to the external face of the cladding panels (ie in the atmosphere), or (b) internally via internal service ducts. In both cases it might be argued that the LPS dwelling block contains a piped gas supply. If so, it would be necessary for the LPS block to be assessed against the 34 kN/m² overpressure criterion. However, the probability of occurrence of an internal gas explosion caused by leaking gas would be significantly less under option (a) than it potentially could be under option (b).

Under option (a) the risk of a significant explosion occurring within an LPS dwelling block is likely to be extremely low as any gas that may leak from external pipework will be vented directly to atmosphere. Therefore the chance of a build-up of a significant volume of an explosive mixture of gas within the block would be expected to be extremely low. However, with option (b) there would be the potential for a build up of an explosive mixture of gas within the block.

In this situation the chance of a build up of an explosive mixture of gas could be reduced by actively ventilating the service ducts (Point 2). Alternatively, a gas detector could be installed within the internal service duct system to trigger an alarm (and some following action) should a build up of gas be detected. In both of these approaches there is the question of the reliability of the apparatus installed to control the risk of a build up of an explosive mixture of gas within the block. Should this
appreciable venting occurring and which in turn could potentially have the effect of increasing the magnitude of the internal overpressure created during an internal explosion.

Similar issues have arisen in respect of dedicated domestic wall mounted gas boilers (Point 3 above). There have been suggestions that these boilers could be mounted on an external wall to the kitchen (using a balanced flue installation), within a central cupboard or in a vented enclosure on an open balcony. Each of these involves very different circumstances and a different likelihood of an internal gas explosion. Again this raises the issue of passive and active measures in reducing the probability of occurrence of an internal explosion and how this should impinge on the structural assessment process.

Another scenario that has been raised is where a communal ‘district’ boiler is housed within a cast in-situ reinforced concrete ground floor podium vented directly to the atmosphere (Point 3). Furthermore the boiler may be serviced regularly (annually) with automatic safety cut off valves fitted to the gas supply pipework (Point 4), with the hot water being delivered to each flat via distribution pipework. Even though the major portion of the LPS dwelling block does not contain a piped gas supply, under current guidance the block would be assessed against the 34 kN/m² overpressure criterion. Clearly one concern in this instance would be the nature of the damage which might occur to the in-situ podium and the implications this might have in terms of potential loss of support to the LPS construction above. However, in these circumstances the question has been posed as to what the likelihood is that an explosive mixture of gas could build up in the LPS construction above the in-situ podium from a leak in the piped gas supply and how this could be taken into account in the structural assessment process.

One approach that may be able to take account of the type of factors noted above in the structural assessment of an LPS dwelling block is a probabilistic methodology. However considerable effort would no doubt be required to develop an acceptable methodology.
**10 OTHER CONSIDERATIONS**

### 10.1 INTRODUCTION

There are a number of additional matters which should also be considered as elements of the overall through-life management of LPS dwelling blocks. The considerations reviewed here concern:

- Development of strategy for the through-life durability management of an LPS dwelling block
- The durability/future service life of the components forming an LPS dwelling block.
- Structural assessment after a severe fire.
- Demolition of an LPS dwelling block.

### 10.2 STRATEGY FOR THE THROUGH-LIFE DURABILITY MANAGEMENT OF AN LPS DWELLING BLOCK

The process of durability related investigation and assessment described in the following sections should identify the most likely deterioration mechanisms and influencing factors, and these should form the most significant threats to the future durability of a dwelling block. The process should, therefore, identify which parts of the building are most likely to deteriorate, such as the external concrete cladding, in-situ joints in flank walls and so on. However, considerations should also be given to related elements of the building fabric, such as sealants, windows, roofing materials and the like, which can have a profound impact upon the through-life cost of owning and caring for an LPS dwelling block.

The following is a suggested durability inspection and monitoring regime, which is a development of the original principles set down in BRE Report BR 107[2].

- Visual inspections at a period not exceeding five years, but preferably this should be driven by risk-based inspection principles with more frequent inspections of particular components/actions if this is considered appropriate by durability/deterioration considerations (eg if there could be a hazard caused by spalling concrete, etc.). Aspects of the external visual inspections are expected to be made at close range (ie touching distance).
- Inspection and assessment of the condition of selected in-situ joints likely to be at higher risk of deterioration or affected by rain penetration/dampness into the outer envelope (eg flank wall joints) and thereby at risk of deterioration due to reinforcement corrosion at a period not exceeding 15 years, but preferably this should be driven by risk-based inspection principles.
- These activities should be supported by associated durability related testing and sampling of selected concrete components at a period not exceeding 15 years applied more generally to the block, but preferably this should be driven by risk-based principles.
- It is necessary to make a prognosis of the influence of future deterioration upon the structural adequacy of an LPS dwelling block, particularly in relation to the corrosion of reinforcement. This should be undertaken following the durability related testing and sampling of selected concrete components described above.

BRE Report BR 107[2] also recommended that a technical log be kept by the owning organisation for each LPS dwelling block. This was intended to contain technical information relating to the particular LPS dwelling block; such as construction drawings and specifications, details of any surveys and inspections carried out, any maintenance and remedial works undertaken, and so on. The goal was that all pertinent data and information relevant to the through-life management of the particular LPS dwelling block would be brought into a single repository. With modern IT systems this does not necessarily mean having all the relevant documents in a single ‘file’, but rather that they should be accessible from a single interface. It is not known how commonly this recommendation is complied with, but the matter is expected to become increasingly important as the buildings age further. It should not be overlooked that many LPS dwelling block are now over 40 years old and that a growing number will show ever greater signs of ‘wear and tear’ and deterioration of various elements as time passes, if they have not already done so.

### 10.3 DURABILITY OF THE CONCRETE COMPONENTS FORMING AN LPS DWELLING BLOCK

#### 10.3.1 Introduction

It is important to recognise that a concrete structure is generally ‘silent’ about how environmental processes and developing deterioration are affecting it. These may not be apparent to observers until deterioration has become advanced and mid- or later-stage damage has occurred which have visible manifestations, such as cracking, delamination and spalling of concrete. This is because the environmental mechanisms involved typically work by
slowly altering the internal chemical environment within the surface (cover) zone of the concrete, potentially over decades. Under appropriate environmental conditions, these changed circumstances then permit deterioration (e.g., corrosion of reinforcement) to occur that eventually leads to the development of directly observable damage (e.g., cracking and spalling of concrete). As such there is a limited relationship between:

- what unaided visual observations can establish,
- the apparent current condition of the structure and its component parts, and
- the potential consequences of the unseen / invisible changes which have taken place or are taking place within the concrete.

The principal mechanism of deterioration which is likely to affect the precast and in-situ concrete components forming the structure and external cladding of an LPS dwelling block is reinforcement corrosion. Carbonation of the concrete removes the ability of the concrete to protect the embedded reinforcing bars against corrosion. The presence of chloride ions within the concrete, which may either have been cast into the concrete at the time of construction or have subsequently ingressed into the concrete (e.g., from chlorides in a marine environment or from de-icing salts), promotes corrosion of the reinforcement and other embedded metals especially carbon steel.

There are four main mechanisms of deterioration that may potentially affect conventional steel reinforcement:

Type A: Carbonation of the concrete, without chlorides.
Type B: Cast-in chlorides, without carbonation.
Type C: Ingressed chlorides, without carbonation.
Type D: Chlorides (either cast-in or ingressed) and carbonation acting in combination.

BRE Digest 444: Parts 1 to 3[39–41] provides guidance upon the corrosion of steel in concrete.

Part 1 explains the physical, chemical and electrochemical processes involved in the deterioration of reinforced concrete by corrosion.

Part 2 provides concise guidance on the format for investigations of corrosion of steel in concrete, the techniques employed and how this can lead to a prognosis for the future performance of existing reinforced concrete structures.

Part 3 describes the repair and protection of concrete structures subject to corrosion damage, or which are expected to need measures to minimise future damage or deterioration.

Other guidance is provided by:

- BRE Digest 455: Corrosion of steel in concrete: Service life design and prediction[42].
- BRE Digest 491: Corrosion of steel in concrete: A review of the effect of humidity[43].

BRE Report BR 254: Repair and maintenance of reinforced concrete[45].

Concrete Bridge Development Group Technical Report No 2: Guide to testing and monitoring of concrete structures[46].

In respect of the through-life management of concrete structures, proactive monitoring and management of concrete structures allows for early detection of deterioration processes and facilitates prompt intervention to extend their useful life and the minimisation of whole-life cost. Conversely, a reactive management approach would for example allow the development of cracking or spalling in the concrete to be caused by reinforcement corrosion before a response or intervention was made.

The application of an anti-carbonation coating to the surface of concrete component affected by carbonation is an example of the proactive approach when it is undertaken to delay the onset and propagation of corrosion of the embedded reinforcing bars. It is an example of a preventive measure undertaken to proactively seek to reduce future maintenance and repair problems, reduce future disruption, whole-life cost, etc.

Other durability sensitive situations that could potentially arise are the absence of dry pack at the base of a wall panel or the absence of the in-situ concrete fill between a floor element and the top of the adjacent (supporting) wall panel – see Figures 19–22 above. In these situations the upper wall panel would be supported on the wall panel levelling bolts, which would therefore be carrying all the vertical loads in the wall at that location. Should such severe deficiencies be discovered, it would be necessary to make a remedial intervention to rectify the problem.

Whilst the occurrence of these extreme situations is considered to be unlikely, they are likely to mean that the wall panel levelling bolts had been left without the protection that might have been given by a surrounding cementitious material. In the case of the dry pack mortar, it is anticipated that the durability implications would be small as it is likely that even ‘good’ dry pack mortar would be expected to have been fully carbonated after a few years in service. As such it would not have helped maintain the embedded steel in a passive state. Thus it would be assumed that the dry pack mortar would not provide any appreciable corrosion protection to the wall panel levelling bolts.

Conversely, the in-situ concrete fill between a floor element and the top of the adjacent (supporting) wall panel would have been expected to maintain the embedded steel in a passive state, thereby giving corrosion protection to that part of the wall panel levelling bolts located in the in-situ concrete fill.

Once depassivated, corrosion of the wall panel levelling bolts would be controlled primarily by the ‘time of wetness’, which is the period of time when there is sufficient moisture available on the surface of the steel to permit corrosion to proceed. The overall severity of corrosion at a particular location would be the accumulation of the corrosion damage occurring within the individual periods of corrosion activity.
Thus the total ‘time of wetness’ would govern the total
degree of corrosion which had occurred; starting from
the time when the levelling bolts became susceptible to
corrosion (ie after any initial corrosion protection became
ineffective).

Moisture might reach the wall panel levelling bolts in
the external façade via a number of routes, such as from
the surface directly through the cracks in the concrete
cladding, through joints between wall panels and from
condensation occurring within the concrete cladding. The
availability of moisture would be expected to fluctuate
with time due to seasonal differences in weather. The
issue of wind driven rainwater penetration should
be considered. Temperature would also have had an
influence upon the rate at which any corrosion reactions
had progressed.

Clearly in the structural assessment process it is necessary
to understand the potential influence of deterioration upon
the structural adequacy of LPS dwelling block.

10.3.2 Passivation of steel in concrete
If the alkalinity of the electrolyte surrounding a piece of
steel is greater than about pH 10.5 a very thin uniform
protective surface oxide layer forms on the surface of the
metal. The microscopic oxide layer which is formed on
the steel surface changes the electrochemical potential of
the steel towards a more noble condition. This oxide layer
constitutes a ‘passive’ film, which impedes the dissolution
of iron. Under these circumstances the rate of corrosion is
insignificant.

In Portland cement concretes the pH is maintained at
levels of at least 12.6 due to the presence of significant
amounts of Ca(OH)₂ (the mineral portlandite, otherwise
known as calcium hydroxide) as well as sodium and/or
potassium salts which can increase the pH further, up to
say 13.5.

Reinforcement corrosion can occur when the
protective oxide layer that is normally formed on the
surface of the reinforcement is lost. There are two main
causes for this in reinforced concrete:

- A reduction in the pH of the pore solution as a result
  of leaching in the vicinity of cracks or carbonation of
  the surface concrete due to the ingress of atmospheric
  CO₂.
- The ingress of chlorides from an external source or the
  presence/release of chlorides from an internal source
  (ie chlorides introduced at the time of casting the
  concrete).

Figures 23 and 24 present examples of chloride induced
corrosion of reinforcing bars embedded in concrete.
Figure 23 shows the corrosion of reinforcing bar located
in an LPS dwelling block floor due to cast-in chloride.
This particular floor was subject to an overpressure load
test by BRE to establish the ultimate load capacity of the
LPS components and the associated joints when acting
together as a three-dimensional structural system – see
Appendix G for details. Figure 24 illustrates the nature of
localised pitting corrosion of a reinforcing bar (see ringed
areas) which can be caused by ingressed chlorides.

10.3.3 Depassivation of steel in concrete by
carbonation
A chemical change within the concrete that increases the
risk of reinforcement corrosion is the reaction between
calcium compounds, primarily Ca(OH)₂, and atmospheric
CO₂. This leads to carbonation of the concrete, causing a
decrease in the alkalinity of the pore solution to below pH
9 once the concrete is fully carbonated. The reduction in
the alkalinity causes the ‘passive’ surface film on the steel
to breakdown and, in the presence of sufficient moisture
and oxygen, the reinforcement to start to corrode. Thus
the reduction in the alkalinity of the concrete removes
its ability to protect the reinforcement against corrosion
— see Figures 25 and 26. Note: When phenolphthalein
test indicator is sprayed onto a freshly cracked concrete
surface a magenta coloration develops where the
cement remains in an uncarbonated condition (pH > 9).
Carbonated concrete does not react and retains its
previous natural colour, which is typically grey.
In Figures 25 and 26 the magenta coloration is shown
as grey shading.

The position of the carbonation front, as shown by
the phenolphthalein test indicator, is taken to provide a
basis for defining when corrosion initiation at the front
face of the reinforcing steel could occur, which allows an
estimate to be made of the age of the structure when this
situation will occur (subject to the availability of sufficient
moisture and oxygen.

Different concrete surfaces, such as the soffit and
sides of a concrete beam, may experience different rates
of carbonation depending upon local environmental
conditions. Concrete elements which are exposed
to a dry internal environment will carbonate at a
significantly different rate to those exposed to an external
environment. The rate of carbonation in an external

environment will depend on the characteristics of the
microclimate to which a particular surface is exposed,
such as whether it is directly wetted by rain or it is in a
sheltered situation.

The rate at which the carbonation front progresses
from the surface of the concrete slows with time and
this relationship is typically portrayed by a simplified
relationship involving the square root of time since
construction. The modelling of the increase in
carbonation depth with time is discussed in BRE Report
254[44].

Figure 27 illustrates the influence of differences in the
cement cover depth, showing how the time required
for the carbonation front to reach a given depth changes
with cover depth. In this example, the nominal cover
depth of 40 mm (refer Point A in Figure 27) is assumed
to give a period of 64 years before the carbonation
front reaches the embedded reinforcement, assuming
uncracked concrete. On the same basis, an increase in
the cover depth to 50 mm (Point B in Figure 27) increases
the time for the carbonation front to reach the embedded
reinforcement to 100 years. Therefore in this example, an
increase in the cover depth from 40 mm to 50 mm (1.25
times) increases the time required for the carbonation
front to reach the embedded reinforcement by 1.56 times
(ie 1.25 squared). However, a reduction in the achieved
cover depth to 25 mm (Point C in Figure 27) decreases
the time for the carbonation front to reach the embedded
reinforcement in uncracked concrete to 25 years.

On the same topic, Figure 28 presents related data
upon the depth of cover and the depth of carbonation
measured during site testing as paired sets at individual
locations on a reinforced concrete structure. This
approach, which considers the ratio between the
depth of cover and the depth of carbonation, is
commonly used to examine implications for the
future durability of reinforced concrete components.
It compares the estimated depth of cover values with
the currently measured depth of carbonation values
and makes prognosis about future durability as the
depth of carbonation increases with time (on the basis
of the relationship shown in Figure 27). When the
depth of carbonation exceeds the depth of cover, the
reinforcement is assumed to be depassivated and at risk
of corrosion, assuming sufficient moisture and oxygen are
available to fuel the corrosion process.

An alternative approach is to treat the data on depth
of cover and depth of carbonation as unrelated variables
and to treat them statistically on this basis to obtain an
overview of current condition and to make a prognosis on
the future durability of the structure.

10.3.4 Depassivation of steel in concrete by
chlorides

The presence of oxygen and sufficient quantities of
free chloride ions in the pore water of concrete can
produce reinforcement corrosion even in highly alkaline
conditions. Chlorides may be present in the components
used to make the concrete at the time of casting (cast-in
chloride) or enter the concrete from the environment
after placing (ingressed chlorides). In the case of LPS
depending on the chloride concentration in the pore water. However, not all the chlorides can be bound as equilibrium has to exist between the bound chlorides and the free chloride ions in the pore water. Only the free chloride ions are relevant to the corrosion of the reinforcement. It is important to note, therefore, that after carbonation of concrete bound chlorides are released, so that the chloride content in the

Figure 27: Carbonation depth versus time and implications of different concrete cover depths on the notional service life of a concrete structure.

Figure 28: Comparison of estimated depth of cover values with measured depth of carbonation values and prognoses of future behaviour as the depth of carbonation increases with time.

dwelling blocks in the UK the principal external sources of chloride are likely to be airborne salt in marine/coastal conditions and possibly also de-icing salts applied to walkways and the like.

Chloride ingress is due to either diffusion, taking place in totally or partially water-filled pores, or capillary suction of water containing chlorides. Cement has a certain chemical and physical binding capacity for chloride ions,
pore water, and consequently the risk of corrosion due to chlorides, will increase considerably. The critical chloride concentration at which corrosion will occur depends on many parameters. It has typically been taken to be in the order of 0.4% chloride ion by weight of cement.

BRE Digest 444: Part 2\textsuperscript{46}, recognising that the incidence of corrosion is probabilistic and that the risk of corrosion increases with time, gives guidance for cast-in chlorides in concrete structures representing three different age groups: 25, 40 and a projection for 60 years old. Based on the chloride ion content (% by weight of cement), the risk of corrosion is estimated in relation to the environment (damp or dry) and the alkalinity of the concrete (carbonated or uncarbonated). This classification of the risk of corrosion in these conditions is reproduced as Figure 29. This shows how the risk of corrosion increases with time, chloride content, in damp conditions and with carbonation.

BRE Digest 444: Part 2\textsuperscript{46} also gives guidance upon the risk of steel reinforcement corrosion associated with ingressed chlorides, but this issue is expected to be mainly applicable to the relatively small proportion of LPS dwelling blocks situated in a marine/coastal environment. Ingressed chlorides might potentially be present in external cladding panels and within in-situ joints at locations such as flank walls, especially if rain penetration could assist the migration of the chlorides.

The source of chloride, that is whether the chloride was cast-in at the time of construction or subsequently ingressed into the concrete, is usually determined by establishing the variation of chloride concentration with depth into the concrete. This is typically done from dust samples drilled incrementally from the surface of the concrete\textsuperscript{18}. Cast-in chloride will have a reasonably uniform profile with depth into the concrete (ie effectively a constant value in particular component, but with the magnitude varying from component to component), whereas the concentration of chloride which has ingressed into the concrete will decrease with greater depth into the concrete. Cast-in chloride can be present in any precast or in-situ concrete component in an LPS dwelling block. It may be present only in particular types of concrete component (perhaps depending upon manufacturing schedules) or possibly just occur at certain floor levels in a block. Experience has shown that sampling rates of 10% have been required to establish the pattern of chloride contamination in an LPS dwelling block.

Figure 30 presents data upon total chloride ion content by weight of cement from an investigation upon an LPS dwelling block, with the results obtained differentiated by the type of member and locations sampled. It will be seen that this approach enables reinforced components at a higher risk of chloride induced corrosion to be identified; such as the precast stair flights in this example. In this case the chloride ion was present in the form of cast-in chlorides.

10.3.5 Durability investigation and assessment procedure

Figure 31 provides an overview of the main process steps for durability inspection and assessment as part of the through-life management of an LPS dwelling block, together with making an intervention to extend its useful service life. This shows the preparatory durability inspection and assessment activities, essential steps involved prior to making an effective and appropriate protection and repair intervention.

BRE Digest 444: Part 2\textsuperscript{46} also gives guidance upon durability investigation and assessment procedures that may be adopted, presenting a flow chart which provides a summary of the decision making process for investigating and assessing corrosion of reinforced concrete. This gives more detail of the steps involved prior to making a durability related intervention. These considerations are based on the four main types of deterioration that affect conventional steel reinforcement noted above (reproduced below), but they also recognise the possible contribution of other deterioration mechanisms that may also be acting:

Type A: Carbonation of the concrete, without chlorides.
Type B: Cast-in chlorides, without carbonation.
Type C: Ingessed chlorides, without carbonation.
Type D: Chlorides (either cast-in or ingressed) and carbonation acting in combination.

A significant proportion of durability related investigations are of a diagnostic nature in response to an observed or reported defect such as cracking or spalling of concrete. It is recommended that initial visual inspections are carried out before durability related testing as these can provide information on defects or deformations. The priority, after an initial inspection and before a programme of detailed durability related testing is undertaken, is to assess the magnitude of the problem and take a view on its likely primary cause.

Having undertaken an inspection and associated durability related testing, the component or the overall structure can be assigned to a corrosion risk category (Figure 32) based upon the cast-in chloride content, the alkalinity of the concrete (carbonated or uncarbonated), environmental conditions (dry or damp) and age of the structure. Figure 32 provides an indication of possible response actions for the future management of the component or the overall structure of the LPS dwelling block for durability related issues. Concrete Bridge Development Group Technical Guide No 2\textsuperscript{45} provides further information and guidance upon durability related testing and monitoring of concrete structures.

\textsuperscript{18} Situations may exist where a ‘facing’ concrete layer to some types of cladding panel are formed from a different concrete mix to that used in the ‘backing’ layer of concrete forming the main body of the panel. Since chloride additives may have been used in one mix and not the other, care must be taken when undertaking sampling. In such situations a decision has to be made on the depth of sampling to avoid cross contamination in the dust samples representing the different concretes.
Figure 29: Estimated risk of steel reinforcement corrosion associated with carbonation, cast-in chloride content and environmental conditions.
Figure 30: Analysis of chloride ion content of LPS dwelling block concrete components

Figure 31: Overview of main process steps for inspection, assessment, management and making an intervention upon a concrete structure to extend its useful service life

| Inspection and assessment to establish condition and nature of any problems or deterioration |
| Evaluate current durability related condition of structure |
| Prediction of future condition/progress of deterioration with time |
| Undertake re-evaluation against structural design or revised performance criteria |
| Confirm through-life management option for structure: prevention/repair of deterioration |
| Specify and execute chosen preventive or remedial intervention method(s) |
| Set out future inspection, monitoring and maintenance requirements |

The above describes steps in the process of inspection and assessment of a concrete structure, the choice of a through-life management option and preventive/remedial intervention options to address established deterioration mechanisms or to delay their development/propagation.

Key:  
- Process steps for inspection and assessment  
- Activities addressed by EN 1504 Part 9 methodology (see Figure 33)
Options for managing the structure include the following technical steps:
- Do nothing and monitor for a certain time.
- Re-analyse structural capability.
- Prevent or reduce further deterioration.
- Improve, strengthen or refurbish all or part.
- Reconstruct all or part.
- Demolish all or part.

Principles are the concepts underlying the method of concrete protection and repair intervention. These relate to the nature of the defect or problem which exists. They are grouped under the two headings given below. It is necessary to choose a Principle appropriate to the Option chosen.

Defects in concrete:
1. Protection against ingress
2. Moisture control
3. Concrete restoration
4. Structural strengthening
5. Physical resistance
6. Resistance to chemicals

Reinforcement corrosion:
7. Preserving or restoring passivity
8. Increasing resistivity
9. Cathodic control
10. Cathodic protection
11. Control of anodic areas
Methods are the means of achieving the objective of the chosen Principle, which is the concrete protection and repair method to be used for the intervention; such as undertaking a patch repair by applying mortar by hand, or by applying sprayed concrete or mortar.

A number of different approaches are included under the broad banner of concrete protection and repair intervention options. The protection actions include the application of some form of surface protection or impregnation, electrochemical re-alkalisation, electrochemical chloride extraction and cathodic prevention/protection. Repair actions include concrete restoration, surface protection or impregnations, overlays or other forms of barrier, as well as cathodic protection. It will be noted that some techniques can be utilised as both a protection and as a repair, depending upon circumstances. However, electrochemical treatments are not, however, best suited for use on large panel construction due to the discontinuous nature of the reinforcement.

10.4 STRUCTURAL ASSESSMENT AFTER A SEVERE FIRE

10.4.1 Introduction
Concrete has good fire resistance and concrete structures are generally capable of being repaired even after a severe fire. However, LPS dwelling blocks are somewhat different from typical concrete structures because of the potential issues associated with progressive collapse. Consideration needs to be given to the potential effects that the fire may have had upon the stability of the affected part of the block and on its collapse resistance/ability to mobilise alternative load paths in the event of local damage or failure occurring.

10.4.2 Effects of heating and subsequent cooling
The heat produced during a fire affects the structure of an LPS dwelling block in two main ways. Firstly, there is the impact associated with the direct heating of the materials in the immediate vicinity of the fire. This can cause varying degrees of change in the material properties in the elements concerned, depending upon the temperatures reached. As these elements heat up they also seek to expand and this will tend to cause a combination of flexural bending and axial movement. Clearly the element(s) will want to extend as it heats up and will seek to contract when it subsequently cools down after the fire. There may also be significant post-fire deformation of the elements and altered material properties remaining once the fire is over.

The second effect is the impact of these movements, or their restraint, that arises in the adjacent elements/parts of the surrounding structure. The forces and movements created can be very significant and can create appreciable damage or potentially collapse in other parts of a structure remote from the seat of the fire.

The outer layers of the concrete element being heated by the fire will become hot. However, because concrete is a relatively poor conductor of heat, the temperature of deeper layers will increase only slowly. The differential heating sets up stresses within the concrete element. Heating drives off free moisture residing within the pores in the concrete, potentially creating significant internal pressures. High temperature changes the mechanical properties of concrete (eg compressive and tensile strength, elastic modulus, etc.) by various mechanisms including breaking down the chemical bonds created within the concrete matrix during the hydration process and reducing the bond between the matrix and the aggregate particles. Damage can also occur to the aggregate particles, depending upon their mineralogy.

When concrete is exposed to high temperature it undergoes colour changes and this characteristic has been used to estimate the temperature attained by the concrete. Above a temperature of around 300°C concrete commonly takes on a pinkish hue; however, care has to be taken in the interpretation of colour as other factors can affect the colour. These changes are important because they coincide with the onset of a significant loss in concrete strength and reduction in bond between the reinforcement and the concrete. It is now recommended assessment practice to discount the residual strength of any concrete heated to more than (circa) 300°C. The temperature attained by the concrete is also relevant to judgements about the potential influence that the heating will have had upon the properties and performance of any embedded reinforcement.

10.4.3 Investigation and structural assessment procedure
An investigation and structural assessment will undoubtedly be required after a severe fire in an LPS dwelling block. It is suggested that the following general approach be adopted:

- Study archive information and, if necessary, investigate various potential archive sources.
- Undertake site and laboratory evaluation of materials directly affected by high temperatures (ie in vicinity of seat of fire).
- Undertake site evaluation of elements and joints between components affected by movements and thermal expansions (ie remote from seat of fire).
- Undertake site investigations to establish the form and standard of construction employed within block – with the need depending on the availability of archive information.
- Make a structural assessment of the affected parts of the block for normal loads.
- Make a structural assessment of the affected parts of the block for accidental loads.
- Evaluate the findings.
- Define repair objectives and select potential remedial options.

These activities might be represented by four main tasks, namely:

Task 1: Preliminary activities including archive search and review, preparation of survey drawings for LPS dwelling block concerned, mapping of cracks, etc.
Figure 33: Overview of EN 1504-9

Key:
- Activities outside EN1504 methodology (necessary preparatory activities – see Figure 31)
- Activity or decision step within EN1504 methodology
- Related information and influencing factors
Task 2: Forensic investigation, site sampling of materials, inspection and assessment of direct and indirect damage caused by fire, including laboratory testing and inspection of samples recovered from site.

Task 3: Associated site investigation to establish the form and standard of construction employed more generally within LPS dwelling block concerned (if necessary).

Task 4: Undertake the necessary assessments and evaluations, together with the reporting of findings and recommendations.

Of particular importance in the circumstances of an LPS dwelling block are the potential effects that the fire may have had upon the stability of the affected part of the block, that is, panels/slabs that are directly or indirectly affected by the heating. Movements induced in the structure can potentially disrupt joints between structural components and cause considerable distortion from the seat of the fire, especially at the flank wall. These effects may adversely affect the collapse resistance of the LPS dwelling block or its ability to mobilise alternative load paths in the event of local damage or failure occurring. Consideration should be given to the effect of the cooling of the structure after the fire, together with the influence of restraint and residual stresses.

### 10.4.4 Procedures for structural repair after a severe fire

Concrete has good fire resistance and concrete structures are generally capable of being repaired even after a severe fire. In the 1980s, Tovey and Crook summarised the information gathered from over 100 fire-damaged structures. They concluded that the structures generally performed well during and after the fire, with most of the structures being successfully repaired and returned to service.


There is a major difference between designing a structure to withstand a fire, that is allowing for safe evacuation and fire fighting, and assessing the extent of damage caused by a fire so that appropriate repair methodologies can be proposed. While designing a structure is concerned with predicting its performance during a future fire event, assessing a structure is concerned with determining its residual strength after such an event.

The emphasis of Technical Report 68 is on methods for assessing a concrete structure following a fire in order to determine the extent of the repairs required. It describes the design approaches used to assess the strength of repaired elements, which are illustrated by design examples, and in accordance with the relevant Structural Eurocodes. The chapter on repairs is more limited than in the previous version of the document as the techniques used are largely common to all concrete repairs, irrespective of the cause of the damage. Technical Report 68 also includes summaries of a number of case studies of the assessment and repair of structures damaged by fire, together with worked examples and historical information on design and material properties given in British Standards and other documents.

Technical Report 68 notes that the person undertaking the assessment needs to be aware when the material properties and calculation methodologies presented in EN 1992, otherwise known as Eurocode 2, may not be applicable to the specific situation, since effects such as cooling of the structure or restraint and residual stresses need consideration after a fire.

### 10.5 DEMOLITION

At the end of its useful life an LPS dwelling block will no doubt either be dismantled or demolished using one of the accepted methodologies employed for this task. However, it should be recognised that, as in all demolition activities, care is required; particularly in respect of the hazards/risks associated with the potential susceptibility of the LPS form of construction to progressive collapse.

It should be noted that there have been a number of incidents where a loss of control has occurred during the process of demolition. One case known to BRE resulted in the progressive collapse of part of a 14 storey LPS dwelling block during demolition, causing damage to a nearby inhabited house. Fortunately there were no reported injuries.

Accordingly, it is advised that risk assessments undertaken for such work consider the sensitivity of the LPS dwelling block to progressive collapse and take account of this in the method of working adopted.

BS 6187 – Code of practice for demolition recognises that a number of demolition methods can be adopted, such as those involving:

- Progressive demolition.
- A deliberate collapse mechanism.
- The deliberate removal of key structural components.

Progressive demolition is a technique which is commonly employed to demolish LPS dwelling blocks. BS 6187 indicates that this process should be considered to be the controlled removal of sections of the structure, whilst retaining the stability of the remaining part and avoiding collapse of the whole part of the building to be demolished. BS 6187 requires that the demolition procedure identifies ‘key structural members’ and recognises this in the method of working adopted.

It needs to be recognised that almost all structural elements/components in an LPS dwelling block are load bearing or contribute to the stability of the loadbearing elements of LPS dwelling block – thus all load bearing elements and stabilising elements (eg floor slabs) should be considered to be ‘key structural members’ for the purposes of demolition. Accordingly, this needs to be reflected in the method of working adopted for the demolition of an LPS dwelling block. Thus in the context of the demolition of an LPS dwelling block the term ‘key
structural members’ would be taken to be equivalent to the term ‘key elements’ discussed previously.

Guidance upon matters relating to demolition and refurbishment is given in a number of publications including the following:

- BS 6187: *Code of practice for demolition*\(^{[3]}\).
- CIRIA Report No. 133, *A guide to the management of building refurbishment*\(^{[3]}\).
- HSE Research Report 204, *Refurbishment involving demolition and structural instability*\(^{[3]}\).
11 SUMMARY – REQUIREMENTS, HAZARDS, RISKS AND PERFORMANCE

11.1 ASSESSMENT CRITERIA

Diagram 24 of Approved Document A – Structure\[9\] requires that the extent of a structural collapse should be limited. In the case of an LPS dwelling block the defined limits of the area at risk of structural collapse would effectively limit damage to that originating from an incident in a single dwelling in the block.

Almost all structural elements in an LPS dwelling block are load bearing or contribute to the stability of the loadbearing elements of LPS dwelling block – thus all load bearing elements and stabilising elements (eg floor slabs) are considered to be ‘key elements’ as defined in Approved Document A – Structure\[9\].

BRE Report 107\[2\] indicates that:

- Robustness can be considered to be demonstrated in an LPS dwelling block, or indeed in any other type of building, if there is adequate provision of horizontal or vertical ties (in terms of their spacing and capacity) to comply with the current requirements set down in the codes and standards quoted in Approved Document A – Structure\[9\] as meeting the requirement set down in the Building Regulations.
- In the absence of appropriate tying, stability will be maintained if an LPS dwelling block is able to resist the accidental loads from an internal gas explosion (ie the LPS dwelling block has an adequate collapse resistance for the foreseeable loads)\[19\].

Research by Ellis and Currie\[26\] indicates that an overpressure of 17 kN/m$^2$ is a reasonable value to use for the structural assessment of LPS dwelling blocks or other buildings without a piped gas supply, encompassing the overpressures which are likely to be generated within such buildings during all accidental (non-piped) gas explosions. The largest of these incidents are classed as ‘severe’ gas explosions.

Buildings which contain a piped gas supply (to any part of the building) may be subjected to larger overpressures/more complex and violent accidental explosions. These are classed as ‘very severe’ gas explosions, which might potentially produce overpressures in excess of the stipulated 34 kN/m$^2$ pressure used for structural design or assessment purposes.

‘Very severe’ accidental gas explosions have historically occurred more frequently in buildings with a basement. In some cases these buildings do not have a piped gas supply and the gas which caused the explosion was found to have entered the building from an external source (such as a leaking gas supply pipe) and accumulated in a poorly ventilated space prior to ignition.

Thus, LPS dwelling blocks with a basement or a piped gas supply need to be differentiated from those without these characteristics.

It is proposed that structural assessments of LPS dwelling blocks be undertaken using accidental overpressure values of 17 kN/m$^2$ and 34 kN/m$^2$ for the circumstances as defined below\[20\].

A. An LPS dwelling block with a piped gas supply within or to any part of the building: an assessment overpressure of 34 kN/m$^2$ should be used generally throughout the building. The overpressure should be applied simultaneously to all surfaces of the single room/bounding enclosure within which the explosion (deflagration) is considered to occur. Alternatively, the approach used for the design of ‘key elements’ in new buildings may be adopted\[21\].

B. An LPS dwelling block with a basement: an assessment overpressure of 34 kN/m$^2$ should be used in the basement and in any other zone where an explosive mixture of gas might accumulate (potentially from an external source). The overpressure should be applied simultaneously to all surfaces of the single room/bounding enclosure within which the explosion (deflagration) is considered to occur. Alternatively, the approach used for the design of ‘key elements’ in new buildings may be adopted\[21\].

C. An LPS dwelling block without a basement and without a piped gas supply to any part of the building: an assessment overpressure of 17 kN/m$^2$ should be used. The overpressure should be applied simultaneously to all surfaces of the single room/bounding enclosure within which the explosion (deflagration) is considered to occur. Alternatively, the

---

\[9\] Previously this approach was designated as Method B: Ability to resist the forces associated with an internal gas explosion as set down in MHLG Circular 62/68\[4\]

\[19\] However, for reliability based assessments it is necessary to recognise and take into account that the 17 kN/m$^2$ (non-piped gas situations) and 34 kN/m$^2$ (piped gas situations) assessment criteria values may be exceeded – indeed it is necessary to estimate the probability distribution function of the complete range of possible overpressures for the respective situations.

\[20\] Refer Clauses 5.1d and 5.3 of Approved Document A – Structure\[9\]
11.2 RISKS

Statistical analysis and historical records of accidental loads or actions experienced in the overall UK population of buildings as described in Appendices B and C indicate that the following risk environment may be applicable to LPS dwelling blocks in general:

- For internal gas explosions (deflagrations) involving cylinder gas or other gaseous substances:
  - Severe explosion – the probability of occurrence during the remaining life of an LPS dwelling block is estimated to be less than 1 in 1000 (ie $10^{-3}$).
  - The probability of a severe explosion occurring in a single stack (ie flats located immediately above/below each other) of dwellings (assuming a maximum of 25 storeys) is estimated to be less than $2.5 \times 10^{-4}$ per annum.
  - The probability of a severe explosion occurring within the upper five storeys of an LPS dwelling block, where the possibility of initiating a progressive collapse in a single stack of dwellings is greatest, is estimated to be less than $0.5 \times 10^{-4}$ per annum.
  - On the basis of the assumption that 20% of severe explosions might hypothetically create accidental loads that exceed the actual collapse resistance of LPS dwelling block elements; the probability of a progressive collapse in an LPS dwelling block is estimated to be less than $0.1 \times 10^{-4}$ per annum.
  - Historically the level of safety achieved in the overall population of LPS dwelling blocks appears to be comparable with the levels expected for the overall population of buildings within the UK.
  - The ‘upper bound’ number of fatalities caused by a progressive collapse in a single stack of dwellings resulting from a severe explosion in a single LPS dwelling is estimated to be about 60 people.
  - The notional risk of death to an individual associated with a severe explosion involving cylinder gas or other gaseous substances is estimated to be in the order of $0.1 \times 10^{-4}$ per annum.

- For internal gas explosions (deflagrations) in blocks with a piped gas supply or a basement the historic evidence suggests that only these circumstances would conceivably result in a ‘very severe’ explosion. The probability of a ‘very severe’ explosion occurring in a single stack of dwellings (assuming a maximum 25 storeys) is estimated to be less than $0.5 \times 10^{-6}$ per annum.

- External gas explosions (deflagrations) – the historic record suggests that these are probably sufficiently rare events not to require specific consideration. However, their magnitude may be such that collapse of part or all of an LPS dwelling block could occur. Such damage may not be disproportionate.

- Road vehicle impact – in the 20 year survey period no road vehicle impact had caused severe or very severe damage to a UK building with a height of five or more storeys.

- Aircraft impact – the annual probability of an aircraft impact on an LPS dwelling block in the London area is estimated to be less than $1 \times 10^{-4}$. The magnitude of the damage caused may be such that collapse of part or all of an LPS dwelling block could occur. Such damage may not be disproportionate. While such an impact might conceivably create 350 fatalities in one of the largest LPS dwelling blocks (assuming a maximum of 150 dwellings in the block and that the average number of people living in a UK dwelling is 2.3), the causalities would generally be expected to be less than 100 people.

- Fire – the risks to life arising from fires within LPS dwelling blocks are estimated to be significantly larger than those arising from the other accidental hazards discussed above. However, in the vast majority of cases, the deaths and injuries which occur during a fire incident are not associated with structural failure of the building. For example, in 2006 there were a total of 55,800 fires in UK dwellings, which corresponds to an annual total fire occurrence of about two fires per 1000 dwellings. In 2006 fires in UK dwellings resulted in a total of 363 deaths, which equates to about 15 fatalities per year per million dwellings. Thus, the notional risk of death to an individual associated with a fire in a dwelling is in the order of $6.5 \times 10^{-4}$ per annum. This is more than an order of magnitude larger than the risks arising from the other accidental hazards discussed above.

Thus the statistical assessment of the annual probability of death for an individual due to an accidental load or action other than fire (eg an internal gas explosion) is estimated to be below the threshold at which action might be required. On this basis, rationally the risks might again be ‘regarded as insignificant and adequately controlled’.

Considering the number of multiple fatalities arising from an incident such as the progressive collapse of a single stack of dwellings in an LPS dwelling block, plotting these on an F–N curve (see Figures C3 to C5 in Appendix C) suggests that risk levels are sufficiently low for them to be considered to be socially acceptable. The situation was also evaluated using the cost of preventing a fatality. This approach also indicated that the annual probability of an individual fatality is below the threshold at which action might be required. On this basis, rationally the risks might again be ‘regarded as insignificant and adequately controlled’.

---

22 A single stack of dwellings is taken to be the vertical projection of an individual dwelling up the full height of the LPS dwelling block. There are commonly four or six different flats on each storey of a typical UK LPS dwelling block, but sometimes there are significantly more in long ‘slab’ style blocks. For example, the LPS dwelling block floor plan from which Figure 3 (a part floor plan) was extracted has eight flats per storey. Whilst the incidence of a non-piped gas explosion occurring in a single stack of individual rooms is expected to be lower than the figures quoted in the main text above, the historical explosion statistics record does not provide such detail.
However, in light of the historical aspects of the partial collapse of Ronan Point, wider societal and emotive considerations might bring a modified perspective to this deliberation. It is expected that this would effectively require that a lower level of risk be achieved than the $10^{-6}$ criterion suggested in the above discussions. An aspirational risk acceptance criterion for LPS dwelling blocks would be to match the general level of safety achieved in the overall population of UK buildings.

If the past performance of the overall population of LPS dwelling blocks compared with that of the general population of UK buildings; this suggests that the level of achieved safety in the two groups is comparable. This comparable performance is thought to result mainly from the following:

- The special measures put in place for the assessment and management of LPS dwelling blocks following the partial collapse of Ronan Point in 1968.
- The fact that LPS dwelling blocks have a higher collapse resistance than previously acknowledged and indicated by simplified structural assessment calculations.
- General safety improvements that have occurred over the intervening period. The overall number of ‘severe’ and ‘very severe’ accidental explosions within UK buildings has fallen significantly over the years, even though the total number of minor (non-piped) gas explosions has increased noticeably over recent years (believed to be due in part to the change from the use of CFCs to butane as the propellant gas in many aerosols).

### 11.3 COLLAPSE RESISTANCE

On the matter of collapse resistance – the BRE structural tests to the ultimate load condition, which simulated internal overpressure loading, demonstrated that the collapse resistance was sufficiently large in the three LPS dwelling blocks tested to avoid the initiation of failure under typically accidental loads associated with a ‘severe’ explosion arising from cylinder gas or other gaseous substances. These results relate only to LPS dwelling blocks which are without a piped gas supply and accordingly whose adequacy would be assessed on the basis of an accidental internal overpressure value of 17 kN/m².

Thus, in conclusion, the above findings suggest that it is rational for LPS dwelling blocks which are without a piped gas supply to be assessed on the basis of an accidental internal overpressure value of 17 kN/m²; and that this would allow the associated risks to be adequately controlled such that LPS dwelling blocks managed in this way could be considered to be ‘adequately safe’.

The BRE structural tests also demonstrated that some elements within LPS dwelling blocks had the ability to create alternative load paths in certain circumstances following damage or the disruption of the intended load path within the type of LPS dwelling block tested.

Therefore, when undertaking a deterministic structural assessment, the collapse resistance of LPS dwelling block elements should be evaluated for the forces associated with accidental overpressure values of 17 kN/m² (non-piped gas situations) and 34 kN/m² (piped gas situations) - as these values provide reasonable higher limit estimates of the overpressure likely in these situations.

For reliability based assessments, however, it is necessary to recognise and take into account that the 17 kN/m² (non-piped gas situations) and 34 kN/m² (piped gas situations) assessment criteria values may be exceeded – indeed it is necessary to estimate the probability distribution function of the complete range of possible overpressures for the respective situations.

Furthermore, it also needs to be recognised that even adequate collapse resistance, which provides the capability to resist the foreseeable accidental load arising from a ‘severe’ internal gas explosion, does not remove the risk of progressive collapse should a load bearing component (i.e a ‘key element’) fail or if the initiation of the failure of key elements of the structure occurs for some reason.

### 11.4 FUTURE MANAGEMENT

It is recommended that all LPS dwelling blocks should be managed by means of risk assessment procedures in order to seek to minimise the probability of a ‘severe’ or ‘very severe’ explosion in the circumstances defined above or from structural damage caused by other means.

Thus in line with contemporary Building Regulation and HSE philosophy involving risk management concepts (the ALARP/SFARP principle), it is recommended that all LPS dwelling blocks be subject to systematic risk assessment procedures to guide through-life management and associated actions taken, with the goal of:

- Eliminating hazards where practicable.
- Reducing hazards and controlling risks to the structure of these buildings, which are generally at very low levels/acceptable levels, as far as is possible.

This approach to risk reduction and management is sometimes referred to by the acronym ‘ERIC’ which stands for actions to Eliminate, Reduce, Inform about and Control hazards and associated risks.

### 11.5 DURABILITY ASSESSMENT

The principal mechanism of deterioration which is likely to affect the precast and in-situ concrete components forming the structure and external cladding of an LPS dwelling block is reinforcement corrosion. Corrosion might occur as a result of either carbonation of the concrete or from the presence of significant levels of chloride ions within the concrete, which may either have been cast into the concrete at the time of construction or have subsequently ingressed into the concrete (i.e. from chlorides in a marine environment or from de-icing salts).

The durability assessment process should be able to identify which parts of the LPS dwelling block are most likely to deteriorate, such as the external concrete cladding, in-situ joints in flank walls, etc. However, considerations should also be given to related elements...
of the building fabric such as sealants, windows, roofing materials, etc. which can have a profound impact upon the through-life cost of an LPS dwelling block.

A prognosis should be made of the influence of future deterioration upon the structural adequacy of an LPS dwelling block, particularly in relation to the corrosion of reinforcement.

### 11.6 PROPOSED ASSESSMENT/INSPECTION REGIME

The following is a suggested durability inspection and monitoring regime for an LPS dwelling block, which is linked to the periodic structural assessment of the block. This is a development of the original principles set down in BRE Report BR 107[2].

- Visual inspections at a period not exceeding five years, but preferably this should be driven by risk-based inspection principles with more frequent inspections of particular components/actions if this is considered appropriate by durability/deterioration considerations (eg if there could be a hazard caused by spalling concrete). Aspects of the external visual inspections are expected to be made at close range (ie touching distance).

- Inspection and assessment of the condition of selected in-situ joints likely to be at higher risk of deterioration or affected by rain penetration/dampness into the outer envelope (eg flank wall joints) and thereby at risk of deterioration due to reinforcement corrosion at a period not exceeding 15 years, but preferably this should be driven by risk-based inspection principles.

- These activities should be supported by associated durability related testing and sampling of selected concrete components at a period not exceeding 15 years applied more generally to the block, but preferably this should be driven by risk-based principles.

- It is necessary to make a prognosis of the influence of future deterioration upon the structural adequacy of an LPS dwelling block, particularly in relation to the corrosion of reinforcement. This should be undertaken following the durability related testing and sampling of selected concrete components described above.

- Full structural assessment of the LPS dwelling block at a period not exceeding 30 years taking into account the effects of current deterioration and the prognosis for any future deterioration, but again this process should preferably be driven by risk-based principles. Thus, if an LPS dwelling block were constructed in 1965, its first structural assessment should have been carried out in the 1990s (BRE is aware that many were) and its second structural assessment would, therefore, be due no later than about 2025. This would be expected to consider future structural and associated performance in light of any anticipated deterioration in the period up to 2055.

A detailed discussion of the proposed methodology for the assessment of LPS blocks for accidental loading and actions is presented in section 12 of this Handbook.

### 11.7 DEMOLITION

When an LPS dwelling block is to be dismantled or demolished at the end of its useful life, the CDM Regulations[55] require a written method of work, which would be derived from an assessment of risks. It is advised that risk assessments undertaken for such work consider the sensitivity of the LPS dwelling block to progressive collapse and take account of this in the method of working adopted. This is in recognition that there have been several incidents where a loss of control during the process of demolition resulted in the progressive collapse of part of an LPS dwelling block during demolition[6]. One incident caused damage to a nearby inhabited house.
12 METHODOLOGY FOR THROUGH-LIFE MANAGEMENT AND ASSESSMENT OF LPS DWELLING BLOCKS FOR ACCIDENTAL LOADS AND ACTIONS

12.1 INTRODUCTION
There are two main approaches to selecting technical treatments as part of the through-life management of concrete structures; these are the reactive and the proactive approaches. The reactive approach to instigating maintenance or remedial works is typically triggered by the occurrence of readily observable damage to a structure, such as the existence of cracking or spalling of concrete associated with the corrosion of embedded steel reinforcement. An intervention (repair) is then made in order to slow the rate of deterioration and extend the service life of the structure.

The proactive approach to maintenance or intervention works involves taking action earlier to extend the period of initiation, thereby delaying the start of corrosion propagation. In the case of corrosion, proactive treatments might include the early application of a coating to the surface of the concrete to delay the onset of corrosion or perhaps the provision of a cathodic prevention system to stop the initiation of corrosion.

When utilising a proactive through-life management approach it is necessary to adopt proactive methods for gathering information about the condition of the structure. Thus it will not be satisfactory to rely solely upon visual inspection procedures, as these methods can only provide an indication once damage has occurred by virtue of the fact they can only record the presence of visible manifestations of deterioration, such as cracking in concrete or spalled concrete. A proactive approach to through-life management requires the use of monitoring or non-destructive test methods to gain an early understanding of the changing condition of the concrete. These methods might include electro-potential mapping, recording of the depths of carbonation and the ingress of aggressive chemical species such as chlorides, as well as more advanced methods involving activities such as corrosion current measurement.

This section outlines:
- An overall through-life management methodology.
- A proposed structural assessment approach for LPS dwelling blocks utilising a four stage methodology comprising:
  - Assessment Stage 1: Review of existing technical information.
  - Assessment Stage 2: Collection of new technical information.
  - Assessment Stage 3: Assessment of block for normal loading.
  - Assessment Stage 4: Assessment of block for accidental loading.
- A suggested supporting regime involving periodic inspection, monitoring and structural assessment; driven by a risk-based methodology.
- Some risk reduction and risk management measures which might be considered.

12.2 BACKGROUND
Almost all structural elements in an LPS dwelling block are load bearing or contribute to the stability of the loadbearing elements of the LPS dwelling block – thus all load bearing elements and stabilising elements are considered to be ‘key elements’ as defined by Approved Document A – Structure[9].

BRE Report 107[2] indicates that:
- Robustness can be considered to be demonstrated for an LPS dwelling block or other type of building if there is adequate provision of horizontal or vertical ties (in terms of spacing and capacity).
- In the absence of appropriate tying, stability will be maintained if ‘key’ structural elements of an LPS dwelling block are able to resist the foreseeable accidental loading and actions. However, unfortunately for some types of LPS dwelling blocks it will not be possible to demonstrate adequate provision of horizontal and/or vertical ties, particularly the latter. This is generally the case with blocks designed/constructed prior to the partial collapse of Ronan Point in 1968.

The structural assessment methodology described below does not specifically consider the potential behaviour of an LPS dwelling block after a local failure has occurred, such as the removal of a critical section of a load bearing wall, which would necessitate alternative paths of support being mobilised to carry the load. Neither does this document specifically deal with the...
type of strengthening works which might be required to 
mobilise an alternative load path in such circumstances\textsuperscript{24}.

In the case of LPS dwelling blocks designed after 1968, 
their post local failure behaviour should be controlled by 
the additional horizontal and vertical tying required by 
guidance introduced soon after the Ronan Point incident.

In regard to ‘key’ structural elements of an LPS 
dwelling block being able to resist the foreseeable 
accidental loading and actions; if this resistance can be 
demonstrated it implies that failure would not occur in 
the structural concrete panel system forming the LPS 
dwelling block for such loads or actions. This ability has 
been termed ‘collapse resistance’ – see section 3.6.4.

Apart from the potential incidence of fire, the historical 
data suggest that the main concern for all buildings of 
over five storeys arises from internal gas explosions. 
Research\textsuperscript{24}\textsuperscript{26} indicates that an overpressure of 17 kN/m\textsuperscript{2} is 
a reasonable value to use for the structural assessment of 
LPS dwelling blocks and other type of building without 
a piped gas supply, as this threshold value is believed to 
be exceed the magnitude of the overpressures generated 
during almost all non-piped gas explosions that have 
happened within UK buildings. Over the two 10-year 
accidental explosion occurred in a UK building which 
was not associated with piped gas. This occurred in the 
Abbeystead water pumping station valve house, where 
methane originating from the ground accumulated in a 
water transfer tunnel and was discharged into the valve 
house, resulting in a very severe explosion\textsuperscript{26}.

12.3 OVERALL THROUGH-LIFE MANAGEMENT 
METHODOLOGY

To establish appropriate procedures and processes for 
the through-life management of an LPS dwelling block, 
consideration needs to be given to the following:

- Hazards and risk environment associated with fires, 
  internal gas explosions, external gas explosions and 
  vehicular impacts to establish threats and potential 
  accidental loads and actions to be considered.
- Environmental conditions within and outside the LPS 
  dwelling block influencing its future durability, together 
  with its current condition.
- The performance and assessment requirements for 
  the LPS dwelling block; by reference to the LPS block 
  characteristics considering details such as the number 
  of floors, the presence of a basement, the existence of 
  a piped gas supply, the anticipated future service life 
  required, etc.
- Managing the risks associated with LPS dwelling blocks 
  by adopting risk assessment procedures and by using 
  hazard elimination and risk reduction measures.
- Establishing proportionality criteria for hazard and risk 
  reduction measures.
- Undertaking a structural assessment of the LPS 
  dwelling block for accidental loads and actions to 
  identify risks that are not ‘regarded as insignificant and 
  adequately controlled’.

- Establishing whether the LPS dwelling block complies 
  with one or more of the following criteria:
  - **LPS Criterion 1.** There is adequate provision of 
    horizontal and vertical ties to comply with the current 
    requirements for Class 2B buildings as set down in the 
    codes and standards quoted in Approved Document 
    A – Structure\textsuperscript{26} or meeting the requirement set down in 
    the Building Regulations.
  - **LPS Criterion 2.** An adequate collapse resistance can be 
    demonstrated for the foreseeable accidental loads and 
    actions.
  - **LPS Criterion 3.** Alternative paths of support can be 
    mobilised to carry the load, assuming the removal of a 
    critical section of the load bearing wall\textsuperscript{25} in the manner 
    defined for Class 2B buildings in Approved Document 
    A – Structure\textsuperscript{26} or alternatively assuming the removal of 
    adjacent floor slabs (taking the floor slabs bearing on 
    one side of the wall at a time) providing lateral stability 
    to the critical section of the load bearing wall being 
    considered.
  - Or, alternatively, if the LPS dwelling block does not 
    satisfy the above (LPS Criteria 1 to 3) establish what 
    structural works need to be undertaken to enhance the 
safety/collapse resistance and/or reduce the prospect 
of progressive collapse or disproportionate damage.
  - Establishing proportionality criteria for structural works 
    undertaken to enhance the safety/collapse resistance 
    and/or reduce the prospect of progressive collapse or 
    disproportionate damage.
  - Implementing the structural works required to enhance 
    the safety/collapse resistance and/or reduce the 
    prospect of progressive collapse or disproportionate 
    damage where risks are not ‘regarded as insignificant and 
    adequately controlled’.
  - Establishing whether the LPS dwelling block is 
    adequately durable for the anticipated future service 
    life required. This will require a prognosis of the 
    durability and deterioration effects, together with any 
    associated implications for the structural performance 
    of the LPS dwelling block over the period until the 
    next proposed structural assessment is undertaken – 
    see section 10.3.
  - If the estimated durability is not adequate, establishing 
    what preventive or remedial interventions are required 
    to meet the future service life required.
  - Implementing the durability related works considered 
    to be required.
  - Updating of the overall through-life management 
    strategy following strengthening or other actions.

\textsuperscript{24} As might have been required by Method A: Ability to mobilise an 
alternative load path as set down in MHLG Circular 62/68\textsuperscript{v}

\textsuperscript{25} The nominal length of a loadbearing reinforced concrete wall is defined 
as the distance between lateral supports subject to a maximum length 
not exceeding 2.25H, where H is the storey height in metres. In the 
case of an external masonry wall, the nominal length is the distance 
between vertical lateral supports.
• If one does not exist, establishing and maintaining a service life/assessment history file containing all technical data relating to an LPS block (ie assessment/inspection reports, maintenance records, remediation details, drawings, etc.). Such records may also be referred to as a technical log/technical file. Such records are expected to be kept under current health and safety legislation.

12.4 OVERVIEW OF STRUCTURAL ASSESSMENT APPROACH FOR LPS DWELLING BLOCKS

The proposed structural assessment approach for LPS dwelling blocks involves a hierarchical series of steps, which utilise increasingly sophisticated forms of structural assessment calculation, in the process of seeking to establish that the ‘collapse resistance’ of an LPS dwelling block is likely to be satisfactory relative to the anticipated accidental loads or actions which might be imposed upon it. These steps are:

Assessment Level 1. A deterministic linear elastic analysis (eg spreadsheet based) of the wall panel and floor slab components, together with an empirical evaluation of the likely performance of the associated joints between components based upon experience gained from full-scale testing of LPS dwelling blocks containing similar joints.

Assessment Level 2. A deterministic non-linear elastic finite element (or alternative) analysis of selected wall panel and floor slab components which failed to satisfy the structural Assessment Level 1 procedure, as may be appropriate.

Assessment Level 3. Probabilistic calculations of selected wall panel and floor slab components which failed to satisfy the structural Assessment Level 2 procedure, as may be appropriate.

With this hierarchical approach to structural assessment, it is assumed that the assessment process will be stopped as soon as is practicable. Thus, if it is possible to justify the behaviour of the all the wall and floor slab components in an LPS dwelling block using structural assessment calculations performed to Assessment Level 1, then the process would stop.

If it was possible to justify the behaviour of only some of the wall and floor slab components of the LPS dwelling block at Assessment Level 1, the process would stop for those components. The remaining wall and floor slab components would then proceed to Assessment Level 2, and so on.

The aim of this particular iterative analytical approach is to seek to show, when this can be justified, that an LPS dwelling block would in practice be able to accommodate the forces developed during an internal gas explosion of a specified magnitude, without the need for supplementary strengthening.

The structural assessment approach has been developed on the back of the programme of full-scale testing and associated finite element structural modelling described previously, which has given experience of the performance of wall panels, floor slabs and the joints connecting these structural components at the level of applied loading associated with a severe internal gas explosion.

If the behaviour of the LPS dwelling block cannot be justified by the structural assessment calculations and the proposed risk reduction/management measures cannot reduce the risks due to accidental loads and actions to the point where they might be ‘regarded as insignificant and adequately controlled’; the respective wall panel and floor slab components and/or associated joints that failed the structural assessment would need to be strengthened. The strengthening provided would need to either prevent initiation of structural failure under the applied accidental loads or would need to control the post-failure behaviour of the LPS dwelling block.

The initial literature review, the collection of new technical information, if required, and the subsequent phased assessment process involve a number of distinct, but nevertheless related, stages. The four stages of the structural assessment methodology outlined below are:

• Assessment Stage 1 – Review of existing technical information.
• Assessment Stage 2 – Collection of new technical information.
• Assessment Stage 3 – Assessment of block for normal loading.
• Assessment Stage 4 – Assessment of block for accidental loading.
  o Assessment Level 1 – A deterministic linear elastic analysis (eg spreadsheet based).
  o Assessment Level 2 – A deterministic non-linear finite element (or alternative) analysis.
  o Assessment Level 3 – Probabilistic based calculations (structural reliability evaluation)

The main steps in the overall structural assessment process are shown mapped in Figure 34.

A detailed description of the form and nature of each stage of the assessment methodology is presented in Sections 12.5 to 12.8. For example, in Section 12.5 and Table 4 the full 21 steps of activity associated with Assessment Level 1 - Deterministic linear elastic analysis are set down. These activities are concerned with:

• Making a general evaluation and defining the classes of wall and floor panels and joints.
• Evaluating the quality of construction of the LPS dwelling block.
• Estimating the vertical loads in the walls at each floor level in the LPS dwelling block.
• Assessing the strength of wall panels and their resistance to overpressure loading.
• Assessing the strength of floor slabs and their resistance to overpressure loading.
• Based upon LPS full-scale test results, making an engineering evaluation of the potential behaviour of
If the behaviour of the LPS dwelling block cannot be justified by the structural assessment calculations and the proposed risk reduction/management measures cannot reduce the risks due to accidental loads and actions to the point where they might be ‘regarded as insignificant and adequately controlled’, the respective wall panel and floor slab components and/or associated joints that failed the structural assessment would need to be strengthened. The strengthening provided would need to either prevent initiation of structural failure under the applied accidental loads or would need to control the post-failure behaviour of the LPS dwelling block.

A detailed description of the form and nature of each stage of the assessment methodology are presented in sections 12.5–12.8.

12.5 ASSESSMENT STAGE 1 – REVIEW OF EXISTING TECHNICAL INFORMATION

12.5.1 General

It will be necessary to obtain all archive information pertinent to the block being assessed. If on searching it is found that the block has not been previously assessed or that the reports from previous assessments are not available, a complete dossier will need to be compiled.
It is likely that this will need to contain the types of information listed described below.

The data required would be expected to include items such as:

- LPS dwelling block system type.
- Height of block or its component parts.
- Existence of a basement and/or poorly ventilated spaces at ground level.
- Date of construction.
- Construction drawings showing plan layout and associated details.
- Wall panel and floor slab joint details.
- Wall panel and floor slab geometry.
- Type of heating system(s).
- Reinforcement provision and original tying provision.
- Material strengths.
- Information on the structure management regime, through-life care plan, etc.

It will also be necessary to establish if any significant gas explosions or fires have occurred during the life of the block, whether the block has experienced any structurally significant damage and what remedial measures were taken, if any, in order to rectify the structural damage. Consideration should also be given to the effects of deterioration or other damage upon the potential structural performance of the block or its components.

In addition, the results of any previous invasive investigations and materials testing programme (ie measurements of the depth of carbonation, concrete cover, chlorides, etc.) will need to be collated and reviewed.

Such information will need to be selectively substantiated by appropriate targeted investigations of the structure, a review of the previous maintenance history, design calculations, etc.

Where it is evident that structural or durability assessments have previously been undertaken, it will be necessary to undertake a desk top review of all the relevant documents, including reports, drawings, photographs, etc. The review will need to establish such facts as:

- Have investigations been undertaken upon the block?
- Has the block been subjected to an assessment, if so when was it undertaken and by whom?
- What was the block being appraised for (ie durability, normal or accidental loads).
- Was the block assessed against the 17 kN/m² or 34 kN/m² overpressure criterion?
- Are reports available detailing the results of any invasive investigations of the structural joints?
- Are the conclusions and recommendations supported by the available data, investigations and any calculations undertaken?
- What were the resulting recommendations and were they implemented fully. If so, when were the preventive and/or remedial works undertaken? Were these works supervised and were quality control/verifications records produced?
- If strengthening was carried out, what methods were adopted and to what extent was the block strengthened?
- Are there any records of calculations undertaken prior to strengthening?
- What maintenance, preventive and/or remedial works have been undertaken?
- Has a technical log been kept for the block?

If the available information, such as the findings of previous invasive investigations, suggest that the LPS dwelling block concerned could be especially ‘fragile’, it may be necessary to implement structural strengthening measures. Damage to a fragile LPS dwelling block could lead to progressive collapse\(^6\). Some points that might contribute to the fragility of an LPS dwelling block are the quality of the tying at the floor slab/cross wall/flank wall panel junctions, where there could be a significant lack of connection (mechanical tying) between floor panels and the cross/flank walls forming a bay, and the lack of end bearing to the floor slabs.

### 12.5.2 Presence of a piped gas supply to any part of the building

There is also the issue of whether an LPS dwelling block has a piped gas supply or not, as this will dictate the level of overpressure loading that should be adopted as the assessment criterion.

Checks for the presence of a piped gas supply are very important. In the past such checks have revealed the existence of appliances powered by piped gas in LPS dwelling blocks that were thought to be without a piped gas supply. Whilst this may be to only a small number of flats in the block, it does demonstrate that there is a piped gas supply within the building. This introduces the potential risk of a very severe gas explosion (see Figures B6 and B7 in Appendix B). In one case this risk had been present for over 25 years. Such examples raise questions about the manner in which a piped gas supply should be terminated. Ideally this should be done outside the LPS dwelling block in a ventilated chamber and with the gas pipe being physically capped off. Checks can be made with energy companies supplying piped gas for consumers with addresses within LPS dwelling blocks.

BRE is also aware of situations where proposals have been advanced in recent years to introduce a piped gas supply into LPS dwelling blocks, fortunately it was realised at a relatively early stage that such action would be inappropriate.

There are various matters relating to the presence of a piped gas supply noted below that may need to be considered in respect of the desired approach of managing the risks associated with LPS dwelling blocks by adopting procedures consistent with those described in Approved Document A – Structure\(^9\) for Class 3 structures, that is by employing hazard assessment procedures and by using hazard elimination and risk reduction measures.

These matters include issues such as the location and routing of the gas supply pipe(s), whether there are any safety systems installed in the gas supply/distribution pipes (eg automatic gas cut off valves which operate in the
event of a leak being detected) as well as the adoption of preventive maintenance contracts for the gas installation and associated appliances.

It is also pertinent to note that in the past a significant proportion of the ‘very severe’ gas explosions that have occurred in buildings have been caused by gas tracking into poorly ventilated rooms/spaces/voids in basement and ground floor areas via service pipe trenches from an external piped gas source. Fortunately this type of event is rare, especially since the introduction of plastic gas supply pipes which replaced the more vulnerable existing cast iron gas supply pipes in the 1970s.

The questions to be raised and addressed might include:

- If a piped gas supply is present, does the block have one or more basement rooms and are these rooms adequately ventilated to the external atmosphere?
- If piped gas is present, but the flats are heated via a central boiler, where is the boiler/gas supply? Is it situated on the roof, ground floor in-situ reinforced concrete podium/basement or in a boiler house away from the block?
- Are there dedicated piped gas supply to each flat? If so, are the gas supply pipes routed internally or externally?
- Are there dedicated boilers located within each flat or at another location, such as on a balcony open to the atmosphere?
- Are the boiler enclosures adequately ventilated to the external atmosphere?
- Are safety cut off valves present and are all gas appliances inspected on an annual/bi-annual basis?
- Are the gas supply pipes made of steel or plastic? Are they vulnerable to deterioration or vandalism?
- If the mains gas supply is reported to have been disconnected from the LPS dwelling block, has a check been made of every flat to ensure that the dedicated gas supply pipe has been removed or capped?
- Are there any underground trenches that run into/beneath a block? Are these adequately sealed or ventilated such that there is minimal risk of leaking gas tracking back into the building into unventilated voids?

12.6 ASSESSMENT STAGE 2 – COLLECTION OF NEW TECHNICAL INFORMATION

Where items of information are not available it may be necessary for appropriate studies and investigations to be undertaken to rectify the shortcomings. New data required might include items such as the following:

- Visual evidence about the performance and condition of the structure, including any indications of significant movement, deterioration, etc. taking place at either the:
  - Component level due to corrosion of reinforcement or fixings, thermal effects, etc.
  - Overall building level due to thermal effects, fire related damage, structural movement due to differential foundation settlement, etc.
- The location, distribution, condition, type and size of reinforcement and steel ties, coupling of reinforcement such as site welded connections, loops placed over dowel bars, lacer bars between projecting loop steel, etc. which are creating ties/continuity.
- The characteristic compressive strength of concrete walls and floors – with strengths being determined for different types of concretes, such as normal weight and lightweight aggregate concretes if both are present.

Thus the low compressive strengths obtained for cores taken from some LPS dwelling blocks in various parts of the UK have highlighted the need to verify the actual concrete strengths of both wall panels and floor slabs when undertaking a structural assessment.

It will, therefore, be necessary to establish whether there is any existing information on measured material strengths, in particular, the results of compression tests on cores taken from internal and external concrete wall panels and floor slabs or established by means of in-situ strength tests (eg Capo test). Data on concrete compressive strength may also need to be obtained where the external load bearing panels contain a different aggregate to that present in the internal wall panels (ie Lytag lightweight aggregate concrete). One beneficial effect is that the compressive strength of the concrete is likely to have increased to some degree over the years since construction.

In addition, it will also be necessary to establish the type of reinforcement used in the wall panels and floor slabs, the detailing adopted and whether any information is available on its tensile properties. It should be noted that some LPS dwelling blocks were constructed during periods when reinforcing steel was in short supply. This had the effect that reinforcement could, in some cases, be obtained from a series of different and potentially varying sources. Thus the steel provision might vary significantly from one point of the building to another, so there may be considerable variability in these details. In addition, the properties of the reinforcement (ie yield strength, ductility etc) may vary considerably depending upon the source of supply and how that varied during the construction period. In these circumstances the reinforcement provision can be very varied, confusing and difficult to establish during site investigations.
chloride content, concrete cover and carbonation depths, quality and placement of dry-pack and in-situ concrete. Consideration should also be given to the presence of voids / significant debris within in-situ structural joints.

- Details of the piped gas installation, if there is one, and any other perceived hazards both within or outside the LPS dwelling block concerned.

It is important to ensure that an adequate number of concrete cores and lengths of reinforcement are obtained from a range of components distributed throughout an LPS dwelling block. In adopting this approach it is envisaged that any variations in concrete quality that might have occurred during the construction phase will be identified. It is recommended that a minimum of six cores should be taken from each element type (ie individual wall panel and floor slab types). Ideally these should be 100 mm diameter cores but 75 mm27 diameter cores have proved adequate and are more practical to obtain in many instances due to the lack of suitably sized areas of unreinforced concrete (especially in voided floor slabs). A similar number of cores should also be taken from load bearing panels that contain concrete using different types of aggregate (eg Lytag lightweight aggregate concrete).

Appendix A in Part 2 of BRE Report 1072 on LPS dwelling blocks provides a summary of some of the physical testing methods that are appropriate for the assessment of this type of building, together with an indication of their limitations. The general categories include visual inspection, non-destructive test methods, mechanical test methods, the recovery of material samples for laboratory testing and structural load tests.

More recent guidance on non-destructive and partially-destructive testing techniques is available in:

- Appendices 7 and 8 of the 3rd edition of the Institution of Structural Engineers document Appraisal of existing structures48.

The most appropriate inspection and testing techniques have been shown to be:

- Visual inspection.
- Cover meter surveys using basic and more advanced reinforcing bar imaging systems, such as the Hilti Ferroscan. The cover meter surveys should be combined with local break-outs to establish reinforcement type and to resolve uncertainties in results.
- Concrete coring to establish concrete strength combined with laboratory testing and CAPO testing to establish concrete compressive strength in situ. BS 608957 covers a range of these and other techniques that may be used to assess the in-situ concrete strength in existing buildings.
- Laboratory testing of specimens of reinforcement taken from non-critical areas of precast concrete components and in-situ joints between precast concrete components.
- Estimating the tensile strength of the reinforcement, perhaps undertaken on lengths of exposed reinforcement by non-destructive techniques (ie ultrasonic methods) or by the removal of samples of reinforcement for destructive laboratory testing (ie tensile testing).
- Localised invasive breaking-out of in-situ concrete joints and precast concrete components using hand held tools to allow direct visual inspection.
- Full-scale structural load testing.
- Testing of the concrete cover for the depth of carbonation and obtaining concrete dust samples for laboratory-based chloride content determinations.

If an LPS dwelling block has been previously strengthened, on-site testing may be able to evaluate aspects of its performance. For example, hydraulic pull-out testing has been used to verify that retro-fitted tension bolts can carry a specified proof tensile load.

Should the investigations reveal severe deficiencies and/or deterioration in elements of an LPS dwelling block, it may be necessary to undertake immediate remedial works. The issue of the fragility of an LPS dwelling block has been mentioned in section 12.5.1.

12.7 ASSESSMENT STAGE 3 – ASSESSMENT OF BLOCK UNDER NORMAL LOADING

Assessment of an existing building differs from the design of a new one in that it is possible to obtain information about the building as-constructed and about its performance in service. It also can reduce uncertainty, in as much as fewer assumptions are required to be made, which should enable a more realistic analysis of a structure to be undertaken. The structural assessment will often be able to take advantage of higher strengths derived from material testing.

The functions of different types of joints present in LPS dwelling blocks are outlined in BRE Report 10751. BRE Report 10752 also describes some of the approaches available for the assessment of an LPS dwelling block. The assessment of a block against normal loading should not normally be an onerous task unless there are already signs of distress.

The satisfactory structural performance of a block during its life should be taken as a positive indication of its behaviour under normal loading conditions. The period that an LPS dwelling block has been in service is now likely to be over 40 years as the majority of blocks in the UK were constructed in the 1960s. Nevertheless, consideration should be given to any evidence of structural distress, such as cracking or crushing of concrete, or known widespread shortcomings in floor slab bearing lengths. Consideration should also be given to any indications of deterioration, such as the cracking or spalling of concrete caused by reinforcement corrosion, together with the potential implications of this both currently and in the future.

27 Assumes a maximum 20 mm aggregate size.
12.8 ASSESSMENT STAGE 4 – ASSESSMENT OF BLOCK UNDER ACCIDENTAL LOADING

12.8.1 Introduction
The aim of this form of structural assessment is to ascertain whether the walls, floors and structural joints within an LPS dwelling block are likely to be able to satisfy the appropriate assessment criterion, and hence be expected to resist the forces generated during either a piped or non-piped gas explosion.

The suggested structural assessment methodology utilises a staged evaluation approach. This would generally be expected to use deterministic calculations in the structural assessment procedure (Assessment Level 3), if sufficient information were available to undertake the necessary calculations. Whilst such an approach might theoretically provide a basis for determining the probability of failure for a range of accidental loads generated by a variety of explosion sources, considerable effort would no doubt be required to develop an acceptable methodology.

The proposed basic principles/concepts behind the proposed staged approach to the assessment of behaviour, and the subsequent management of performance are as follows:

- Undertake an evaluation using appropriate inspection techniques to determine whether appropriate horizontal ties or/and vertical ties within a block are provided and comply with the current requirements for Class 2B buildings as set down in the codes and standards quoted in Approved Document A – Structure[9]. If this is so the LPS dwelling block will be considered to satisfy Requirement A3 in Approved Document A – Structure[9].
- Evaluate whether the structural elements and associated joints forming the LPS dwelling block are sufficiently strong to accommodate the required accidental loading – various methods for establishing this are discussed below as Assessment Levels 1–3.
- If the evaluations made above fail to satisfy the assessment criteria, then there will be a need to undertake other actions. These could involve a combination of the following:
  - Removing or minimising the risk of an explosion or the threat from another hazard occurring, if this is possible.
  - Strengthening selected components and/or associated joints to minimise the risk of failure should an explosion occur or to control the behaviour after local failure by mobilising alternative load paths.
  - Alternatively, if neither of these approaches is acceptable or cost effective, it then might be necessary for the LPS dwelling block to be demolished and the site re-developed.

Summary details of the various structural assessment approaches (Assessment Levels 1 to 3) are presented in the following sections and associated Tables 4 to 6.

With the introduction of the Structural Eurocodes, which replaced existing UK national guidance for the design of concrete structures (eg BS 8110) from April 2010, it is perhaps appropriate to consider aspects of the guidance they give on various matters. For example, BS EN 1990: Basis of design[11] provides guidance on the combination of actions to be used for accidental design situations by means of the following expression (Equation 6.11b in BS EN 1990):

\[
\sum_{j=1}^{n} A_{j} + P + \sum_{i=1}^{m} Q_{i} + d = \sum_{j=1}^{n} A_{j} + P + \sum_{i=1}^{m} Q_{i} + d
\]

where:
- \( A_{j} \) = design value of an accidental action
- \( C_{ij} \) = characteristic value of permanent action \( i \)
- \( Q_{ij} \) = characteristic value of leading variable action \( j \)
- \( P \) = relevant representative value of a prestressing action
- \( \psi_{1} \) = factor for frequent value of a variable action
- \( \psi_{2} \) = factor for quasi-permanent value of a variable action
- \( \psi_{Q} \) = quasi-permanent value of a variable action

The UK National Annex to BS EN 1990[11] defines the value of \( \psi_{1} \), which is to be taken with the leading variable load. \( \psi_{1} \) is 0.5 for residential areas (Table A1.1 of BS EN 1990).

BS EN 1990 notes that the choice between \( \psi_{1} \) and \( \psi_{2} \) should be related to the relevant accidental design situation (impact, fire or survival after an accidental event or situation) and that guidance for concrete structures is given in BS EN 1992[10,11].

The values are somewhat different to contemporary UK practice for undertaking an assessment under accidental loading circumstances, where the proportion of imposed load assumed to apply would perhaps be taken to be 33% of the notional imposed load value.

12.8.2 Assessment Level 1 – Deterministic linear elastic analysis (eg spreadsheet based)

This section describes the simplest level of structural assessment which is most likely to be undertaken using some form of linear elastic spreadsheet based analysis or similar tool, but alternatively the analyses are generally simple enough to be undertaken by hand if that is appropriate. However the approach would be extremely laborious. Evaluations have been carried out on the basis of criteria established for both (a) permissible stress and (b) partial load and material factor methods. The latter approach appears to result in a more conservative assessment, but of course this depends upon the relative values taken for permissible stress and the partial load and material factors adopted.
Following the introduction of the Structural Eurocodes, it is assumed that structural assessments in the future will increasingly be undertaken within this framework. It is understood that CEN is due to initiate a project in 2012 looking into the application of the Structural Eurocodes for the structural assessment of existing buildings.

The structural assessment example presented in Table 4 is for an internal overpressure loading.

<table>
<thead>
<tr>
<th>Table 4: Assessment Level 1 – A deterministic linear elastic analysis (eg spreadsheet based)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Part 1:</strong> Make a general evaluation and define classes of wall and floor panels and joints</td>
</tr>
<tr>
<td><strong>Step 1</strong> An evaluation should be made of the ‘general condition/form’ of the LPS dwelling block on the basis of the drawings and an initial site inspection in order to understand the basic structural form of the LPS dwelling block and how the various loads are transferred to the foundations, together with any implications there might be about the vulnerability of the structure to damage and/or deterioration. Loadbearing and non-loadbearing wall and elevation components should be identified. Account should be taken of/implications of any damage, etc. assessed. Divide the structure into a number of selected classes of precast concrete wall panel and floor slab precast concrete component types, together with the in-situ joints between them. The main component types might include flank wall, internal cross wall, internal spine walls, elevation walls, floors (categorised by span length). Identify various types of joint such as wall/wall, floor/floor and wall/floor. It is necessary to define the location of the various ‘classes’ of wall panels and floor slabs and the associated joints within a typical floor plan layout. This enables a standard layout LPS dwelling block with a repetitive floor layout to be prepared for the following ‘run-down’ calculations and for comparison with previous full-scale load test/performance data.</td>
</tr>
<tr>
<td><strong>Step 2</strong> Evaluate the quality of construction Considered should be given to whether any observed variations in detailing are consistent with intended local variations in standard system details. Could any discrepancies identified potentially have beneficial effects on the performance under accidental load? If so, what are they? Note: Many standard systems exhibit marked variation from one contract (includes both geographical and time dimensions) to another depending on factors such as the standard of supervision and local engineering requirements or stipulations that had to be complied with.</td>
</tr>
<tr>
<td><strong>Step 3</strong> Load run-down methodology Calculate actual vertical loads at top, mid-height and base of each class of wall panel at each floor level as outlined below. The task of determining the proportion of the combined dead and imposed loads that is distributed into each of the supporting walls can be undertaken using two approaches. The first is applicable to buildings which contain primarily one-way spanning floor slabs supported upon two walls of equal height. In this case it is assumed that half of the total dead and imposed load associated with a floor slab is taken by one wall, with the other half by the second wall. The second approach, using hand calculations or linear finite element modelling, can be used where the floor slabs are supported upon three or more walls and/or where the floor slabs are required to support intermediate walls located away from the perimeter of the floor slab. The load distribution coefficients derived from hand calculations or linear finite element model analysis for each floor slab bearing on a particular wall will need to be summed to produce a total applied load for that wall at a particular floor level (the geometry of floor slabs may vary by floor level). This process will need to be repeated for each supporting wall and its associated floor slabs. Each load derived from the individual load distribution coefficients will then need to be summed and added to the total combined dead and imposed load from the structure above the wall being considered. However, in following these procedures at least some engineering judgement is required for dealing with the effects as such where there are wall supports that do not extend the whole length of a slab side.</td>
</tr>
</tbody>
</table>

| 30 In common with the outcomes for all structural analysis situations sensitive to differential movements and deflection effects, and therefore where such behaviours are a significant factor in load take down calculations, it must be recognised that the reaction loads derived in this manner are ‘idealised’ and therefore are best estimates for the assumed support and related conditions. In this situation even relatively small variations in the degree of support and differential vertical movement of individual walls will result in a notional redistribution of dead and imposed loads between elements of the load bearing structure. Therefore the actual loads carried by the supporting walls could potentially be significantly different from the ‘idealised’ calculated values. |
Assess strength of wall panels and their resistance to overpressure loading

Step 4
Calculate the flexural strength (moment capacity) of each class of wall panel using actual characteristic material strengths derived by sampling, taking account of the vertical load.

Step 5
Calculate the maximum moment generated by the appropriate level of accidental loading (ie 17 or 34 kN/m²) overpressure criterion, making appropriate assumptions about the boundary and support conditions to the elements being assessed.

Step 6
Compare the resistance and loading values/calculate the factor of safety in flexure for each of wall panel at each floor level taking account of the vertical load effects: Do any of the wall panels fail the assessment criterion in flexure?

Step 7
Calculate the shear strength of the base and head (depends on LPS form of construction) of each class of wall panel using actual characteristic strengths of materials derived by sampling and taking account of vertical load if appropriate.

Assess strength of floor slabs and their resistance to overpressure loading

Step 11
Calculate the flexural strength (moment capacity) of each class of floor panel using actual characteristic strengths of materials derived by sampling for UPWARD loading.

Step 12
Calculate the flexural strength (moment capacity) of each class of floor panel using actual characteristic strengths of material derived by sampling for DOWNWARD loading.

Step 13
Calculate the maximum moment generated by UPWARD load under the appropriate levels of dead, imposed and accidental loading (ie 17 or 34 kN/m²) criterion – it may be necessary to consider behaviour of a plain concrete section where no top reinforcement is present.

Step 14
Calculate the maximum moment generated by DOWNWARD load under the appropriate levels of dead, imposed and accidental loading (ie 17 or 34 kN/m²) criterion - consider reinforced concrete section.

Step 8
Calculate the applied shear force for the lateral load under the appropriate level of accidental loading (ie 17 or 34 kN/m²) overpressure criterion.

Step 9
Compare the resistance and loading values/calculate the factor of safety in shear at the base and head (depends on LPS form of construction) of each class of wall panel at each floor level: Do any of the wall panels fail the assessment criterion by shearing?

Step 10
Evaluate risk of sliding at base and top (depends on LPS form of construction) of each class of wall panel at each floor level using loads derived at Steps 3 and 8: Do any of the wall panels fail the assessment criterion by sliding?

(Note: Take into consideration floor slab uplift forces where appropriate)

Step 15
Compare the resistance and loading values/calculate the safety/overload factor for both UPWARD and DOWNWARD flexure for each class of floor panel: Do any of the floor slabs fail the assessment criterion in flexure?

Step 16
Calculate shear capacity of each class of floor panel for both UPWARD and DOWNWARD loading conditions using actual characteristic strengths of material derived by sampling.

Step 17
Calculate the total applied shear force for both UPWARD and DOWNWARD loading conditions.

Step 18
Compare the resistance and loading values/calculate the safety/overload factor for both UPWARD and DOWNWARD shear for each class of floor panel: Do any of the floor slabs fail the assessment criterion in shear?

Engineering evaluation of potential behaviour of joints based upon LPS full-scale test results

Step 19
Review the identified types of joint such as wall/wall, floor/ floor and wall/floor for particular situations and make comparison with previous full-scale load test/performance data: Come to a view on the applicability of previous data to current circumstances: Are any of the joints unlikely to perform satisfactorily under the level of load applied?

Engineering evaluation of results of deterministic calculations etc and previous performance data

Step 20
Undertake review of outcomes of structural assessment making comparison with BRE full-scale load tests/previous load tests upon LPS components, joints and buildings by others and other published performance data; particularly for items which fail the assessment criterion.

Step 21
Based on results of the simplified calculations and results obtained from Steps 19 and 20; form an engineering view upon the likely behaviour of each class of wall panel and floor slab, joint, etc. under accidental loading at each floor level. Identify potential inadequacies and items which fail the assessment criterion, reviewing these against previous performance data.
12.8.3 Assessment Level 2 – Deterministic non-linear finite element analysis
In cases where the ‘simplified’ deterministic calculations and associated engineering reviews undertaken as part of Assessment Level 1 process are unable to show that certain classes of wall and/or floor components meet the assessment criteria, then consideration should be given to modelling the appropriate component using a non-linear finite element package (Assessment Level 2), or other appropriate analysis procedure, to gain more understanding about their potential behaviour under accidental overpressure loading.

A summary of the main factors that might need to be taken into consideration when undertaking this form of advanced analysis is given in Table 5.

12.8.4 Assessment Level 3: Probabilistic-based structural assessment methodology
In situations where calculations to both Assessment Levels 1 and 2 are not able to show that all classes of walls and floors within a LPS dwelling block are able to meet the assessment criteria, then the use of a probabilistic evaluation approach/structural reliability evaluation (Assessment Level 3) may offer an appropriate way forward. The basic concept and factors that should be considered when developing and undertaking a probabilistic assessment of this type are outlined in Table 6.

12.9 SUGGESTED SUPPORTING REGIME INVOLVING PERIODIC INSPECTION, MONITORING AND STRUCTURAL ASSESSMENT
The following is a suggested durability inspection and monitoring regime for an LPS dwelling block, which is linked to the periodic structural assessment of the block. This is a development of the original principles set down in BRE Report BR 107[7].

- Visual inspections at a period not exceeding five years, but preferably this should be driven by risk-based inspection principles with more frequent inspections of particular components/actions if this is considered appropriate by durability/deterioration considerations (eg if there could be a hazard caused by spalling concrete, etc.). Aspects of the external visual inspections are expected to be made at close range (ie touching distance).
- Inspection and assessment of the condition of selected in-situ joints likely to be at higher risk of deterioration or affected by rain penetration/dampness into the outer envelope (eg flank wall joints) and thereby at risk of deterioration due to reinforcement corrosion at a period not exceeding 15 years, but preferably this should be driven by risk-based inspection principles.
- These activities should be supported by associated durability related testing and sampling of selected concrete components at a period not exceeding 15 years applied more generally to the block, but preferably this should be driven by risk-based principles.

Table 5: Assessment Level 2 – A deterministic non-linear finite element analysis

Step 22
Where it cannot be shown by simplified calculations that a wall or floor panel is able to meet the appropriate assessment criteria then the next approach could be to construct a non-linear finite element analysis model. Alternatively another appropriate analysis procedure may be employed.

This model should, wherever possible, use measured component geometry and representative material properties derived by sampling on site. Where there are uncertainties relating to geometry or material properties then it will be necessary to use reasonable estimates based on previous experience or obtained from technical literature. Consideration will need to be given on a case by case basis as to what ‘representative’ material properties means in the particular circumstances encountered.

As well as modelling the geometry of the floor slab, bottom and top reinforcement and the concrete; it is also necessary to consider the possible contributions made by in-plane tying and non-load bearing partitions (above and below floor slab) which may be able to provide supplementary support. The same considerations also apply to wall panels.

It is appropriate to adopt conservative estimates of the unknown parameters which would result in an underestimate of the likely strength (or beneficial effects) behaviour of the element being modelled when subjected to accidental loading.

The contribution made by friction and adhesion forces should also be considered as this plays a significant role in the behaviour of wall panels under accidental loading.

The modelling of boundary conditions should be as realistic as possible, leading to neither an overestimate nor an unrealistic underestimate of the flexural or shear (strength) capacities. This may require multiple models to be run to bracket the most likely, the favourable and least favourable combinations of material properties and geometric considerations, etc.

Careful consideration must be given to the type of finite element elements used when constructing the model, as well as their number and distribution. Such work should be performed by an appropriately experienced specialist with knowledge pertinent to the field of reinforced concrete finite element modelling.

The assessment of the capability of a floor or wall panel to resist the required loading is based on the pressure-deflection profile at a relevant reference point (usually at the location of maximum deflection at a given instant). In the case of a floor panel this is for both upward and downward loading conditions. For wall panels this will be for lateral loading from either side; but in the case of the flank wall this will be only from the inside. It is also appropriate to assess the level of distress likely to be experienced by the concrete (in terms of cracking strains) and reinforcement (yielding stress). A substantial increase in deflection for a small increase in applied load is generally taken to be indicative of the impending failure of an element (ie a low stiffness value at the load associated with the assessment criterion).
In conducting probabilistic analysis the concrete compressive strength data and steel strength data derived from laboratory testing from material samples taken on site should be statistically analysed to transpose them into an appropriate format for the analysis. However in undertaking this form of assessment it will be necessary to make justifiable assumptions and judicial simplifications, as appropriate, in relation to material properties, geometry, reinforcement detailing, boundary conditions and the applied overpressure loads when specific factual information/measured properties are not available.

The probabilistic assessment methodology could also be developed such that it is able to utilise the results of full-scale load testing undertaken to establish the likely range of failure loads in elements such as vents, windows (and their fixings), balcony partitions, etc. The risk assessments undertaken as part of the assessment methodology will need to consider the safety risks, in terms of reliability levels achieved and fatalities, which are considered to be of primary concern to the building owner. Consideration of economical risks associated with an ‘event’ can also be included, if desired, once safety risks have been addressed and resolved.

Three types of safety risk criteria may be considered in a probabilistic evaluation. The building can be considered to have failed the assessment if any one of the three criteria is not met. The three criteria previously adopted by BRE for this type of assessment are:

- Structural reliability of wall panels and floor slabs and that achieved by the overall building in relation to reliability levels considered acceptable under current design codes.
- Safety risks to an individual: This level of risk was compared with that perceived to be acceptable by society, and although not directly relevant, that recommended by the HSE for occupational health and safety purposes. Appendix C discusses these issues.
- Safety risks for multiple fatalities: This may be expressed in terms of the significance of an ‘event’ in relation to the consequences of failure and public perception of such an ‘event’. This considers society’s ‘demand’ that accidents causing significant fatalities should have a very low probability of occurrence whereas it is prepared to accept a higher frequency of ‘events’ where there is a low number of casualties per ‘event’. Appendix C discusses these issues.

If one of the above criteria is not met then one final option could be considered. This consists of seeking to obtain more complete and/or accurate data on factors such as the ability to vent the products of the deflagration, since it can be argued that controlled venting can help to reduce the overpressures generated within compartments subject to an internal gas explosion. Accordingly, it has been possible to derive an evaluation process using gas explosion pressures that may occur when a room is vented, say as a result of the failure of windows. However, the magnitude of the reduction in overpressure depends upon the degree of venting which occurs and this is influenced by a number of factors including the strength of the windows and the area/size of the openings they are situated in. In any such probabilistic assessment, however, it is important to be able to evaluate the levels of risk and to establish that these are satisfactory/acceptable in relation to the possible number of people that are at risk from the event being considered.

The judgement on this period would be assessed on a risk basis.

12.10 RISK REDUCTION AND RISK MANAGEMENT MEASURES

12.10.1 Introduction

If it has been possible to demonstrate that all classes of wall panels and floor slabs within the LPS dwelling block are sufficiently strong to accommodate the forces associated with the assessment criteria, then no remedial actions need to be taken. However, building owners should continue to exercise best practices in the management of the LPS dwelling block.

One aspect of these practices is risk reduction and risk management measures consistent with ALARP/SFARP practice. For example, risks of an internal gas explosion associated with piped gas can essentially be eliminated by removing the piped gas supply from the building. The LPS dwelling block could then be re-assessed for the lower overpressure assessment criterion of 17 kN/m².
However, if the piped gas supply needs to be retained, the already low risk of an internal explosion might be further reduced by the introduction of active control measures. Currently this management strategy is considered to be suitable only as a short-term measure whilst a long-term strategy is developed.

Some risk reduction and risk management measures which may be adopted in different circumstances are given below.

### 12.10.2 LPS dwelling block without a piped gas supply

The following give some suggestions for risk reduction measures which might be considered:

- Installing signs in prominent positions within the LPS dwelling block warning of the implications of storing and/or using explosive substances such as petrol, bottled gas, etc. in any room within the building.
- Briefing of all residents before taking up residence in a block, and regularly during their tenancy, to explain the risks of storing potentially explosive substances in their flats. This might be supported by appropriate penalties should LPG cylinders, petrol or the like be found in the possession of the residents/others within an LPS dwelling block. These materials might potentially lead to ‘severe’ explosions (see Appendix B, section B2). It would also be necessary to consider the potential risks of storing aerosols. BRE research suggests that generally the magnitude of any resulting explosion would not be expected to producing an overpressure exceeding about 10 kN/m². As a consequence such explosions would be typically be classified as being ‘moderate’ explosions (see Appendix B).
- Adopting the practice of ensuring forced ventilation during building/repair works which involve the use of bottled gas, such as during ‘hot works’ associated with plumbing works. This could include the stipulation that internal doors and nearby windows should remain open throughout the period of the works to enable any gas that does escape is able to vent safely to the external atmosphere. However, since this requirement is likely to compromise the fire safety of the area of the building in which the work is being undertaken, it would be necessary to shut doors in the event of a fire, where they provide a fire compartment boundary. The use of portable gas meters for monitoring air quality would provide a further line of defence by providing an audible warning of the build-up of significant concentrations of LPG.
- Ideally there would be some degree of proactive checking of flats to ensure that they were not being used to store flammable or explosive materials. It would be necessary for any high risk substances discovered to be removed for storage in an appropriate location elsewhere or safely disposed of. This approach might be supported by appropriate penalties. However, such inspections may be difficult to implement if there could conflict with issues of privacy and personal/civil liberties, etc.
- Re-location of potentially high-risk materials to storage areas where the consequences of an explosion would be less significant.

### 12.10.3 LPS dwelling block with a piped gas supply

The following give some suggestions for risk reduction measures which might be considered:

- Installation of gas monitors in flats and other locations to detect gas leaks.
- Installing safety cut-off valves on each gas appliance.
- Annual servicing of all gas appliances.
- Installation and regular inspection of vents in boiler enclosures to external atmosphere upon, to minimise the build-up of significant quantities of escaping gas.
- Ensuring that all enclosed spaces/voids are adequately ventilated to the external atmosphere.

Section 9.4 also discusses a number of issues associated with piped gas supplies.

In addition if the piped gas supply were removed, the management strategies for a LPS dwelling block without a piped gas supply could be implemented, as described above.
13.1 BACKGROUND

Structural strengthening of LPS dwelling blocks may serve two purposes. The first is to avoid the initiation of failure of individual components under accidental loads or actions. The second is aimed at enabling the structure to develop alternative load paths, thereby enabling an LPS dwelling block to bridge over an area of local damage.

During the process of inspecting a range of LPS system types and reviewing technical reports that have been prepared by consulting and design engineers, a range of strengthening techniques have been identified which have been installed to enhance the strength of local areas of a block or more globally. Other systems have been introduced to provide alternative load paths with a view to preventing progressive collapse/disproportionate damage from occurring. Whilst the purpose of many schemes has been immediately evident, the role and effectiveness of others have been somewhat unclear.

In many instances there is a complete absence of design calculations or supporting technical documentation (ie drawings, previous structural assessment reports, etc.). Had they been available, then it would have been possible to undertake a design review. It would also have been possible to establish and verify the underlying design philosophy.

An overview of the general forms of strengthening that has previously been undertaken in a number of LPS dwelling blocks within the UK together with the underlying principles and limitations is included in Appendix I to illustrate some of the strengthening approaches that have previously been employed.

13.2 DEVELOPMENT OF STRENGTHENING/THROUGH-LIFE RISK MANAGEMENT STRATEGY

The results of a structural evaluation of a block under normal or accidental loads or actions (associated with either an overpressure criterion of 17 or 34 kN/m²) may indicate that the panels and associated joints are likely to be able to accommodate the relevant overpressure loading. In which case, no further action is required apart from meeting general obligations in respect of statutory duty of care to the residents of the block being considered.

If, however, the evaluation indicates that there are deficiencies or that there is a higher than acceptable risk of failure in one or more types of connections between components or particular deficiencies in performance with respect to the predicted flexural or shear behaviour of specific wall panels or/and floor slabs under accidental loading, then appropriate remedial actions will need to be taken to redress these shortcomings.

One approach would be to provide strengthening to enhance the local strength of the panels and / or joints. Another would be to provide strengthening to facilitate the mobilisation of secondary load paths in the event of local damage resulting in the loss of one or more key load bearing elements.

A possible alternative/complementary approach could be to implement a management strategy with the aim of minimising the risk of structural damage being caused by an explosion or other incident. Where the probability of occurrence of a hazard and the associated risks are sufficiently low, under the ALARP/SFARP principle the risks may be considered to be ‘insignificant and adequately controlled’. These matters are discussed in Appendices B and C.

Up to now and because of uncertainty in respect of the legal aspects of adopting such a stand-alone management strategy, it is understood that this approach has been regarded as providing only a short-term solution whilst a long-term approach was developed.

Perhaps the most effective approach would be the selective combination of both of these approaches.

Whichever approach is decided upon it is important to take into account a number of technical, social, economic and regulatory factors when developing an appropriate strategy for the through-life risk management of the LPS dwelling block being considered. Figure 35 illustrates how a strengthening/remedial works strategy might be developed. Figure 36 shows how an approach to through-life risk management might be developed with respect to the identified hazards.

Compliance with health and safety and wellbeing requirements, local planning requirements and the environment conditions in and around the LPS dwelling block concerned could have a significant bearing upon the form and implementation of the final scheme. Therefore, these issues will need to be considered and the outcome incorporated into a final strengthening scheme.
Figure 35: Development of strategies for the strengthening of an LPS dwelling block
Future through-life management strategy – Issues associated with identified hazards

<table>
<thead>
<tr>
<th>Consider future management issues and options to minimise risks associated with an internal gas explosion or arising from other hazards. Define actions to be taken</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>General</strong></td>
</tr>
<tr>
<td>• Education of residents of the risks and implications of storing hazardous substances through meetings, notices, etc.</td>
</tr>
<tr>
<td>• Active policing of work practices to minimise risk of explosive mixture of gas build-up within a block during works involving LPG bottles. Also consider temporary forced ventilation during repair works using LPG</td>
</tr>
<tr>
<td>• Installation of temporary/permanent air quality monitors</td>
</tr>
<tr>
<td>• Ventilation of enclosed spaces/rooms to avoid build-up of gas</td>
</tr>
<tr>
<td>• Underground services/trenches to building – prevent entry of flammable gas from leaks by use of ventilated entry chamber</td>
</tr>
<tr>
<td>• Re-locate potentially hazardous materials to outside block</td>
</tr>
<tr>
<td><strong>Blocks with a piped gas supply</strong></td>
</tr>
<tr>
<td>• Installation of safety devices, e.g. gas cut-off valves</td>
</tr>
<tr>
<td>• Regular (annual/bi-annual) inspection of all gas fixtures and fittings/appliances by qualified gas fitter</td>
</tr>
<tr>
<td>• Ventilation of enclosed rooms/boiler enclosures to external atmosphere</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Consider end-of-life issues</th>
</tr>
</thead>
<tbody>
<tr>
<td>• Demolition</td>
</tr>
<tr>
<td>• Dismantling/re-use of structural components and materials</td>
</tr>
</tbody>
</table>

Figure 36: Development of an approach to through-life management considering future risk issues with respect to identified hazards
14 CONCLUDING REMARKS

14.1 HISTORICAL PERSPECTIVE ON LPS DWELLING BLOCK STRUCTURAL ASSESSMENT

There was a huge push in the late 1950s to provide increasing numbers of housing units within a very short space of time and by making the maximum use of restricted site space. These constraints and cost limits lead to the introduction of the large panel system form of construction in the UK using existing technology first developed in Denmark in 1948. One such system was the Larsen Neilsen system and Taylor Woodrow–Anglian Ltd were the UK licensees.

The first structure of this type erected in the UK was for the London County Council in 1963. When it came into being in 1965, the London Borough of Newham commissioned nine 22 storey Larsen Neilsen LPS dwelling blocks.

The construction of the now infamous Ronan Point block was started on the 25 July 1966 and it was the second to be completed and was handed over on the 11 March 1968. At the time of the collapse (5.45 am Thursday 16 May 1968) only eight flats remained vacant at the time of the explosion and it was fortunate that four of these were situated in the south-east corner which collapsed.

It was also fortunate that it was mainly the living room floors that failed below the explosion and in most dwellings the bedroom areas remained intact. At the time the partial collapse occurred most residents were still in bed and therefore the number of lives lost was much lower than might have been expected had the explosion occurred a few hours later.

It is significant that the explosion took place four floors from the top of the building, where the vertical pre-load in the load bearing walls was quite low. In essence the mechanism of collapse was that pressure was simultaneously generated on the external flank wall and the floor and ceiling of the flat. This lifted the top four floors momentarily while the, now unrestrained, flank wall was blown out. When the load from the top four floors returned, the supporting walls were no longer present. Accordingly the weight of much of the upper construction descended through a storey height before impacting on the next lower floor of the south east corner. Once initiated the kinetic energy contained in the failure load grew rapidly, allowing the collapse to progress at almost free fall speed.

Estimates of the pressures arising from the gas explosion were made by assessing the forces which had deformed a number of items in the flat.

Loads from gas explosions develop relatively slowly and as far as the structural components are concerned can be considered as short duration static loads. The best estimate which could be made at the time was that an overpressure of 5 psi (pounds per square inch), equivalent to approximately 34 kN/m², had occurred producing sliding failure at the base of the flank wall panel concerned. This value was adopted as the checking criteria for LPS dwelling blocks containing a piped gas supply.

On 15 November 1968 the Ministry of Housing and Local Government (MHLG) issued Circular 62/68 instructing owners of LPS dwellings to appraise all blocks over six storeys in height.

MHLG Circular 71/68 was issued on 20 December 1968, which gave further advice on the assessment method to be adopted.

The six storey criteria was adopted on the basis that the explosion had completely destroyed the flat concerned and the one below and above, and substantially damaged the flats above and below these. In essence five floors of flats were severely damaged directly by the effects of the piped gas explosion. It was not thought practical or sensible in the event of a similar size explosion to try to design against such damage occurring within five floors of construction. It was therefore accepted at that time that five storeys would be lost and that the engineering task was to ensure that the collapse would not spread to the rest of the building, ie the damage should be proportional to the cause.

Circular 62/68 states there are two basic methods of avoiding progressive collapse, namely:

Method A: By providing alternative paths of support to carry the load, assuming the removal of a critical section of the load bearing walls.

Method B: By providing a form of construction of such stiffness and continuity so as to ensure the stability of the building against forces liable to damage the load supporting members.

Over the intervening years the stability concepts developed following the Ronan Point collapse have been
incorporated into Building Regulations and research has better quantified the overpressures generated and the frequency of different types of explosions occurring within UK buildings.

In 1968 the climate of opinion generated by the Ronan Point collapse was extreme, leading to fear amongst tenants housed in similar construction and widespread blighting of properties. Naturally enough engineers tended to take a conservative view when making structural assessments and some locally augmented the requirements of Circulars 62/68[4] and 71/68[5] or perhaps interpreted them in a way which today we might consider unnecessarily cautious.

**14.2 CONTEMPORARY GUIDANCE ON LPS DWELLING BLOCK STRUCTURAL ASSESSMENT**

The Ronan Point collapse has clearly proved to be a very significant event not only in the history of structural engineering but also in terms of some of the central tenets of contemporary structural engineering philosophy, particularly to the ongoing and relevant discussions on matters concerning disproportionate damage and associated topics. However, it is in the nature of things that, with the passage of time, priorities change and past events feature less prominently in people’s consciousness. Those unfamiliar with the Ronan Point incident may be unaware of the origin of some of today’s performance requirements and the provisions which need to be made.

This Handbook attempts to counter this trend by bringing together the relevant background information and putting this into the context of current performance expectations. It has done this by providing insight into the evolution of the Building Regulations and the current Requirement A3 of Approved Document A – Structure[9].

This is supported by a review of statistical data as far as this is available for the hazard environment within which LPS dwelling blocks exist, linking this information to contemporary risk assessment and management concepts (ie ALARP/SFARP principles) to allow the adoption of rational risk-based strategies for the through-life management of this class of buildings.

These developments have been facilitated by the knowledge gained by BRE about the behaviour of LPS dwelling blocks from two programmes of full-scale structural testing to the ultimate load condition within three LPS dwelling blocks, comprising two of Bison Wallframe design and one of a Reema Conclad design. The Bison Wallframe blocks were 15 and 22 storeys high; and the Reema Conclad block was 10 storeys high.

These tests enabled certain facets of structural behaviour to be explored and, in all three cases, to demonstrate that adequate reserves of strength existed in these LPS dwelling blocks for the accidental loading situations that they were likely to be exposed to. Thus the three LPS dwelling blocks concerned were able to resist the overpressure loadings associated with the specified structural assessment criterion of 17 kN/m² applicable to buildings without a piped gas supply.

This evaluation took account of the fact that the quality of the execution of the construction (workmanship) was not perfect and that errors had been made when erecting the buildings (eg some reinforcing bars was not correctly located in joints, erection was not fully compliant with rules for the lapping of bars or for their anchorage, etc.).

Taken together the above work indicated that:

- Apart from the potential incidence of fire, the historical data suggest that the main concern for LPS dwelling blocks over five storeys high arises from internal gas explosions. This is taken as a reasonable basis for a structural assessment to be undertaken for accidental loads.
- A building without a piped gas supply was unlikely to experience an explosion that is more onerous than the ‘severe’ explosion category.
- The probability of occurrence of a ‘severe’ internal gas explosion in a dwelling involving cylinder gas or other gaseous substance is very low (circa 0.1 x 10⁻⁶ per annum).
- The three LPS dwelling blocks load tested by BRE should have been able to resist the overpressure loadings associated with a ‘severe’ internal gas explosion.
- The probability of a progressive collapse in an LPS dwelling block caused by a ‘severe’ internal gas explosion involving cylinder gas or other gaseous substance is expected to be less than about 0.1 x 10⁻⁶ per annum (assuming that 20% of such explosions exceed the collapse resistance of the LPS dwelling block concerned).

Thus, in an LPS dwelling block without a piped gas supply the annual probability of an individual fatality due to an incident such as an internal gas explosion (but excluding fires within the LPS dwelling block) is estimated to be below the threshold at which action might be required. On this basis the risks might be regarded as insignificant and adequately controlled’.

Thus in conclusion, the above findings suggest that it is rational for LPS dwelling blocks which are without a piped gas supply to be assessed on the basis of an accidental internal overpressure value of 17 kN/m²; and that this would allow the associated risks to be adequately controlled such that LPS dwelling blocks managed in this way could be considered to be ‘adequately safe’.

On the basis of the above, revised technical requirements for the structural assessment of LPS dwelling blocks are developed which are to be used in place of those given in MHLG Circulars 62/68[4] and 71/68[5]. The revised technical requirements are given in section 2 of this Handbook. Three options are given as potential ways to satisfy Requirement A3 of Approved Document A – Structure[9], namely the provision of:

- Suitable horizontal and vertical ties.
- Adequate collapse resistance.
- Means of mobilising alternative paths of support.

The concept of collapse resistance does not preclude the possibility of progressive collapse should the forces created during an internal gas explosions or by some other incident exceeded the collapse resistance of the LPS dwelling block concerned. However, such damage may not be disproportionate.
It is recommended that the through-life management of all LPS dwelling blocks should utilise risk assessment and reduction procedures to minimise the possibility of a ‘severe’ or ‘very severe’ explosion or from structural damage caused by other means. It is anticipated that on the basis of the methodology presented and the associated information provided, it should be possible to establish where risks may be regarded as insignificant and adequately controlled'. Where they are not, strengthening or other responses, such as risk reduction measures, should be considered.

It should be noted that this Handbook has been written primarily from the perspective of The Building Regulations 2010[2] for England and Wales. However, it should be recognised that Scotland and Northern Ireland are governed by separate legislation. Whilst the general objectives and requirements are similar, there are subtle differences. In spite of this the principles involved will be generally applicable.

14.3 SOME POINTS FROM AN OWNER’S PERSPECTIVE

Owners of LPS dwelling blocks have ongoing through-life management responsibilities and, many years after the Ronan Point incident, they find that these responsibilities still create ‘live issues’, especially when blocks become due for remedial or refurbishment works.

Local authorities are obliged to have a comprehensive knowledge of the structure and condition of their LPS dwelling blocks. However, they have been under pressure on several fronts: from leaseholders trying to safeguard their investment; from tenants wanting improvements; and from their own housing officers who wish to establish a clear housing investment strategy. Good historical data have not always been available, because it may have been lost or because earlier surveys were not sufficiently comprehensive. This may mean that a new structural assessment of the LPS dwelling block is necessary.

The structural assessment itself involves costs (direct and indirect) and potentially disturbance and disruption to residents. If carried out according to BRE guidance, several vacant dwellings are needed for opening up of the structure; drilling/breaking-out causes intrusive noise throughout the block and tenants need to be kept informed of what is going on, without causing alarm.

If, following structural assessment, strengthening is recommended then residents and other interested parties will commonly question why this was not done before. It may be necessary to explain that some early surveys were carried out in an age when professionals were perhaps less constrained by indemnity insurance issues. It has also been suggested that in more recent years engineers may have become less willing to interpret results favourably.

Because of the major consequences of any need for strengthening, owners may demand a second opinion, leaseholders may become suspicious of what they were told (or not told) when they bought their properties and, if works proceed, then tenants will be obliged to pay through their rents for work which causes further disruption but does not offer them any direct usable benefit.

In the past owners have questioned the need for strengthening and whether the structural assessment criteria values of 17 kN/m² and 34 kN/m² for accidental loading were soundly based. They have also posed the question whether, as there has been no major incident since Ronan Point, a very severe gas explosion remains a real risk.

Typically owners want a logical, consistent and legal defensible position. However, remedial or strengthening proposals which seem to err excessively on the safe side, and which would be very expensive and disruptive to implement, may be hard to justify to others such as technically lay (on engineering matters) oversight committees and residents.

The structural assessment process has expanded to consider risk, particularly as many LPS dwelling blocks are able to satisfy the assessment criteria if piped gas is absent, but fail when piped gas is present. Previously, housing officers have indicated that the provision of piped gas could provide a cheap heating source, which would improve the ease with which flats in LPS dwelling blocks could be let. The ability to minimise the cost of heating was seen as an immediate benefit by residents, with the vast majority of residents apparently being unworried by the risk that an internal gas explosion or another hazard might potentially cause structural collapse. Put simply, people’s perception has been that their LPS dwelling block would not fall down.

Interestingly risk analyses have shown that electricity kills more people than gas explosions. Some people have, perhaps quite reasonably, suggested that this consideration allied to the ability to reduce the cost of heating should form part of the benefit considerations in any risk assessment procedure.

Risk analysis has shown that owners can take reasonable steps to improve the safety of gas installations and that this should be part of a through-life management regime for the LPS dwelling block.

Once convinced that strengthening works are unavoidable, owners have often sought to reduce its disruptive effect by decanting residents to other properties or buildings, operating buy-back schemes for leaseholders and by taking great care in the phasing of the works.

At the end of the process, and given other pressures on their budgets, owners have indicated that they found themselves still doubting, in spite of all the investigation and assessment effort expended, whether strengthening works were really necessary.

It is hoped that the adoption of a risk-based approach to the through-life management of LPS dwelling blocks, together with the additional insight provided by the BRE load testing of existing LPS dwelling blocks and associated information on assessment methodology, will allow any necessary strengthening or remedial works to be closely targeted at locations giving clear enhancement of the safety of the LPS dwelling block concerned.

14.4 SOME ISSUES FROM A CONSULTING ENGINEER’S PERSPECTIVE

There are a number of important considerations relating to the role and responsibilities of the consulting engineer commissioned undertake a structural assessment of an
LPS dwelling block. The consulting structural engineer should have a significant knowledge of this particular field of work, in particular progressive collapse issues and understanding of the forms of construction of LPS dwelling blocks.

The starting point for the consulting engineer must be to establish what the client wants and to agree a clear brief for his activities. These should be related not only to the specific technical requirements for the structural assessment of an LPS dwelling block as described in this document, but also consideration will be given to the more general aspects of making a structural assessment. If necessary the engineer should meet the client, ideally at the site, and through pertinent questions establish all aspects of needs to be included in the brief. The engineer should take account of all background information that may be relevant, as well as clarifying the technical issues. It is important for the engineer (as well as the client) to establish a clear understanding of the scope of the structural assessment, its limitations and above all those matters upon which recommendations are required to be made.

The engineer may be best placed to prepare a draft brief once he has established the client’s needs. When reaching an agreed brief, the engineer must clearly act professionally and have the client’s ‘best interests’ at heart. Detailed investigation and structural assessment of existing LPS dwelling blocks can be a complex and time consuming task, often taking an extended period to complete due to the lack of availability of suitably placed vacant properties.

It is suggested that a client should not commission structural assessment work of this nature on a purely lowest cost tender basis. It is likely to be a false economy. If a fee is cut down to the minimum for commercial reasons, the engineer will have to simplify his assessment work and recommendations are then likely to err on the conservative side.

The structural assessment should be planned and arranged. This should include any specialist input required, such as physical investigations and material tests. However, the work should proceed in the knowledge that changes may become necessary, subject to findings on site or the revelations of archive drawings as they are studied, etc. The technical and commercial implications of such changes will need to be explained to the client and his agreement to the proposed changes in the work programme/scope of works should be obtained.

The investigative and survey work on site will probably be carried out with residents in occupation. Accordingly appropriate care and liaison will be needed. It may be necessary to meet with residents groups, probably in conjunction with the owner/client, and the planned work activities explained to residents in straightforward terms. Work on site to expose joint details will always be noisy and the difficulties involved should not be underestimated.

The extent of the investigative and survey work necessary is often a difficult issue and all parties should be aware that a structural assessment can only ever be based on samples, albeit hopefully on a reasonably representative number of them. Guidance on the rate of sampling is given in the BRE Report 107 The structural adequacy and durability of large panel system dwellings.[2]

but each case will need to be assessed on its own merits or restrictions. It may be that only a limited number of ‘void’ flats are available in which to carry out detailed exploratory work. It will be necessary for the engineer to highlight in his report the limitations of the investigative and survey work, including the obvious issues associated with the limited availability of ‘void’ flats/locations in which sample could be taken.

Owners and residents may well ask if their building complies with ‘current’ standards; a difficult but understandable question which warrants a careful response.

The engineer may also need to acknowledge and deal with the sometimes emotive approaches that can be received from residents. Questions are understandably asked by concerned residents. These may include questions along the lines of ‘Is my building safe’ and ‘Is my building wearing out’, especially when deterioration of the concrete and/or building fabric is apparent. Such questions can be difficult to answer. However, with the owner’s agreement, it has generally been found the best course of action is to offer well considered, honest and clear responses.

It may also be necessary to recognise that there can be substantial differences in the opinions of engineers appraising the same buildings.

The Institution of Structural Engineers report Appraisal of existing structures[18] provides guidance on issues such as the formulation of the brief for the commission, the more general aspects of undertaking a structural assessment and the reporting of results and recommendations.

14.5 FINALLY – A NOTE OF CAUTION!
Whilst the historically observed performance of LPS dwelling blocks has in general been satisfactory and the results of the BRE load tests performed to the ultimate condition on three (uninhabited) LPS dwelling blocks (two Bison Wallframe and 1No Reema Conclad) were reassuring, the potential vulnerability of this form of construction has been highlighted by the recent progressive collapse of parts of a number of LPS dwelling blocks during demolition[6].

The LPS dwelling blocks concerned were pre-Ronan Point Fram Russell LPS. Limited investigation of one of the collapses by BRE suggested that the LPS dwelling blocks concerned could have been constructed to a relatively poor standard. Particular points were the quality of the tying at the floor slab/cross wall panel junctions, where there was a significant lack of connection (mechanical tying) between floor panels and the cross walls forming a bay, and the lack of end bearing to the floor slabs. BRE is aware that similar LPS dwelling blocks located in another part of the UK have been successfully deconstructed without incident.

However, these incidents do highlight the need to be cautious when making a structural assessment and the critical need for adequate invasive investigation of the nature and quality of construction achieved in individual LPS dwelling blocks before embarking upon extensive structural calculations in the structural assessment process.


BRE Internal Supporting Documents. Of the following reports, only D is published and, therefore, publicly available, the others are internal reports.
A Structural damage in buildings caused by gaseous explosions and other accidental loadings. BRE CP45/74, 1974.

B The hazards of explosion, impact and other random loadings on tall buildings. BRE CP64/74, 1974.


F Nicholson H G and Buller P S J. The incidence of damage to buildings caused by explosions and vehicle impacts. BRE Internal Note N130/92, 1992.


35 London Borough of Newham vs Taylor Woodrow-Anglian Ltd. Court of Appeal (Civil Division), 9 July 1981.


16 BIBLIOGRAPHY AND FURTHER READING

This information is provided under the following headings:

- Fire.
- Concrete and reinforced concrete as a material.
- Large panel blocks and related topics.
- Building regulation, legislation and related guidance/issues.
- Codes for structural design/design of concrete structures.
- Loading and actions.
- Inspection, monitoring and assessment of existing structures.
- Structural safety and risk assessment.
- Progressive/disproportionate structural collapse.
- Concrete durability, protection and repair of concrete structures.
- Demolition, structural alteration and refurbishment.

Within these sections the individual items are presented in chronological order.

Fire


Concrete and reinforced concrete as a material


Large panel blocks and related topics


Bison cladding falls worry councils. New Civil Engineer, 5 January 1978, p. 5.

Blacker G B. Bison Wall frame flats and houses, Department of the Environment (DoE) Circular letter about safety concerns regarding cladding panels on these blocks to all Local Authorities. London, DoE, October 1983.


BSI. The structural use of precast concrete: Part 2 – Metric Units. CP 116. London, BSI 1969 (NB. Part 1 was Imperial Units).


Concrete Ltd. The Bison Wall Frame system for high flats. London, Concrete Ltd, publicity material, November 1963.

Concrete Ltd. 137 Tests on large panel construction, Bison Bulletin No. 7. London, Concrete Ltd, 1968.


Full-scale strength tests on large panel construction. Precast Concrete, October 1970, pp 283–290.

Gas remains a worry on system estates. New Civil Engineer, 25 September 1986, 20–21.


Hilti (Great Britain) Ltd. Fixing method selected for strengthening tower blocks – GLC's solution to Ronan Point problem. Manchester, Hilti (Great Britain) Ltd, undated publicity material.


London Borough of Newham vs Taylor Woodrow-Anglian Ltd. Royal Courts of Justice, Queen's Bench Division, Judgement, 21 December 1979.

London Borough of Newham vs Taylor Woodrow-Anglian Ltd. Courts of Appeal (Civil Division), 9 July 1981.


Reema Construction Ltd. Appraisal of tall blocks: Large panel construction. Test reports on the strength of gable wall joints of the three 16 storey ‘scissor-type’ blocks of flats in Southampton, E W H Gifford and Partners. Date believed to be July 1971.


Watson V and Hurst M J S. Experiments for the design of a system using large precast concrete panels components. The Structural Engineer, 1972 50 (9), 345–354.


Building regulation, legislation and related guidance/ issues


London Borough of Newham vs Taylor Woodrow – Anglian Ltd. Royal Courts of Justice, Queen’s Bench Division, Judgement, 21 December 1979.

London Borough of Newham vs Taylor Woodrow – Anglian Ltd. Courts of Appeal (Civil Division), 9 July 1981.


Codes for structural design/design of concrete structures


BSI. The structural use of reinforced concrete in buildings: Part 2 – Metric Units, CP 114. London, BSI, 1969. (NB. Part 1 was Imperial Units.)

BSI. The structural use of precast concrete: Part 2 – Metric Units, CP 116. London, BSI, 1969. (NB. Part 1 was Imperial Units.)


Loading and actions

BRE. Structural damage in buildings caused by gaseous explosions and other accidental loadings. BRE CP45/74. Watford, BRE, 1974.

BRE. The hazards of explosion, impact and other random loadings on tall buildings. BRE CP64/74. Watford, BRE, 1974.


Ellis B R, Crowhurst D. The response of several LPS maisonettes to small gas explosions, Joint ISE/BRE seminar Structural design for hazardous loads: The role of physical testing, Brighton, 1991.


Ellis B R, Crowhurst D. The response of several LPS maisonettes to small gas explosions, Joint ISE/BRE seminar Structural design for hazardous loads: The role of physical testing, Brighton, 1991.


Nicholson H G and Buller P S J. The incidence of damage to buildings caused by explosions and vehicle impacts. BRE Internal Note N130/92, 1992.


**Inspection, monitoring and assessment of existing structures**


**Structural safety and risk assessment**


Mann A. Construction safety: An agenda for the profession. The Structural Engineer, 1 August 2006, 28–34.


Progressive/disproportionate structural collapse

Brooker O. How to design concrete buildings to satisfy disproportionate collapse requirements. Camberley, The Concrete Centre, 2008.


Keane W and Esper P. Forensic investigation of blast damage to British buildings. Proceedings of the Institution of Civil Engineers, Civil Engineering, 2009, 162 (Special Issue 2), 4–11 (Special issue on Forensic Engineering).


Concrete durability, protection and repair of concrete structures


Demolition, structural alteration and refurbishment


There are eleven Appendices, as detailed below:

**Appendix A:** Development of ‘Regulatory’ requirements.

**Appendix B:** Hazard environment.

**Appendix C:** Risk issues.

**Appendix D:** Outline historical review of LPS dwelling blocks in the UK since 1968.

**Appendix E:** BRE tests on LPS dwelling blocks and laboratory structures pre-2000.

**Appendix F:** Overview of finite element analyses and calibration exercises for the 1990s’ BRE load tests on LPS dwelling blocks.

**Appendix G:** BRE load testing of an existing Bison Wallframe block, Liverpool.

**Appendix H:** Overview of finite element analyses and calibration exercises for the Liverpool Bison Wallframe block tested by BRE.

**Appendix I:** Strengthening options.

**Appendix J:** LPS dwelling block assessment case studies.

**Appendix K:** BRE trials to determine the coefficient of friction at the base of wall panels.

These Appendices present an overview of a number of key subject areas including the historical background to LPS dwelling blocks within the UK and associated guidance, together with a summary of previously unpublished information on load testing undertaken by BRE on LPS dwelling blocks or assemblies of components, spanning the last three and a half decades.

Summary information is presented on various strengthening techniques that have been considered or employed to enhance the strength and/or robustness of LPS dwelling blocks.

Case histories of several consultancy commissions have been included to demonstrate the various approaches that have been adopted by BRE and by other consultants when assessing various buildings of this type. A number of these case histories also include references to the remedial and/or strengthening works techniques that were adopted.
APPENDIX A

DEVELOPMENT OF ‘REGULATORY’ REQUIREMENTS

A1  INTRODUCTION
This Appendix reviews the historical development of the technical requirements for the structural assessment of existing LPS dwelling blocks and seeks to put these into context by considering the evolution of contemporary regulatory requirements, as given in Approved Document A – Structure\(^4\), for reducing the sensitivity of all new buildings to disproportionate damage.

Section A2 of this Appendix describes the historical development of the requirements for the structural assessment of existing LPS dwelling blocks for normal and accidental loads. Following the partial progressive collapse of Ronan Point in May 1968, existing LPS dwelling blocks have effectively been treated as a special class of building with specific guidance and requirements being defined for their structural assessment and aspects of their management.

Section A3 of this Appendix consideration is given to the development of the requirements for the design of new buildings, as given in Approved Document A – Structure\(^5\), to reduce their sensitivity to disproportionate damage in the event of an accident. The approaches include:

- The provision of continuity within the building in the form of vertical and horizontal ties.
- The potential response of the building to specific forms of damage (ie the notional loss of individual vertical load bearing elements such as walls and columns) and whether the structure can mobilise alternative load paths to bridge over the notionally damaged zone and thereby maintain its overall stability.
- Consideration of the forces arising when critical elements of the building are subject to ‘notional’ accidental loads or actions (ie the design of ‘key’ elements for specified loads).

The following parts of this Appendix (sections A4 to A6) concerns related regulatory requirements defined in the Workplace (Health, Safety and Welfare) Regulations 1992 (section A4) and also those in the Construction (Design and Management) Regulations (section A5), to establish if these statutes impose additional requirements that need to be taken into account when undertaking a structural assessment of an LPS dwelling block. Section A6 considers whether there may be other legislation which imposes additional obligations and/or constraints upon those undertaking a structural assessment or construction works upon an existing multi-storey LPS dwelling block. Two examples of such legislation are considered, namely The Defective Premises Act 1972 and The Occupiers Liability Act 1984.

A2  DEVELOPMENT OF THE REQUIREMENTS FOR THE STRUCTURAL ASSESSMENT OF EXISTING LPS DWELLING BLOCKS

After the partial progressive collapse of Ronan Point in 1968, a Tribunal was established under Section 318 of The Public Health Act 1936 and Section 290 of The Local Government Act 1933, to investigate the cause or causes of the collapse, to consider the implications of the findings and to make appropriate recommendations.

Tests and detailed investigations were carried out upon certain elements of the structure and the collapse debris to resolve a number of questions relating to matters such as the cause of the explosion, the likely strength of key components and associated joints, explosion pressures, etc. One of the key questions that the Tribunal sought to answer was ‘what pressure was required to displace a flank wall panel’ in the Ronan Point LPS dwelling block.

The Tribunal’s report\(^1\) was published towards the end of 1968.

The Tribunal made a number of recommendations affecting system-built blocks of over six storeys (ie seven storeys and above) in height. These included measures to:

- strengthen Ronan Point,
- appraise and, if needed, to strengthen other existing LPS buildings,
- factors to be taken into consideration when designing new LPS buildings.

There were also interim recommendations relating to the disconnection of piped-gas supplies from LPS dwelling blocks located in the UK. In spite of this, over the intervening years BRE has discovered a number of LPS dwelling blocks where the piped-gas supply had not been disconnected, even though the blocks concerned had not been assessed as being sufficiently strong to resist the specified overpressure loads associated with a piped gas supply (ie an overpressure of 34 kN/m\(^2\)).

Following these recommendations, in late 1968 the then Ministry of Housing and Local Government (MHLG) issued advice to local authorities via MHLG Circulars 62/68\(^2\) and 71/68\(^3\). The advice given to building owners at that time was to appraise all their LPS dwelling blocks of 7 storeys and above for their susceptibility to progressive collapse. Owners were required to consider...
whether strengthening was necessary. Buildings were to be appraised using an equivalent static pressure of 3 lb/in² (≈ 34 kN/m²), but where the building was without a piped gas supply the assessment criterion could be halved to 2.5 lb/in² (≈ 17 kN/m²).

It was indicated that compliance with the requirements could be achieved either by providing:

Method A. Alternative paths of support to carry the load, assuming the removal of a critical section of the load-bearing walls, or

Method B. A form of construction of such stiffness and continuity so as to ensure the stability of the building against forces liable to damage (but not fail) the load-supporting components (i.e. to be sufficiently strong to resist the applied accidental loads).

These two approaches are significantly different in concept. They currently remain the approaches considered in contemporary structural assessment procedures for existing LPS blocks.

The Institution of Structural Engineers prepared document RP/68/02[10] to provide guidance to assist in the interpretation of MHLG Circular 62/68[1]. This document (RP/68/02) was subsequently officially circulated to local authorities and their professional advisers as MHLG Circular 71/68[11].

The Institution of Structural Engineers also prepared document RP/68/01[11] to provide more general guidance upon issues of structural stability and the prevention of progressive collapse. Whilst these recommendations were concerned primarily with LPS type dwelling blocks, they were also applicable in principle to other forms of concrete structure constructed of precast concrete components and other tall buildings without a structural framework (e.g. load bearing masonry).

The MHLG Circulars[4,5] make no mention of the risk of the failure of one or more floor panels resulting in the vertical progressive collapse of the floors in part of an LPS dwelling block, nor do they mention the implication of the removal of lateral restraint to the loadbearing walls. However, the issue of the lateral restraint of loadbearing walls was recognised by the Institution of Structural Engineers and was mentioned in at least one discussion paper[12] of the time.

Summarising, it was necessary to check whether an LPS dwelling block was able to accommodate the forces associated with a static overpressure of 5 lb/in² where piped gas was present, or 2½ lb/in² where it was not. The metric equivalent of these overpressures is ≈ 34 and ≈ 17 kN/m², respectively. The metric or imperial equivalent overpressures are taken to be interchangeable (units used will depend on published date of document being referenced).

The Tribunal also warned about the risks of storing explosive substances such as liquefied petroleum gas (LPG), commonly referred to as bottled gas or cylinder gas. This warning was repeated in MHLG Circular 62/68[1].

The standards set also applied to the design of new LPS blocks pending revision of Building Regulations and the relevant Codes of Practice for structural design.

In 1970, provisions to resist progressive collapse based on the 5 lb/in² (≈ 34 kN/m²) accidental loading criterion were introduced in Section D17 of the Building Regulations[13] for blocks of five storeys and above. It was at this time that the height threshold for considering progressive collapse and accidental loading issues in new construction changed by two storeys. That is from being buildings of seven storeys or more high, to being buildings of five storeys or more high. It is understood that this change was not reflected in the available guidance for the structural assessment of LPS dwelling blocks[15].

Also in 1970, Addendum No 1 to CP 116: Part 2: 1969 was issued by BSI, updating advice given in the Code of Practice in line with that given in the MHLG Circulars[4,5], related material published by the Institution of Structural Engineers[13,58], the Building (Fifth Amendment) Regulations 1970[14] and Section D17 of the Building Regulations[15], discussed above.

In addition to the changes introduced to the Building Regulations[10,14], parallel regulations in the form of local by-laws were put in place by the former Greater London Council (GLC) following the collapse of Ronan Point. These parallel regulations stayed in force until the abolition of the GLC in April 1986. The GLC by-laws concerning disproportionate damage criteria in buildings applied only to the inner London area. They were published by the GLC as the London Building (Constructional) By-law[16].

BRE subsequently published a number of reports relating to issues such as structural adequacy and durability, habitability, overcladding and weathertightness of LPS dwelling blocks. These publications formed outputs from a research programme funded by the then Department of the Environment. BRE Report 107: Part 2[17], published in 1987, gave guidance on the assessment of structural adequacy and durability of LPS dwellings. This BRE report defined the requirements for a structural assessment of an LPS dwelling block, offering non-mandatory guidance on the assessment of structural adequacy, sampling and inspection of structural connections, assessment of findings and procedures for evaluation of the future durability of the structural components and connections. Consideration was given to the accidental loads which might be applied to an LPS dwelling block, the possible forms of progressive collapse and the significance of connections/tying reinforcement between precast structural components.

BRE Report 107: Part 2[17] indicates that:

- Adequate robustness can be considered to exist if there is an adequate provision of horizontal and/or vertical ties (in terms of their spacing and load capacity); that is the provision of horizontal or vertical ties meets the prescribed rules for new buildings.
• Stability will be maintained if ‘key’ structural elements of an LPS dwelling block are able to resist the foreseeable accidental load from an internal gas explosion; with the local strength of the structural elements being checked against the recommended equivalent static pressures of 17 kN/m² for a building without a piped gas supply and 34 kN/m² for buildings which have a piped gas supply. Thus, in this situation local failure is assumed to be avoided if the structural assessment criteria are satisfied (ie the wall and floor panels and associated joints are considered strong enough to resist the applied accidental loads).

• In the event of the building failing to satisfy either of the above criteria, then strengthening should be considered to either:
  * enhance the local strength of the structural elements such that failure is not initiated under the recommended equivalent static pressures (as defined above) associated with the foreseeable accidental load, or
  * facilitate the mobilisation of alternative load paths in the event of local damage being caused to the building structure.

The 1996 version of the Institution of Structural Engineers report *Appraisal of existing structures* repeated the advice that the assessment criterion for key elements could be halved (17 kN/m²) for LPS dwelling blocks where piped gas was absent and, importantly, that it was certain the building would remain without a piped gas supply in the future.

BRE Report 107 recommended that a full structural assessment of a complete LPS dwelling block be carried out every 20 years. For buildings of 5 storeys and more (including basement storeys) the assessment was to be made in respect of both normal and accidental loads.

Amongst other recommendations made in BRE Report 107 was that supporting visual inspections of the external envelope of the building should be made at intervals of about 5 years, together with intrusive investigations every 10 years to check upon the condition of the reinforcement within in-situ joints which might experience rain penetration (eg at flank wall and roof locations).

Figure A1 presents the changes that have taken place since 1968 in relation to the requirements for structural assessment relative to the height of LPS dwelling blocks. It is important to note the change in the height of buildings that were required to be checked for adequacy in respect of accidental loads between the MHLG circulars published in 1968, and the *Building (5th Amendment) Regulations, 1970*. Figure A1 also indicates that some uncertainty remains as to whether structural assessments were carried out for accidental loads for all five and six storey LPS dwelling blocks during the early 1970s, or subsequently. For completeness, Figure A1 also shows the contemporary requirements as defined in Approved Document A – Structure, in the restricted context of the circumstances relating to LPS dwelling blocks. These requirements are discussed further below.

### A3 EVOLUTION OF THE REQUIREMENTS FOR THE DESIGN OF NEW BUILDINGS TO REDUCE THEIR SENSITIVITY TO DISPROPORTIONATE DAMAGE IN THE EVENT OF AN ACCIDENT

We will now turn our attention to the development in the requirements for the design of new buildings to reduce their sensitivity to disproportionate damage in the event of an accident. Some aspects of these requirements overlap with those discussed in the previous section of this Appendix which examined the development over the years of the requirements for the structural assessment of existing LPS dwelling blocks. However, this Appendix looks at the wider implications of the requirements and the different concepts now being applied in respect of new buildings.

In the 1970 revision of the Building Regulations provisions to resist progressive collapse were introduced. These were based on the 5 lb/in² (≈ 34 kN/m²) accidental loading criterion for buildings of 5 storeys and above. This requirement had to be applied irrespective of whether there was a piped gas supply to the building or not.

As noted above, following the collapse of Ronan Point local by-law regulations were put in place by the former Greater London Council (GLC) in respect of buildings in the inner London area. These by-law regulations, which existed in parallel with the Building Regulations, stayed in force until the abolition of the GLC in April 1986. The GLC by-law requirements concerning disproportionate damage criteria in buildings applied only to the Inner London area and were published in the *GLC London Building (Constructional) By-laws*. While being similar to the criteria in the national regulations, the GLC by-laws included some variations relating to particular types of building structures. The principal difference was in respect of party walls to buildings of two storey and over which were required to be designed to resist a lateral pressure of 1 lb/in² (7 kN/m²) with a load factor of 1.05 at the ultimate limit state, unless these walls had a mass greater than the equivalent of a 200 mm thick brick wall (assuming a density of some 16 kN/m³). Importantly this requirement could apparently be complied with relatively easily and for only a small additional cost.

Also, the Inner London Education Authority required that all new school premises of two storeys and over built and owned by the authority should comply with the disproportionate damage criteria set down in the GLC’s by-laws. Thus all school buildings of two storeys and fewer than five storeys in height were required to be designed to resist a lateral pressure of 1 lb/in² (7 kN/m²).

---

31 BRE Report 107: Part 2 did not consider other potential options such as venting of explosion overpressures, which might now be considered as possible approaches in some situations.

32 Please note that the current Handbook (ie this document) gives slightly different guidance about the recommended frequency of inspections, testing and structural assessments for LPS dwelling blocks.

33 These requirements were in response to concerns that reinforcement within such in-situ joints could experience corrosion, especially in the presence of significant levels of chlorides. The chlorides might have been cast into the concrete at the time of manufacture of the components/ construction of the LPS dwelling block, added as an accelerator (eg as calcium chloride) to the concrete mix, and/or have ingressed into the concrete where blocks are situated at coastal locations.
Subsequent Building Regulations\textsuperscript{[9,22]} have all included a requirement that ‘the building shall be constructed so that in the event of an accident the building will not suffer collapse to an extent disproportionate to the cause’. This has been embodied as Requirement A3 which applied only to a building having five and more storeys (each basement level being counted as one storey) excluding a storey within the roof space where the slope of the roof does not exceed 70° to the horizontal. Where the roof slope is in excess of 70°, the storey was included.

The philosophy which underpinned this latter amendment was that a structure was required to be designed so that if any single ‘component’\textsuperscript{33} was damaged or dislodged the load it carried would be transferred to

\textsuperscript{33} This is a design procedure/assumption and it does not necessarily mean that any building would actually behave in this way. This issue highlights one difference between structural design and structural assessment procedures. The notional design case is applied to elements sequentially and is not intended to simulate the effects of a gas explosion producing an overpressure loading.
adjacent components which were able to (safely) sustain the additional loads. This of course assumes that only one ‘component’ was lost. Whilst it is not clear how the decision to assess the behaviour of a building for the loss of a single ‘component’ was originally arrived at; this empirical measure is considered to have had the desired effect in that the general population of buildings achieves a satisfactory degree of resistance to collapse arising from local damage to buildings. Logically it is desirable that the probability of failure should be related to the amount of damage.

Contemporary guidance on the potential extent of acceptable damage is given under Requirement A3 of Approved Document A – Structure[9]. Clause 5.1d states: ‘... upon the notional removal of each supporting column and each beam supporting one or more columns, or any nominal length of load-bearing wall (one at a time in each storey of the building) that the building remains stable and that the area of floor at any storey at risk of collapse does not exceed 15% of the floor area of that storey or 70 m², whichever is smaller, and does not extend further than the immediately adjacent storeys (ie storey immediately above and the one below).’

This extent of damage, that is the area at risk of collapse in the event of an accident, is illustrated in Figure A2. In the context of an LPS dwelling block, this requirement essentially limits the extent of collapse to that associated with an incident originating in a single dwelling (ie an explosion within a single flat) causing damage to that dwelling and to the ones above and below.

Clause 5.1d of Approved Document A – Structure[9] goes on to state that: “Where the notional removal of such columns and lengths of walls would result in an extent of damage in excess of the above limit, then such elements should be designed as a ‘key element’...”[34]

For concrete buildings BS 8110 Part 1[61] sets down its interpretation of these requirements, noting in Clause 2.2.2.2 Robustness that: “Structures should be planned and designed so that they are not unreasonably susceptible to the effects of accidents. In particular, situations should be avoided where damage to small areas of a structure or failure of single elements may lead to collapse of major parts of the structure.”

BS 8110 Parts 1 and 2[61] give details of what constitutes effective horizontal and vertical ties, together with the design approaches for checking the integrity of the building following the notional removal of a vertical load bearing component and the design of key elements.

However, the introduction of the structural Eurocodes into the UK[9] has brought a slightly different perspective to meeting the requirements set down in Approved Document A – Structure[9]. The DCLG has indicated[9] that the structural Eurocodes are now an acceptable way of satisfying Approved Document A – Structure[9]. Some of the implications of this development are considered below.

In terms of the general population of buildings, the current design philosophy with respect to the consequences of an accident and the sensitivity of a building to disproportionate damage, which underpins Approved Document A – Structure[9], now incorporates the concept of ‘Building Class’ (categories 1 to 3). Under Approved Document A – Structure[9] buildings of five storeys and above are now categorised as either Class 2B (5–15 storeys) or Class 3 (>15 storeys).

The requirements for Class 2B and Class 3 buildings are set down in paraphrased form below.

**For Class 2B buildings – There is a requirement to provide:**
- Effective horizontal ties as described in the relevant British Standards for framed and loadbearing wall construction.
- Effective vertical ties as described in the relevant British Standards in all supporting column and wall (vertical load bearing) elements,

Or, alternatively:
- To check that upon the notional removal of each supporting column and each beam supporting one or more columns or any nominal length[37] of loadbearing wall (one at a time in each storey of the building) that the building remains stable and that the area of floor at any storey at risk of collapse does not exceed 15% of the floor area of that storey or 70 m², whichever is smaller, and does not extend further than the immediately adjacent storeys (ie storey immediately above and the one below).

Where the notional removal of such lengths of wall would result in an extent of damage in excess of the above limit, then such elements should be designed as a ‘key elements’ as defined previously (see footnote 35).

34 A ‘key element’ should be capable of sustaining an accidental design load of 34 kN/m² applied in the horizontal and vertical directions (in one direction at a time) to the particular unit and any attached components (eg cladding etc.) having regard to the ultimate strength of such components and their connections. Such accidental design loading should be assumed to act simultaneously with one-third of all normal characteristic loading (ie wind and imposed loading). See Clause 5.3, The Building Regulations 2000, Approved Document A – Structure[9].

35 The structural Eurocodes formally replaced the UK national standards for structural design in April 2010. At that time the British Standards Institution (BSI) was obliged to withdraw conflicting UK national structural design standards, some of which are referenced in the Building Regulations Approved Documents, particularly Approved Document A – Structure. The British Standards withdrawn on 31 March 2010 remain available from BSI. However, BSI committees had previously stopped updated those standards; so in the medium and long term they are expected to become less suitable for aspects of structural design.

36 In 2008 and again in January 2010 the DCLG wrote to Local Authorities and other bodies in England indicating that the structural Eurocodes are now an acceptable way of satisfying the current Approved Document A. The January 2010 DCLG Circular Letter is available at: http://www.communities.gov.uk/publications/planningandbuilding/dlevelandersignostandards

37 The nominal length of a loadbearing reinforced concrete wall is defined as the distance between lateral supports subject to a maximum length not exceeding 2.25H, where H is the storey height in metres. In the case of an external masonry wall, the nominal length is the distance between vertical lateral supports. In the case of an internal masonry wall, the nominal length is a maximum of 2.25H. See Clause 5.3, Approved Document A – Structure[9].
APPENDIX A  DEVELOPMENT OF ‘REGULATORY’ REQUIREMENTS  101

Figure A2: Area at risk of collapse in the event of an accident (This is a reproduction of Diagram 24, Approved Document A – Structure: Requirement A3\(^\text{[9]}\)), reproduced under the terms of the Click-Use Licence.

Note to Figure A2: It is understood that the limit upon the area of floor at risk of collapse will be increased to 100 m\(^2\) in a future edition of the Approved Document A – Structure (probably in the proposed 2013 review and revision) in order to be consistent with the recommendations given in the structural Eurocodes – see the later discussion in Appendix A of this Handbook relating to BS EN 1991-1-1-7: 2006: Eurocode 1 – Actions on structures: Part 1–7: General actions – Accidental actions\(^\text{[20]}\).

For Class 3 buildings – In addition to the requirements for Class 2B it is necessary to:

- Undertake a systematic risk assessment of the building taking into account all the normal hazards that may reasonably be foreseen, together with any abnormal hazards.
- Select critical situations for design that reflect the conditions that can reasonably be foreseen as possible during the life of the building. The structural form and concept and any protective measures should then be chosen and the detailed design of the structure and its elements undertaken in accordance with the relevant British Standards.

In addition, Clause 5.4 of Approved Document A – Structure\(^\text{[9]}\) permits what is described as an alternative approach. Again this is set down in paraphrased form below.

**Alternative approach** - for any building which does not fall into the defined classes (Reference Table 11 of Approved Document A – Structure\(^\text{[9]}\)) or for which the consequences of collapse may warrant particular examination of the risks involved, the performance may be met by the criteria given in other guidance available on the DCLG website\(^\text{[38]}\), namely:

- ‘Guidance on robustness and provision against accidental actions, together with the associated BRE Report’

- ‘Calibration of proposed revised guidance on meeting the compliance with the requirements of Building Regulation Part A3’

On basis of the classifications given in Approved Document A – Structure\(^\text{[9]}\), multiple occupancy residential buildings are categorised as follows:

Class 2A: Buildings not exceeding four storeys in height (ie four storeys or less).

Class 2B: Buildings greater than 4 storeys but not exceeding 15 storeys in height.

Class 3: Buildings that exceed the area and/or number of storey limits for defined for Classes 2A and 2B, thus for residential buildings this equates to buildings over 15 storeys high.

The introduction of the structural Eurocodes into the UK has brought a slightly different perspective to the requirements set down in Approved Document A – Structure\(^\text{[9]}\). In due course the structural Eurocodes will be referenced within Approved Document A – Structure\(^\text{[9]}\) as being an acceptable means of meeting compliance with these requirements.

In the context of the current discussion, the most relevant parts of the structural Eurocodes are:

- BS EN 1990: 2002: Basis of structural design\(^\text{[31]}\)

\(^\text{38}\) The Approved Document A research guides are available from the following website: http://www.planningportal.gov.uk/england/professionals/buildingreg/technicalguidance/bcstructuralsaftyparta/bcassociateddocuments
The requirements set down in structural Eurocodes generally reflect those of Approved Document A – Structure\textsuperscript{10}, however they are worded and presented somewhat differently. For example, the structural Eurocodes define and use the concept of Consequence Classes CC1–CC3. These are defined in Table B1 of BS EN 1990: 2002: Basis of structural design\textsuperscript{11} as follows: Consequence Class CC1. Low consequence for loss of human life and economic, social or environmental consequences are small or negligible. Consequence Class CC2. Medium consequence for loss of human life or economic, social or environmental consequences are considerable. Consequence Class CC3. High consequence for loss of human life or economic, social or environmental consequences are very great.

Table B1 of BS EN 1990: 2002: Basis of structural design\textsuperscript{11} also indicates that residential buildings would typically be considered to be within Consequence Class CC2. For buildings Consequence Classes CC1–CC3 are taken to be compatible with Classes 1 to 3 in Table 11 of Approved Document A – Structure\textsuperscript{10}.

The normative text of BS EN 1991-1-7: 2006\textsuperscript{20} is supplemented by a number of ‘informative’ notes and additions. The observations made include:

- That for buildings the recommended value for the uniformly distributed load to be used in accidental design situations be 34 kN/m\(^2\) – see Clause 3.3 of BS EN 1991-1-7: 2006\textsuperscript{20}.

- It is possible to have different elements of the same structure assigned to different consequence classes. The example given in Clause 3.4 of BS EN 1991-1-7: 2006\textsuperscript{20} highlights the case of a structurally separate low rise wing of a building that is serving a less critical function than the main building. Thus it is possible for only part of a structure to be classified as CC3 or CC2. Conversely Approved Document A – Structure\textsuperscript{10} advises that the more onerous Class should be used for the entire building.

- Annex A of BS EN 1991-1-7: 2006\textsuperscript{20} offers guidance on one of the two\textsuperscript{29} ‘recommended strategies’ which ‘should provide a building with an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse’ – see Figure A3. For buildings in Consequence Class CC3, Clause A4 states ‘A systematic risk assessment of the building should be undertaken taking into account both the foreseeable and unforeseeable hazards’. This is taken as compatible with the guidance given in Approved Document A – Structure\textsuperscript{10}.

- Clause A4 of Annex A of BS EN 1991-1-7: 2006\textsuperscript{20} indicates for buildings in Consequence Class CC3 that ‘The limit of admissible local failure may be different for each type of building. The recommended value is 15\% of the floor, or 100 m\(^2\), whichever is smaller, in each of two adjacent storeys’. This is different to the guidance given in Approved Document A – Structure\textsuperscript{10}, where the limit of floor collapse is currently defined as 70 m\(^2\), rather than the 100 m\(^2\) permitted in the structural Eurocode. The UK National Annex for BS EN 1991-1-7: 2006\textsuperscript{20} is now published and it recommends that an area of 100 m\(^2\) be adopted.

- Annex B of BS EN 1991-1-7: 2006\textsuperscript{20} offers guidance on risk assessment procedures for qualitative and quantitative risk analysis, together with risk acceptance and mitigation measures. Annex B of BS EN 1991-1-7: 2006\textsuperscript{20} indicates that the ALARP (As Low As Reasonably Practicable) principle is to be used to guide risk acceptance and mitigation measures, with various approaches and criteria being outlined. Appendix B indicates that risk acceptance levels should be specified and that they will usually be formulated on the basis of:
  - Individual risk – with the risks to an individual being expressed as a fatal accident rate, an annual fatality probability or as the probability per unit time of a single fatality when involved in a specific activity.
  - Socially acceptable risk to human life – often presented as an F–N curve, indicating a maximum yearly probability F of having an accident with more than N causalities.

- Annex B of BS EN 1991-1-7: 2006\textsuperscript{20} also indicates that alternatively, concepts such as ‘value of a prevented fatality’ (VPF) or a ‘quality of life index’ might be used.

- Appendix D of BS EN 1991-1-7: 2006\textsuperscript{20} provides supplementary information for internal explosions, with Clause D2 presenting equations to estimate the nominal equivalent static pressure associated with natural gas explosions taking into account venting effects. The nominal equivalent static pressure is considered to act simultaneously on all the bounding surfaces of the room or enclosure within which the explosion (deflagration) occurs.

Figure A3 sets out the two recommended strategies given in BS EN 1991-1-7: 2006\textsuperscript{20} which should provide a building with an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse. Additionally, Figure A3 classifies the six accidental design situations portrayed as ADS1 to ADS6. In the context of LPS dwelling blocks, two approaches are identified:

- Resistance measures: Utilising the concept of collapse resistance of an LPS dwelling block, relating to accidental design situations ADS3 and ADS5.

- Preventive measure: Avoidance of explosive substances, gases, etc. in an LPS dwelling block, relating to accidental design situation ADS2.

\textsuperscript{29} The two recommended strategies are defined in Figure 3.1 of BS EN 1991-1-7: 2006\textsuperscript{20} entitled ‘Strategies for accidental design situations’. The two strategies given are:

1. Strategies based on identified accidental actions, eg explosions and impact.
2. Strategies based on limiting the extent of localised failure.

Workplace legislation has also introduced responsibilities in respect of the through-life structural performance of buildings. This is concerned with their stability and solidity and was introduced as an amendment of the Workplace (Health, Safety and Welfare) Regulations 1992. This amendment was introduced within Statutory Instrument 2002 No 2174, The Health and Safety (Miscellaneous Amendments) Regulations 2002\[62\]. The amendment was made by the introduction of Regulation 4A which states:

Where a workplace is in a building, the building shall have a stability and solidity appropriate to the nature of the use of the workplace.

So far there appears to be no official or agreed definition of the meaning of the terms ‘stability’ and ‘solidity’, and, accordingly, these appear to remain open to interpretation.

The applicability of this legislation would appear to revolve around whether an LPS dwelling block would be considered to be a workplace and the circumstances under which this might apply or, indeed, might not apply. It is understood that common parts of buildings are treated as workplaces.

This UK legislation is linked with European legislation on the same topic, namely The Workplace Directive (Council Directive 89/654/EEC of 30 November 1989)\[63\], which is concerning the minimum safety and health requirements for workplaces.

More generally the Health and Safety at Work, etc. Act 1974 imposes duties to maintain premises in a safe condition. Under Section 2 a duty is imposed on employers to do whatever is reasonably practicable to maintain any place of work under their control so that it is safe and that means of access to and egress from it are safe for his employees. Section 4 imposes a similar duty on any person who has to any extent control of non-domestic premises, to ensure the safety of persons who are not his/her employees but use the premises made available to them as a place of work. These duties are elaborated in the Workplace (Health, Safety and Welfare) Regulations 1992.

Section 3 of the Health and Safety at Work, etc. Act 1974 imposes a broader duty on an employer to ‘conduct his undertaking in such a way as to ensure, so far as is reasonably practicable, that persons not in his employment are not thereby exposed to risk to their health and safety’. This does create duties to maintain the premises for the safety of visitors and the public who might be affected (eg passers-by). Breach of the above duties is a criminal offence.
There are also legal duties associated with construction work. In the UK the Construction (Design and Management) Regulations 2007[64] – which are generally known as the CDM Regulations – place legal duties upon virtually everyone involved in construction work[40]. These duties are wide ranging in respect of the health and safety of persons involved in or who may be affected by the works; with considerations including the design and methodologies for the construction of the works; as well as their subsequent operation during the envisaged service life of the building.

The following extract is taken from Part 2: General management duties applying to construction projects of the CDM Regulations[64]. Clause 11 sets down the duties of designers as defined within this legislation. In the following extract, the paragraph numbers shown are those used in the legislation. Clause 11 states:

... (3) Every designer shall in preparing or modifying a design which may be used in construction work in Great Britain avoid foreseeable risks to the health and safety of any person—
(a) carrying out construction work;
(b) liable to be affected by such construction work;
(c) cleaning any window or any transparent or translucent wall, ceiling or roof in or on a structure;
(d) maintaining the permanent fixtures and fittings of a structure; or
(e) using a structure designed as a workplace.
(4) In discharging the duty in paragraph (3), the designer shall—
(a) eliminate hazards which may give rise to risks; and
(b) reduce risks from any remaining hazards, and in so doing shall give collective measures priority over individual measures.
(5) In designing any structure for use as a workplace the designer shall take account of the provisions of the Workplace (Health, Safety and Welfare) Regulations 1992 which relate to the design of, and materials used in, the structure.

Whilst the CDM Regulations[64] indicate that ‘construction work’ means the carrying out of any building, civil engineering or engineering construction work (see Clause 2.1 of the CDM Regulations[64] for the full definition of ‘construction work’), it would seem that an engineer undertaking a structural assessment of an existing LPS dwelling block should consider the design of, and materials used in, the structure.

From the above it seems clear that the CDM Regulations[64] would be applicable to any intervention/preventive/remedial works undertaken to enhance the resistance of an existing LPS dwelling block to collapse or to extend its useful life. Similarly activities relating to improving durability and habitability of blocks would also come within the remit of the CDM Regulations[64], again depending upon whether the works were ‘construction works’. Professional activities related to the associated design and planning of the above types of works would fall within the remit of the CDM Regulations[64].

It appears that the intent of the CDM legislation is focussed upon workplace related activities associated with construction and maintenance, and that it does not concern itself with the performance of the building or other matters which fall within the remit of the Building Regulations. Common parts of buildings are treated as workplaces.

The CDM Regulations[64] and the associated Approved Code of Practice (ACOP)[55] both raise points and provide guidance on other matters which may be relevant to the current discussion. For example, the ACOP[55] recognises that the effort expended needs to be proportionate to the risk involved, with Clause 4 of the ACOP[55] stating: The effort devoted to planning and managing health and safety should be in proportion to the risks and complexity associated with the project. When deciding what you need to do to comply with these Regulations, your focus should always be on action necessary to reduce and manage risks. Any paperwork produced should help with communication and risk management. Paperwork which adds little to the management of risk is a waste of effort, and can be a dangerous distraction from the real business of risk reduction and management.

The ACOP and the primary legislation also both raise the issue of competence. In Appendix 4 of the ACOP[55], HSE has set out core criteria to demonstrate competence in the context of the requirements of the CDM Regulations[64]. These are relatively straightforward, with helpful examples of what should be included.

It seems that the CDM Regulations[64] do not impose any additional obligations and/or constraints upon those undertaking a structural assessment of an existing multi-storey LPS dwelling block. However, aspects of the CDM Regulations[64] and the associated Approved Code of Practice (ACOP)[55] both note circumstances where the appraising engineer might be called upon to provide additional evidence. In the examples considered this might be both in respect of professional competence and in the proportionality of the effort expended in relation to the risks involved.

40 At the time of writing a project is notifiable under the CDM Regulations 2007[62] if the construction phase is likely to involve more than (a) 30 days; or (b) 500 person days, of construction work.
A6 OTHER LEGISLATION

In the context of the current considerations, the key issue is whether there is other legislation which imposes additional obligations and/or constraints upon those undertaking a structural assessment or construction works upon an existing multi-storey LPS dwelling block. Two examples of legislation are considered in outline below. It is possible that there other items of legislation which would have a bearing upon the current considerations and should be borne in mind.

The Defective Premises Act 1972

- Maintenance/repair obligations exist to ensure personal safety of individuals.
- Maintenance/repair obligations exist to ensure that defects do not damage the property.
- The landlord has responsibilities for works done to a property.

The Occupiers Liability Acts 1957 and 1984

- There is a common duty of care on occupiers to ensure the safety of visitors/people who enter the premises.
- Safety in the common parts of a building remains the responsibility of the landlord.

A7 OVERVIEW OF THE TECHNICAL REQUIREMENTS FOR THE STRUCTURAL ASSESSMENT OF EXISTING LPS DWELLING BLOCKS

The current guidance for the structural assessment of LPS dwelling blocks[2,4,5], as discussed in section A2 of this Handbook, and in Approved Document A – Structure[9], as discussed in section A3 of this Handbook, recognise that the adoption of:

- the concept of ‘key elements’ able to resist the forces associated with accidental loads, in conjunction with
- a systematic risk assessment procedure for the building, taking into account all the normal hazards that may reasonably be foreseen, together with any abnormal hazards,

can provide a valid and appropriate methodology for dealing with accidental loads and actions.

In the case of an LPS dwelling block the forces associated with accidental loads are generally accepted to be those arising from an internal gas explosion (deflagration).

The preliminary review of other legislation undertaken indicates that workplace legislation could impose obligations in terms of the adequate maintenance of the structure of a building throughout its life. Unfortunately aspects of the mandatory obligations on duty holders in respect of the general population of buildings are perhaps not as clear as they might be.

However, in the case of LPS dwelling blocks the existing non-mandatory technical guidance for the structural assessment and management of existing multi-storey LPS dwelling blocks[2] is relatively clear and appears to be generally consistent with the overall objectives of the workplace legislation. Thus it appears that no significant additional obligations and/or constraints are imposed by the workplace legislation upon those undertaking a structural assessment of an LPS dwelling block for accidental loads and actions.
APPENDIX B
HAZARD ENVIRONMENT

B1 INTRODUCTION
This Appendix provides an overview of the accidental loadings and actions which form part of the hazard environment within which LPS dwelling blocks exist. This information is derived from statistical and historical information about the accidental loading and actions encountered in the general population of UK buildings.

The accidental loadings and actions upon which statistics have been collected by BRE and others are primarily concerned with various forms of gas explosion and impact by various types of vehicles including road vehicles, trains and aircraft; together with the occurrence of fires within various categories of UK buildings. Details about the frequency of occurrence of various types of incident are presented where these are available and pertinent.

This information is used to derive estimates of the probability of occurrence of an internal gas explosions in a building without a piped gas supply, together with risk levels associated with such incidents. These estimates are compared with the target ultimate limit state design life-time reliability index values and the associated life-time probability of occurrence (failure) of new buildings. To facilitate benchmarking with the risks to people which arise from some other hazards, comparison is made with a range of statistics published by the Health and Safety Executive.

These matters are addressed in the following sections:
B3 Hazard environment: Accidental external gas explosions.
B4 Hazard environment: Accidental road vehicle impact statistics.
B5 Hazard environment: Accidental impact by other types of vehicle.
B6 Hazard environment: Fire.
B7 The hazard environment – Probability of occurrence and risk levels associated with internal gas explosions in a building without a piped gas supply.
B8 Benchmarking – Comparison of risks from some other hazards

Whilst it also has to be recognised that LPS dwelling blocks could potentially be subject to malicious acts or attacks, such a matter is outside the intended scope of this document. Perhaps bizarrely such events could be inadvertent; recent experience has shown that flats within some LPS dwelling blocks have been used by those involved in the manufacture of improvised explosive devices (IEDs). Accordingly, it may be of interest that the Home Office, through the Centre for Protection of National Infrastructure and other UK security agencies, makes available guidance on how to create safer places and buildings that are less vulnerable to terrorist attack[3].

B2 ACCIDENTAL INTERNAL GAS EXPLOSION STATISTICS
The Ronan Point Inquiry[12] examined the circumstances which led to the explosion and considered the likelihood of similar events occurring. On the basis of the limited statistical data available, it was concluded that the frequency of explosions involving town gas, in premises supplied with town gas, was approximately 8 per million dwellings per year; of which some 3.5 per million dwellings per year would cause some degree of structural damage (defined at that time as more severe than window breakage).

Assuming that flats in LPS dwelling blocks were at no greater risk from explosion that any other dwelling, the above figures were used to estimate that during the envisaged 60-year life of a block of 110 dwellings (ie similar to Ronan Point) there was about a 2% probability of a gas explosion occurring in one of the dwellings that was severe enough to cause structural damage. Thus on this basis, in a population of fifty LPS dwelling blocks each containing 110 dwellings, it would be reasonable to expect on average one gas explosion a year severe enough to cause structural damage. The Inquiry concluded that the associated risk of progressive collapse after such an explosion was not acceptable, which lead to the actions described previously in Appendix A.

To improve the statistical data available upon explosions in non-industrial buildings, BRE was commissioned to undertake a survey and data were collected between 1971–1981 and between 1984–1994. Three classes of explosion are recognised in the statistics collected for reported and significant explosions in the UK building stock, as is discussed by Ellis and Currie[25].

The classes of structurally significant explosion are defined as follows:
1. Moderate Explosion – destroys relatively weak structures up to and including the typical small bungalow, or for more substantial and larger structures destroys light cladding and windows, damages stud partitioning and similar elements and blows doors off hinges.
2. **Severe Explosion** – destroys or seriously damages heavier claddings, infills and partitions (e.g. of brickwork or blockwork) and destroys or damages load bearing elements such as the gable wall and 1st floor of a typical post-1919 two storey house.

3. **Very Severe Explosion** – destroys or seriously damages stronger load bearing elements (e.g. of reinforced concrete) or causes very substantial damage over several stories of less substantial construction.

It should be noted that, according to these definitions, ‘severe’ and ‘very severe’ explosions can only occur in buildings of the size and strength specified in the definitions. An explosion which destroys a small bungalow will always be classed as a ‘moderate explosion’, ‘even though this may appear to be an incongruous description for the remaining rubble’ – to quote Ellis and Currie.[25]

Figures B1 to B7 illustrate the effects of various explosions which have been classified in accordance with the definitions given above.

Figures B1 to B5 have been reproduced from *The incidence of damage to buildings caused by explosions and vehicle impacts*.[25F] They illustrate the effects of:

- Moderate explosions involving piped gas (Figure B1) and cylinder gas (Figure B4).
- Severe explosions involving piped gas (Figures B2 and B3) and cylinder gas (Figure B5).

Figures B6 and B7 illustrate the effects of a very severe piped gas explosion which occurred in Hulme Point, an LPS dwelling block situated in Manchester. The severity of the explosion was such that three dwellings were very badly damaged/partly demolished, with windows and debris hurled up to 300 m away, which resulted in a substantial debris load accumulating at the sixth storey level. In spite of this there was no progressive collapse and the degree of damage to the structure was not considered to be disproportionate.

Summary details of the findings for the two 10-year periods of the BRE survey are presented in Tables B1 to
B4. These data concern only incidents involving gaseous explosions (deflagrations) and excludes detonations (bombs). These tables show:

- That the total number of reported explosions from all sources approximately halved between the periods 1971–1981 and 1984–1994 (Table B1).
- That whilst the number of moderate and severe explosions from all sources remained approximately constant in the two periods 1971–1981 and 1984–1994, the number of very severe explosions from all sources dropped by about 70% (Table B1).
- That whilst the number of moderate and severe explosions associated with piped gas fell by 9% and 30%, respectively, between 1971–1981 and 1984–1994, the number of very severe explosions associated with piped gas fell by about 75% (Table B2).
- That the number of moderate and severe explosions associated with cylinder gas or other gaseous substances increased by 46% and 42%, respectively, between 1971–1981 and 1984–1994 (Table B3).
- That whilst the number of moderate and severe explosions associated with cylinder gas or other gaseous substances in the period 1984–1994 was 98, that is about 10 per year in the entire UK building stock of permanent non-industrial buildings (Table B3).
- That the number of severe explosions associated with cylinder gas or other gaseous substances in the period 1984–1994 was 27, that is about three per year in the entire UK building stock of permanent non-industrial buildings (Table B3).
- That there was only one very severe explosion which was associated with other gaseous substances in the two periods 1971–1981 and 1984–1994. This was the Abbeystead water pumping valve house explosion caused by a methane accumulation\[56\]. Thus there were no very severe explosions associated with cylinder gas in the 20-year survey period between 1971–1981 and 1984–1994 in the entire UK building stock of permanent non-industrial buildings (Table B3).

Other data (this information is not separated out in Tables B1–B4) reveal that the number of severe explosions in the entire UK building stock of permanent non-industrial buildings associated with cylinder gas increased from nine in the period 1971–1981 to 15 in the period 1984–1994; that is to between one and two per year on average.

The survey data were used by Ellis and Currie\[25\] to estimate yearly probabilities of explosions in dwellings. These are presented in Table B5.

Assuming that the risk of an explosion occurring in flats within LPS dwelling blocks is no greater that of any other dwelling, the above figures have been used to estimate that the annual probability of a ‘severe’ explosion in a dwelling involving cylinder gas or other gaseous substances would be about \(0.1 \times 10^{-6}\). This value is about 20% of that quoted in Table B5 for all sources of explosions. This value is considerably lower than the explosion estimate made at the time of the Ronan Point Inquiry (circa \(3.5 \times 10^{-6}\) – see above, first paragraph of section B2).
### Table B1: Reported and significant explosions in the UK building stock: all causes

<table>
<thead>
<tr>
<th>Period</th>
<th>All</th>
<th>Other</th>
<th>Moderate</th>
<th>Severe</th>
<th>Very Severe</th>
</tr>
</thead>
<tbody>
<tr>
<td>1971–1981</td>
<td>5644</td>
<td>5212</td>
<td>282</td>
<td>133</td>
<td>17</td>
</tr>
<tr>
<td>1984–1994</td>
<td>2844</td>
<td>2437</td>
<td>295</td>
<td>107</td>
<td>5</td>
</tr>
<tr>
<td>1984–1994</td>
<td>100%</td>
<td>85.6%</td>
<td>10.4%</td>
<td>3.8%</td>
<td>0.2%</td>
</tr>
</tbody>
</table>

### Table B2: Reported and significant explosions in the UK building stock: piped gas

<table>
<thead>
<tr>
<th>Period</th>
<th>All</th>
<th>Other</th>
<th>Moderate</th>
<th>Severe</th>
<th>Very Severe</th>
</tr>
</thead>
<tbody>
<tr>
<td>1971–1981</td>
<td>2349</td>
<td>2002</td>
<td>216</td>
<td>114</td>
<td>17</td>
</tr>
<tr>
<td>1984–1994</td>
<td>1274</td>
<td>993</td>
<td>197</td>
<td>80</td>
<td>4</td>
</tr>
<tr>
<td>1984–1994</td>
<td>44.8%</td>
<td>35.0%</td>
<td>6.9%</td>
<td>2.8%</td>
<td>0.1%</td>
</tr>
</tbody>
</table>

### Table B3: Reported and significant explosions in the UK building stock: other gaseous substances, including cylinder gas

<table>
<thead>
<tr>
<th>Period</th>
<th>All</th>
<th>Other</th>
<th>Moderate</th>
<th>Severe</th>
<th>Very Severe</th>
</tr>
</thead>
<tbody>
<tr>
<td>1971–1981</td>
<td>3295</td>
<td>3209</td>
<td>67</td>
<td>19</td>
<td>0</td>
</tr>
<tr>
<td>1984–1994</td>
<td>1570</td>
<td>1444</td>
<td>98</td>
<td>27</td>
<td>1</td>
</tr>
<tr>
<td>1984–1994</td>
<td>55.2%</td>
<td>50.8%</td>
<td>3.4%</td>
<td>0.9%</td>
<td>&lt; 0.1%</td>
</tr>
</tbody>
</table>

### Table B4: Reported and significant explosions in the UK building stock: 1984–1994

<table>
<thead>
<tr>
<th>Explosive substance</th>
<th>All</th>
<th>Other</th>
<th>Moderate</th>
<th>Severe</th>
<th>Very Severe</th>
</tr>
</thead>
<tbody>
<tr>
<td>Piped gas</td>
<td>1274</td>
<td>993</td>
<td>197</td>
<td>80</td>
<td>4</td>
</tr>
<tr>
<td>Other gas and cylinder gas</td>
<td>1570</td>
<td>1444</td>
<td>98</td>
<td>27</td>
<td>1</td>
</tr>
</tbody>
</table>

### Table B5: Yearly probability of explosions in UK dwellings: 1984–1994

<table>
<thead>
<tr>
<th>Event</th>
<th>Estimated annual probability of occurrence (× 10⁻⁶)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Any explosion</td>
<td>7.9 (this may be an underestimate as not all minor incidents are recorded)</td>
</tr>
<tr>
<td>Significant explosion</td>
<td>1.5</td>
</tr>
<tr>
<td>Severe explosion</td>
<td>0.5</td>
</tr>
<tr>
<td>Very severe explosion</td>
<td>0.02</td>
</tr>
</tbody>
</table>

**Note to Tables B1–B5:** The percentage figures for the survey period 1984–1994 are based upon the total number of explosions recorded for the 1984–1994 period, namely 2844, as reported in Table B1. ‘Other’ indicates explosions that are not structurally significant. ‘Moderate’, ‘severe’ and ‘very severe’ explosions are classed as structurally significant explosions.

As noted in Tables B1–B5, no very severe explosions involving cylinder gas or other gaseous substances have been recorded in dwellings (NB. this excludes the Abbeystead incident and piped gas events).

Although explicit values of the overpressures generated during these incidents are not available, this matter was considered by Ellis and Currie[25] who made the following observations:

- **Single Room Explosions:** The rise in pressure in a room produced during combustion of a gas–air mixture could, for the stoichiometric41 gas–air mixture, reach a theoretical maximum of 700–1000 kN/m². However, in most cases the rising pressure will break the weakest link in the bounding surface of the room, typically a window or a lightweight partition, allowing venting to take place which greatly reduces the peak pressure reached in the room. Single room explosions probably accounts for over 93% of all gaseous explosions within dwellings in the UK. The maximum pressure recorded during a limited number of tests undertaken in housing units[24] where explosions were created in a single room was 13 kN/m². The resulting damage was classed as ‘moderate’.

  Whilst larger explosions might occur in bigger enclosed spaces, Ellis and Currie[25] indicate that an overpressure of 17 kN/m² is a reasonable upper bound for a single room explosion. This corresponds with the recommended overpressure to be considered during structural assessment of LPS dwelling blocks for accidental loads where a piped gas supply is not present.

---

41 Stoichiometric mixture: when there is just enough oxygen in the mixture to burn all the flammable gas.
• **Multiple Room Explosions**: Explosions in the ‘severe’ and ‘very severe’ category tend to generate damage in more than a single room. The peak pressure in a ‘very severe’ explosion has not been monitored, but it is anticipated that it could significantly exceed 34 kN/m². ‘Very severe’ explosions typically require large volumes of gas which practically can only be delivered by some form of piped gas supply.

Ellis and Currie[25] also examined the circumstances relating to ‘severe’ explosions not involving piped gas. They note that these are rare and that the majority had been caused by cylinder gas leaking into basements.

Summary of historical data on accidental internal gas explosion incidents:
- An overpressure of 17 kN/m² forms a reasonable upper bound for a single room explosion. This corresponds with the recommended overpressure to be considered during structural assessment of LPS dwelling blocks for accidental loads where a piped gas supply is not present.
- Explosions in dwellings arising from cylinder gas and other (non-piped) gaseous substances are unlikely to be more onerous than the ‘severe’ explosion category.
- The number of structurally significant explosions in dwellings arising from cylinder gas or other gaseous substances (i.e., the combined number of moderate, severe and very severe explosions in Table B3) increased by almost 50% between the 1971–1981 and 1984–1994 survey periods. However, the total number of such explosions remains small.
- Cylinder gas and other (non-piped) gaseous substances cause about three ‘severe’ explosions per year in the entire UK building stock of permanent non-industrial buildings.
- The yearly probability of a ‘severe’ explosion in a dwelling involving cylinder gas or other gaseous substances is about $0.1 \times 10^{-6}$. This frequency is about 20% of that for all sources of explosions. This value (frequency) is considerably lower than the explosion estimate made at the time of the Ronan Point Inquiry (circa $3.5 \times 10^{-6}$).

**B3 ACCIDENTAL EXTERNAL GAS EXPLOSIONS**

Whilst major accidental external gas explosions creating a substantial overpressure can cause severe or very severe damage to multi-storey buildings, they appear to be very rare. The incidents which may create this type of accidental loading include events such as vapour cloud explosions (VCEs) and boiling liquid expanding vapour explosions (BLEVEs).

Such incidents might be expected to originate from leakage of vapour or other products from chemical manufacturing or storage sites. A recent UK example of such an incident was the explosion which occurred at the Buncefield fuel depot located on the outskirts of Hemel Hempstead, Hertfordshire. This incident, which occurred on 11 December 2005, was the largest UK peacetime explosion. It caused a considerable degree of devastation in the area surrounding the depot.

These incidents typify events which have a very low-probability of occurrence, but could potentially have high or even very high consequences. Guidance on the overpressures which may be generated versus distance from the seat of the explosion, together with associated matters such as the levels at which risk mitigation measures are required, has been prepared by the Chemical Industries Association[66]. It is possible for substantial overpressures to be generated 100s of metres from the seat of the explosion.

In the context of LPS dwelling blocks they are probably sufficiently rare events not to require specific consideration. However, it should be recognised that the magnitude of the damage potentially caused by such incidents could lead to overall collapse of the zone of the affected building, or possibly the whole building in an LPS dwelling block which was potentially susceptible to progressive collapse. Such damage may not be disproportionate.

**B4 ACCIDENTAL ROAD VEHICLE IMPACT STATISTICS**

Three classes of vehicle impact damage are recognised in the statistics collected for reported and significant road vehicle impact in the UK building stock, as discussed by Ellis and Dillon[66].

The classes of structurally significant vehicle impacts are defined as follows:
1. **Moderate Impact** – is one which causes local collapse, and is confined to a single-storey and a similar width, of the typical post-1918 two-storey house, or which does comparable damage to other types of buildings.
2. **Severe Impact** – is one which causes damage as in a moderate impact, but of greater extent, or which causes local collapse of stronger, load bearing elements, for example of reinforced concrete.
3. **Very Severe Impact** – is one which causes damage more than local collapse of strong load bearing elements or which causes very extensive damage over several storeys or to several buildings of less substantial construction.

Data on road vehicle impact damage to UK buildings were collected by BRE between 1971 and 1981 and also between 1985 and 2000. The survey data revealed that road vehicle impacts most often occur with older housing, which is primarily related to the location of these buildings nearer to main roads and not to the age of the buildings. Larger UK buildings are rarely affected by significant road vehicle impacts. The survey data indicated that for the periods in question there had been no road vehicle impact which had caused severe or very severe damage to a UK building with a height of five storeys and over.

**B5 ACCIDENTAL IMPACT BY OTHER TYPES OF VEHICLE**

Accidental impacts by other types of vehicle causing severe or very severe damage to multi-storey buildings appear to be very rare. The literature does contain occasional reports of both train and aircraft impacts with
multi-storey buildings, potentially causing considerable damage and casualties, often from the fire associated with the fuel carried by the vehicles. It is reported that the chemical energy of the fuel in an aircraft may be 1000 times its kinetic energy\cite{26,27}. The terrorist attack which resulted in the death of so many people in the progressive collapse of the steel framed World Trade Center Towers in New York City on 11 September 2001 highlighted the potential implications of aircraft impact upon buildings.

Brief details of three aircraft impact incidents and one train impact incident are presented below, giving some insight into the nature of some of the accidental impact incidents involving concrete buildings.

Perhaps the most pertinent to considerations of LPS dwelling blocks was the crash on 4 October 1992 of a large cargo plane (Boeing 747–200) into an extensive apartment complex in Bijlmermeer, near Amsterdam. The death toll included four people on the plane and a further forty-three people on the ground (including occupants of the dwelling block). A further eleven people were seriously injured and another fifteen received minor injuries. The impact damage was compounded by a fire and explosions associated with burning fuel, chemicals and munitions carried on the aircraft. The damage to the building is shown in Figure B8.

An Iranian Air Force military transport aircraft (a C-130 Hercules) crashed into the top floor of a 10-storey concrete apartment building in Towhid, a densely-populated residential suburb of Tehran. The crash on 6 December 2005 killed all 94 people on board the aircraft and injured a further 132. Reports suggest an overall death toll of about 128. The building was damaged, but survived the impact and subsequent fire. The damage to the building is shown in Figure B9.

On 18 April 2002 a small aircraft crashed into the Pirelli building in Milan, Italy. The impact caused fairly widespread damage to the cladding around the point of impact, but only relatively minor damage to the load bearing structure. The crash killed the pilot and two others in the building. Sixty more people sustained injuries in the building and on the ground. The damage to the building is shown in Figure B10.

Instances of buildings being impacted by trains are rare, although such an event did occur in Japan in 2004. A train derailed near Amagasaki, 400 km west of Tokyo.

The first carriage slid into the first floor of the car park to a high-rise apartment block situated immediately adjacent the raised rail track bed. The derailed carriages after impact with the building are shown in Figure B11.

Whilst the force of the impact resulted in significant localised damage, the overall extent of damage was considered to be proportional to the impact. As the impact occurred in Japan, it is assumed that the high-rise apartment block would have been designed for earthquake loading. The seismic design requirements would undoubtedly have ensured that the Japanese apartment block had a high degree of robustness and it seems likely that it would have been more tolerant of local damage than an equivalent UK apartment block. It was reported that 106 passengers and the driver were killed. A further 549 others were injured.

In the context of aircraft impacts upon buildings; Table B5 of the HSE publication The tolerability of risk from nuclear power stations\cite{26} quotes the annual probability of an aircraft crashing into one of London’s football stadia when empty as $1 \times 10^{-6}$ (ie a chance of one in a million per year). Apparently this estimate was derived using figures from the observed frequency of aircraft crashes involving planes of all sizes in the Home Counties and the land area covered by a typical football stadium.

Looking at this in the context of LPS dwelling blocks of five storeys or more in the London area, whilst there clearly are many more LPS dwelling blocks than football stadia, if a comparison is made of the projected plan area of the LPS dwelling blocks (assuming a 5° aircraft glide angle) with that of the stadia, it is judged that the overall the area of land occupied by all the LPS dwelling blocks is of the same order as that occupied by the football stadia. On this basis, it seems reasonable to anticipate that...
the annual probability of an aircraft crashing into one of
London’s LPS dwelling blocks would be of the same order
as the value quoted above; that is $< 1 \times 10^{-6}$. This annual
probability is comparable with that obtained previously
for the occurrence of a severe explosion in a dwelling
involving cylinder or other gaseous substance
(Appendix B2).

However, incidents such as aircraft impacts into
multi-storey buildings would typically be considered to
be events with a very low-probability of occurrence,
but which could potentially have high or even very high
consequences. In the context of medium height multi-
storey buildings (say, less than 30 storeys) it might be
argued that such incidents are sufficiently rare as not
to require specific consideration. However, it would
also need to be recognised that the magnitude of the
damage potentially caused by such incidents could lead
to overall collapse of the zone of the building affected,
or possibly even the whole building. In the context of an
LPS dwelling block potentially susceptible to progressive
collapse, it is likely that there would be a significant
chance that some form or degree of progressive collapse
would occur. Of course, such damage may not be
considered to be disproportionate to the cause.

B6    FIRE

Fires are generally not classified on the basis of the
medium, severe and very severe incident categories used
for gas explosions and vehicle impacts with buildings
which have been described previously in this Handbook.
The occurrence of fires in the UK is captured as fire
statistics, via a fire and rescue service reporting system
which collects information on all fires in buildings,
vehicles and outdoor structures and any fires involving
casualties or rescues in England, Wales, Scotland and
Northern Ireland. The findings are analysed and published
every year as the Department for Communities and Local
Government Fire statistics[28].

In addition, the Fire Statistics Monitor[67] is published
four times a year, concentrating on the headline figures
only. At the time of writing, the latest full fire statistics
available for review are from 2006. These present a
more detailed analysis of fires and their causes under the
following main headings:

• Summary details for:
  * Causes of fires.
  * Deaths from fires.
  * Non-fatal casualties.
  * Country and fire and rescue service areas.
• Fires in dwellings: Casualties, source of ignition, smoke
  alarm analysis, deliberate fires, fire spreads and time of
  call to fire services.
• Fires in other buildings: Accidental and deliberate fires,
  casualties and fire detector analysis.
• Road vehicle fires.

The collection of data enables regulators and fire
services to monitor trends in the occurrences of fires and
associated casualties, which thereby helps the review and
adaptation of fire regulations. As such the occurrence
of fires in multi-storey buildings, such as LPS blocks as
opposed to other buildings, is not determined separately.
The statistics merely distinguish between fires in dwellings
and other building fires. The statistics also do not detail
the structural performance of buildings in fire incidents.
follows:

42% of accidental dwelling fire fatalities occurred for fires starting in the living or dining room, with a fatality rate of 29 deaths per 1000 fires, making it the type of fire incident most likely to result in a fatality. This compares with two deaths per 1000 fires, when the fire starts in the kitchen.

Most dwelling fires are confined to the room of origin and do not spread elsewhere in the building (88% in 2006). Of these, 48% were confined to the item first ignited.

Most accidental fires occurred between midday and 6 pm (37% in 2006). However, most casualties in accidental dwelling fires occurred between 6 pm and 12 pm (33% in 2006). Similar trends are observed for deliberate fires, with casualty rates being significantly higher during the night, with the fatality rate rising to 365 per 1000 fires.

Historically the number of injuries and deaths associated with fire incidents have gradually fallen over the years as safety measures and preventative measures have continued to improve. However, they are still significantly larger than the numbers associated with the other accidental hazards discussed in this Section.

The trends described above are similar for all industrialised nations[67], with the UK, US, France and Germany incurring direct cost fire losses of between 0.1 and 0.2% of Gross Domestic Product (GDP) between 2003 and 2005. Reported death rates[67] ranged from 1.2 per million persons in Singapore to 19 per million persons in Finland. The reported death rate for the UK was nine per million persons.

Fire is considered as a design case for all buildings, with an assumed occurrence of at least one incident in the lifetime of a building. It is considered in all design and refurbishment work. The acceptability of the design provisions for fire is related to the risk of structural failure that may result in loss of life or other adverse social or economic consequences. As such structural design for the fire state follows the general probabilistic design approach adopted for structural reliability assessments. A structure is only considered to be impacted by a fire, if the fire becomes fully developed. The reference probability for structural failure in one year has historically been taken as \( P_{rel} = 10^{-5} \). The corresponding reference probability for 50 years is \( P_{rel,50} = 5 \times 10^{-5} \).

For a 50 year working/service life, the structural Eurocodes assume a basic value of reliability index of \( \beta = 3.8 \), which corresponds to a target probability of occurrence of \( P_{rel} = 7.2 \times 10^{-4} \).

The extent of structural damage after a fire (not directly related to flashover) is assumed in various countries. Information from the UK[28] suggests that about 10% of fires in buildings are assumed to cause severe structural damage and about one third of this number leads to destruction of the structure. However, information about the structural behaviour of structures in actual fire incidents is scarce. Therefore the differentiation between design assumptions and actual failure rates is difficult to establish. A recent detailed investigation of structural performance during and post fire was undertaken in the Dalmarnock fire tests, where an LPS dwelling block is reported to have been subjected to a realistic fire scenario[68]. This work confirmed that the LPS dwelling block concerned performed well in fire, with no structural or compartmentation failure and no spalling of concrete occurring. The maximum vertical downward deflections in the floor slab were recorded as 10 mm, with inward horizontal deflections of the internal structural wall of the fire compartment being some 0.4 mm.

From the above and the observations made in previous sections of Appendix B, it will be seen that the risks to life arising from fires within LPS dwelling blocks are significantly larger than those arising from the other accidental hazards discussed earlier in this section.
B7 PROBABILITY OF OCCURRENCE AND RISK LEVELS ASSOCIATED WITH INTERNAL GAS EXPLOSIONS IN A BUILDING WITHOUT A PIPED GAS SUPPLY

It was previously shown that the frequency of severe explosions in dwellings involving cylinder gas or other gaseous substances \((0.1 \times 10^{-4})\) is about 20% of that for all sources of explosions \((\text{circa } 0.5 \times 10^{-4})\). These estimates of the frequency of occurrence are considerably lower than the estimate made at the time of the Ronan Point Inquiry, which estimated that the probability of an internal gas explosion likely to cause some degree of structural damage would be in the order of \(3.5 \times 10^{-4}\).

Using the above probability of the occurrence of a ‘severe’ explosion and assuming that:

- the largest LPS dwelling blocks might contain up to (say) 150 dwellings;
- be a maximum of 25 storeys tall;
- have an anticipated remaining life of 50 years (giving the blocks an assumed overall life of about 90 – 100 years);

the following estimates (Est 1 to Est 6) can be derived and the associated observations made:

**Est 1.** The maximum probability of a ‘severe’ explosion (deflagration) involving cylinder gas or other gaseous substances occurring during the remaining life (assuming this to be 50 years) of an LPS dwelling block would be less than:

\[
0.1 \times 10^{-4} \times 150 \times 50 = 7500 \times 10^{-4} = 7.5 \times 10^{-4}. 
\]

Thus, the probability of such an incident occurring during the remaining life of an LPS dwelling block would be less than one in 1000 (ie \(10^{-3}\)).

**Est 2.** The maximum probability of a ‘severe’ explosion (deflagration) involving cylinder gas or other gaseous substances occurring in a single stack of dwellings (maximum of 25 storeys) in an LPS dwelling block during its remaining life (taken as 50 years) would be less than:

\[
0.1 \times 10^{-4} \times 25 = 2.5 \times 10^{-4} \text{ per annum.} \]  

This referred to below as the estimate for a ‘standard’ LPS dwelling block.

On a limited number of occasions BRE has found that the compressive strength of the concrete used in an LPS dwelling block was exceptionally low. This may apply to particular localised parts of the block concerned or more rarely, because the concrete was generally of poor quality, to the LPS dwelling block overall.

A low characteristic compressive strength affects other properties of the concrete and would, for example, reduce the flexural and tensile strength of the precast concrete floor and wall panels affected. Low strength issues can also affect the in-situ joints between the precast concrete panels. In these circumstances the risk of structural failure may extend below the upper five storeys of an LPS dwelling block and therefore it is considered necessary to consider the potential effect of a ‘severe’ explosion occurring at any floor level.

For the purposes of making an estimate of the probability of occurrence, if it is assumed that (1) the maximum height of an LPS dwelling block is 25 storeys and (2) that up to 10% of blocks might contain concrete with an exceptionally low characteristic concrete compressive strength (which is believed to greatly overestimate the scale of the problem), in regard to the overall population of LPS dwelling blocks the maximum probability of a ‘severe’ explosion creating the possibility of a progressive collapse would therefore increase and become:

\[
[(0.1 \times 10^{-4} \times 25) + (0.1 \times 10^{-4} \times 5 \times 9)] / 10 = 0.1 \times 10^{-4} \times 7 = 0.7 \times 10^{-4} \text{ per annum.} \]  

This is a slight increase over the estimate given above for ‘standard’ LPS dwelling blocks.

However, the probability of a ‘severe’ explosion creating the possibility of a progressive collapse in an individual LPS dwelling block affected by the problem of exceptionally low characteristic concrete compressive strength would be:

\[
0.7 \times 10^{-4} \times 25 = 17.5 \times 10^{-4} \text{ per annum.} \]  

This is 35 times larger than the estimate given above for a ‘standard’ LPS dwelling block. However, it should be recognised that should exceptionally low concrete compressive strength be discovered in an LPS dwelling block, action would undoubtedly be taken to assess and address the problems created. So the expectation is that issues associated with exceptionally low concrete compressive strength would be resolved perhaps by strengthening works or possibly by demolition of the particular building.

**Est 4.** The structural loading tests to the ultimate load condition carried out upon three LPS dwelling blocks have indicated that the LPS structure should be capable of resisting the accidental
loads applied during severe explosion (ie the LPS block would have adequate collapse resistance). However, if conservatively it is assumed that (say) 20% of these incidents exceed the collapse resistance of the LPS dwelling block elements, so that a progressive collapse might occur; then from Est 3 an ‘upper bound’ estimate of the probability of a progressive collapse in an LPS dwelling block (assuming that risk arose within upper 5 floors) would be:

\[
0.1 \times 10^{-6} \times 5 \times 0.2 = 0.1 \times 10^{-6} \text{ per annum.}
\]

During the anticipated remaining life of (say) 50 years this would yield an ‘upper bound’ estimate of the probability of a progressive collapse in an LPS dwelling block of:

\[
0.1 \times 10^{-6} \times 50 = 5 \times 10^{-6}
\]

To get a sense of perspective, this estimate can be comparing with the notional probabilities of structural failures given in Tables E1 and E2 of Appendix E of ISO 2394: General principles of reliability for structures\(^{[33]}\).

ISO 2394\(^{[33]}\) indicates that where the consequences of failure are great, for ultimate limit state design the target life-time reliability (the target \(\beta\) – value) of a structure should be as detailed in Table B6.

In the case of existing LPS dwelling blocks, the relative costs of introducing additional safety measures into the existing structure to achieve a higher level of safety are judged by BRE to be in the ‘medium/high’ classes of Table B6. These costs relate to attempting to achieve a marginal increase in the estimated safety level of an LPS dwelling block. Thus on the basis of this criterion, the level of safety achieved in the overall population of LPS dwelling blocks would appear to achieve the levels of reliability expected for the overall population of buildings. However, the issue of potential progressive collapse remains for LPS dwelling blocks.

**Est 5.** Data used by Ellis and Currie\(^{[25]}\) indicate that the average number of inhabitants per UK dwelling was about 2.5 in 1989. The most recent data (2007) available for the UK population\(^{[69]}\) and the overall number of UK dwellings\(^{[29]}\) suggest that the average number of inhabitants per UK dwelling is now about 2.3. If it is assumed that this value is valid for LPS dwelling blocks, this enables a potential ‘upper bound’ estimate to be made for the possible number of fatalities caused by a progressive collapse in a single stack of dwellings (assumed maximum 25 storeys) in an LPS dwelling block. Thus the estimated ‘upper bound’ number of fatalities resulting from a ‘severe’ explosion involving cylinder gas or other gaseous substances in a single LPS dwelling is:

\[25 \times 2.3 = 58\] people, round this up to say 60 people

with an estimated probability of occurrence of 0.1 to 0.7 \(\times 10^{-4}\) per annum (from Est 3B and Est 4).

If the average number of inhabitants in each LPS dwelling block flat was higher than 2.3, the potential death toll might be higher. The above figures could be adjusted taking account of the actual average number of people living in the LPS dwelling block flats concerned.

In the case of an individual LPS dwelling block affected by exceptionally low concrete compressive strength, the estimated probability of occurrence could increase to 17.5 \(\times 10^{-4}\) per annum (from Est 3C). In most instances it is expected that once the issue of exceptionally low characteristic concrete compressive strength was discovered it would be addressed by some type of intervention. This might range from actions such as strengthening or possibly even demolition of the LPS block concerned.

An estimate of the notional risk of death to an individual can be obtained if it is assumed that one person is present and killed when a ‘severe’ explosion involving cylinder gas or other gaseous substances occurred in an LPS dwelling. This would give the notional risk of death to an individual in these circumstances as being in the order of 0.1 \(\times 10^{-4}\) per annum.

Comparison with the notional limit for the ‘tolerable’ limit of risk at work of 1 \(\times 10^{-5}\) per annum and with the ‘tolerable’ limit of risk to any member of the public of 1 \(\times 10^{-4}\) per annum\(^{[26,27]}\) reveals that the risk associated with a ‘severe’ explosion involving cylinder gas or other gaseous

<table>
<thead>
<tr>
<th>Column A</th>
<th>Target reliability: (\beta)–value</th>
<th>Column B</th>
<th>Column C</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>3.1</td>
<td></td>
<td>(10^{-6})</td>
</tr>
<tr>
<td>Medium</td>
<td>3.8</td>
<td></td>
<td>(7 \times 10^{-5}) [say circa (10^{-4})]</td>
</tr>
<tr>
<td>Low</td>
<td>4.3</td>
<td></td>
<td>(7 \times 10^{-6}) [say circa (10^{-5})]</td>
</tr>
</tbody>
</table>

Notes:
1. The information in columns A and B is derived from Table E2: ISO 2394\(^{[33]}\).
2. The information in column C is derived from Table E1: ISO 2394\(^{[33]}\), which gives the relationship between the target reliability index (\(\beta\)–value) and the life-time probability of occurrence (failure).
B8 BENCHMARKING – COMPARISON OF RISKS FROM SOME OTHER HAZARDS

At this juncture it may be helpful to benchmark the probability of occurrence and risk levels associated with internal gas explosions in a building without a piped gas supply, as detailed in Appendix B, section B7, with some other hazards and risks that are notionally exposed to during their everyday lives. Some risks are accepted voluntarily, such as when we chose to participate in a sporting activity or to smoke cigarettes. However, other risks are involuntary, perhaps being imposed by circumstances or as a consequence of others activities beyond the control of the affected party.

CIRIA[32] has reported on the incidence of structural failure in the UK, with the estimated/accepted risk of death due to structural collapse being 0.14 × 10⁻⁶ per annum.

Reid[70] has reported on the incidence of deaths in building fires in Australia, with the estimated/accepted risk of death being 4 × 10⁻⁶ per annum.

The Health and Safety Executive (HSE)[27] have reported statistics for the relative risk of death due to a range of other causes and the various hazards we encounter. However, comparing the degree and probability of the various risks we run is not a simple task.

Table B7 reproduces the statistics cited by HSE in Table 3 of Reducing risks, protecting people[27]. These indicate that the annual probability of death for an individual expressed as annual experience per million (10⁻⁶) ranged from about 0.05, in respect of being killed by lightning, to about 2600 for all forms of cancer.

The HSE also reported[27] statistics for the annual probability of death from industrial accidents to employees for various industry sectors. Table B9 reproduces the statistics cited by HSE in Table 3 of Reducing risks, protecting people[27]. These indicate that the annual probability of death for an individual expressed as annual experience per million (10⁻⁶) ranged from about two for the manufacture of electrical and optical equipment, to about 109 for those involved in mining and quarrying of energy producing minerals.

From the above it can be seen that the probability of being killed in a road traffic accident (60 × 10⁻⁶) is about an order of magnitude greater than those associated with death in a gas incident (6.6 × 10⁻⁶). However, we all readily accept the probability when we get into a car. This reflects the differing values that society places upon different hazards. Society seems much more ready to accept fatalities associated with road traffic accidents than it does where a similar number of casualties arise in a building collapse; perhaps because building collapses are much less frequent and more notionally more avoidable than road traffic accidents.

Comparing the annual probability of death for an individual expressed as annual experience per million (10⁻⁶) given in Tables B7–B9 with the estimated probabilities of occurrence calculated in section B7, which typically had values of less than 1 × 10⁻⁶, demonstrates that the probability of a gas explosion is relatively low, as are the number of fatalities associated with these incidents.

To place the consequences of internal gas explosions into context, it might be useful to also compare the consequences associated with them with those arising from other hazards affecting dwellings and people.

---

Table B7: Annual probability of death of an individual for various United Kingdom age groups based upon deaths in 1999 (after Table 2 of reference [27])

<table>
<thead>
<tr>
<th>Section of population</th>
<th>Annual probability</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hazard as an annual experience</td>
</tr>
<tr>
<td>Entire population</td>
<td>1 in 97</td>
</tr>
<tr>
<td>Men aged 65–74</td>
<td>1 in 36</td>
</tr>
<tr>
<td>Women aged 65–74</td>
<td>1 in 51</td>
</tr>
<tr>
<td>Men aged 35–44</td>
<td>1 in 637</td>
</tr>
<tr>
<td>Women aged 35–44</td>
<td>1 in 988</td>
</tr>
<tr>
<td>Boys aged 5–14</td>
<td>1 in 6907</td>
</tr>
<tr>
<td>Girls aged 5–14</td>
<td>1 in 8696</td>
</tr>
</tbody>
</table>

---

* Relating to fire, explosion or carbon monoxide poisoning – there are no data on the size of the associated explosion.
**Table B8**: Annual probability of death of an individual from various causes averaged over the entire population (after Table 2 of reference [27])

<table>
<thead>
<tr>
<th>Cause of death</th>
<th>Annual probability</th>
<th>Hazard as an annual experience</th>
<th>Hazard as annual experience per million ($10^{-6}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cancer</td>
<td>1 in 387</td>
<td>2600</td>
<td></td>
</tr>
<tr>
<td>Injury and poisoning</td>
<td>1 in 3137</td>
<td>320</td>
<td></td>
</tr>
<tr>
<td>All types of accidents and other external causes</td>
<td>1 in 4064</td>
<td>250</td>
<td></td>
</tr>
<tr>
<td>All forms of road accident</td>
<td>1 in 16,800</td>
<td>60</td>
<td></td>
</tr>
<tr>
<td>Lung cancer caused by radon in dwellings</td>
<td>1 in 29,000</td>
<td>34</td>
<td></td>
</tr>
<tr>
<td>Gas incident (fire, explosion or CO poisoning)</td>
<td>1 in 1,510,000</td>
<td>6.6</td>
<td></td>
</tr>
<tr>
<td>Lightning</td>
<td>1 in 18,700,000</td>
<td>0.05</td>
<td></td>
</tr>
</tbody>
</table>

**Table B9**: Annual probability of death from industrial accidents to employees for various industry sectors (based upon Health and Safety Commission figures for 2001)

<table>
<thead>
<tr>
<th>Industry sector</th>
<th>Annual probability</th>
<th>Hazard as an annual experience</th>
<th>Hazard as annual experience per million ($10^{-6}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fatalities to employees</td>
<td>1 in 125,000</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Fatalities to the self-employed</td>
<td>1 in 50,000</td>
<td>20</td>
<td></td>
</tr>
<tr>
<td>Mining and quarrying of energy producing minerals</td>
<td>1 in 9200</td>
<td>109</td>
<td></td>
</tr>
<tr>
<td>Construction</td>
<td>1 in 17,000</td>
<td>59</td>
<td></td>
</tr>
<tr>
<td>Extractive and utility supply industries</td>
<td>1 in 20,000</td>
<td>50</td>
<td></td>
</tr>
<tr>
<td>Agriculture, hunting, forestry &amp; fishing (not sea fishing)</td>
<td>1 in 17,200</td>
<td>58</td>
<td></td>
</tr>
<tr>
<td>Manufacturing of basic metals &amp; metal products</td>
<td>1 in 34,000</td>
<td>29</td>
<td></td>
</tr>
<tr>
<td>Manufacturing industry</td>
<td>1 in 77,000</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>Manufacture of electrical and optical equipment</td>
<td>1 in 500,000</td>
<td>2</td>
<td></td>
</tr>
<tr>
<td>Service industry</td>
<td>1 in 333,000</td>
<td>3</td>
<td></td>
</tr>
</tbody>
</table>

In the 1987 gale that occurred in the south east of England, 1.3 million houses were damaged to some degree. Similarly, in the gales of January and February 1990, it was estimated that 1.16 million domestic houses were damaged. In comparison, between 1984 and 1994 it has been reported that an explosion occurred in approximately 188 dwellings per annum. However, it must be noted that the magnitude and frequency of damage that occurs as a result of gas explosions (of all types) and gales can differ quite significantly. During storms the probability of damage to properties is relatively high, but often the damage is of only minor severity (i.e., often not structurally significant). Conversely, with internal gas explosions, there is a greater risk that the building structure will be damaged, but this is generally limited to one or two properties per incident.

During the same 10-year survey period, 385 deaths due to explosion incidents were recorded in non-industrial buildings. Of these, 137 following piped gas explosions, 72 were due to cylinder gas explosions and 176 resulted from other incidents. Nearly 75% of the deaths occurred in incidents in which structural damage to the affected building was minor. The 38 or so deaths per annum due to explosions should be compared to the six deaths during gales, the 707 deaths per annum due to fire in buildings (average for 1984–1994) and perhaps some 3000 deaths per annum (based on 2005 statistical survey data) on UK roads.
C1 INTRODUCTION
This Appendix provides an overview of risk issues, initially in terms of some generic concepts. Both qualitative and quantitative risk assessment approaches are considered. The concepts of risk identification, communication and risk management are discussed; and reference is made to the approaches to risk analysis adopted in the Structural Eurocodes \[11,30\].

Estimates of risk of individual fatalities are considered, together with approaches employed for defining a socially acceptable level of risk where there may be multiple fatalities. The approaches considered in detail are the use of F–N curves and the methodology employed for cost of preventing a fatality (CPF)/value of preventing a fatality (VPF) analyses.

This Appendix describes the application of some of the quantitative approaches in the context of some of the hazard circumstances applicable to LPS dwelling blocks as identified in section B7, Appendix B.

These matters are addressed in the following sections:
C2 Risk and risk management.
C3 Qualitative risk assessment.
C4 Quantitative risk assessment approaches.

C2 RISK AND RISK MANAGEMENT
Risk is a term used in the context of exposure to some form of adverse or unpleasant outcome, typically in human terms in relation to the danger of injury or death, but also potentially in relation to some form of economic or other loss. Risk is defined as the combination of the likelihood of occurrence of a particular hazard and the magnitude of the consequences thereof. In engineering situations the concept of risk is typically expressed by an equation in the following format:

\[
\text{Risk (consequence/unit time)} = \text{Frequency (event/unit time)} \times \text{Magnitude (consequence/event)}
\]

Appendix B considers the various hazards which might affect LPS dwelling blocks and provided estimates of their likely frequency/probability of occurrence. Consideration is also given to the potential consequences of the hazards. Thus these factors provide a basis for estimating the relative importance of the potential risks arising from the various types of hazards affecting LPS dwelling blocks.

Risk management is a systematic approach which is used to avoid, reduce or control risks. The course of action followed is to assess uncertainty by identifying and assessing hazards, understanding, acting on and communicating the risk issues involved. The goal of risk management is to help protect owners and users from various factors such as economic losses, injury and the possibility of death. There needs to be a balance between the cost of managing risk and the benefits expected from accepting the risk. The generally accepted components of risk management are illustrated in Figure C1.

In the processes illustrated in Figure C1 there clearly are numerous aspects to be considered including a need to:

- Establish the nature of the hazards involved.
- Define the risks to be carried and what criteria should be used in this process.
- Consider whether there are particular classes of risk, such as those imposed by statutory obligations under law or any associated with specific business/organisation objectives.
- Consider the potential outcomes.
- Clarify whether the risks change with time.
- Identify the internal and external stakeholders involved.
- Establish what hazard scenarios should be considered.
- Clarify when a risk should be controlled or reduced and how can this be done.

Once the relevant hazards have been identified and evaluated by some procedure, the decision has to be
made whether the associated risks can be accepted or not. If risks are considered to be too large for direct acceptance, the standard approach is to investigate and plan appropriate counter measures. The aim is to detect those events or processes where a significant benefit can be obtained from a proportionally small input effort.

Possible countermeasures can be technical or administrative and can fall within the following strategies:

- **Avoid** the hazard by changing the concept or the objectives (ie eliminate the hazard).
- **Reduce** the occurrence of the hazard.
- **Control** the hazards by using suitable alarm systems, vigilance, inspections, etc.
- **Minimise** the potential consequences by providing adequate strength or capacity for the worst credible loading or performance requirement.

Eliminating hazards is the prime objective and then, if this has not proved possible, to invoke the steps involved in reducing risks from the remaining hazards.

It is also relevant to note that sources of hazards can extend beyond the physical (as discussed in the previous section). The Standing Committee on Structural Safety (SCOSS) noted that hazards can also arise also from ‘soft’ issues such as the people involved (owners, designers, contractors, etc.) and the processes used. SCOSS termed these issues as the ‘3Ps’ – People, Processes and Products. Examples of such ‘soft hazards’ have been described by SCOSS in its 15th Biennial Report [71]. Carpenter and Harding [72] have also discussed these issues.

In regard to people issues it is noted that:

- **Competence**: There must be sufficient competence within the overall design or assessment team, supplemented by appropriate supervision.
- **Resource**: Sufficient resource must be provided to undertake the design or assessment activities in the timescale available.

In regard to process issues it is noted that:

- **The 3Cs**: Communication, co-operation and co-ordination are critical aspects of ensuring that structures are safe; with there being an implicit linkage between sound business and service principles and statutory requirements to achieve satisfactory through-life safety of structures.
- **Knowledge**: Ensuring use of contemporary knowledge and advice beyond design codes and available structure assessment guidance.
- **Procurement**: Is the method of procurement appropriate for the task, and does this allow for a sufficient involvement of the designer/assessment engineer in the construction/remedial works phase to minimise the risk of a diminution of structural safety which may be created by disjointed and isolated working practices?
- **QA and QC**: Are the quality assurance, control and supervision requirements compatible with the safety requirements of the structure?

There are two approaches to undertaking risk analysis and assessment, these are:

- Qualitative risk assessment.
- Quantitative risk assessment.

Figure C2 provides an overview of risk analysis, showing schematically the factors associated with both of these approaches.

**C3 QUALITATIVE RISK ASSESSMENT**

It may not be practical to make an explicit quantitative (numerical) evaluation of all the significant factors contributing to the risk environment. In these circumstances, a common approach is to use some form of risk matrix (frequency versus severity) concept. This provides a simpler basis to support aspects of the decision making needed for assessing or managing the structure concerned. The approach is essentially qualitative, with either high, medium and low ratings generally being given to the factors under consideration.

Table C3 illustrates one possible qualitative risk matrix format. It may be necessary to employ a number of risk matrices to cover the range of risk issues to be considered. Table C3 illustrates one possible qualitative approach to risk assessment and management.

The categories of severity/consequence of a hazard are defined in Table C1 to provide a qualitative measure of the worst possible incident resulting from the hazardous situation. Table C2 provides qualitative categories for the frequency and a possible categorisation/indication of the anticipated probability of occurrence of a hazard.

Using these classifications, Table C3 presents one possible template for risk profiling and categorisation, it will be noted that in this case the potential risks are divided into four categories. Other systems of classification may use more or less categories.

**C4 QUANTITATIVE RISK ASSESSMENT APPROACHES**

**C4.1 Approaches for defining a socially acceptable level of risk: F–N curves**

The social acceptance of risk to human life is commonly presented in a so called F–N curve. The F–N curve is a tool used to establish whether incidents are likely to be considered as “acceptable” or “unacceptable” in a graphical form, with the annual frequency (probability) of occurrence presented on the (logarithmic) vertical axis and the potential number of fatalities on the (logarithmic) horizontal axis. This is formally written as:

\[
P(N_d \geq n) < F(n) \quad \text{(for all } n)\]

where \(N_d\) is the number of fatalities in one year in one accident.

The probability of occurrence \(P(N_d \geq n)\) depends on the probability of failure of the system under consideration and on the factors that determine the number of fatalities in the event of a failure. The criterion is not applicable to frequent small scale accidents such as road traffic accidents (ie where \(n\) is small, \(n < 10\)).

Guidance on the use of this and related methods used for the communication of information upon risk issues is
Figure C2: Overview of risk analysis (after BS EN 1991-1-7, Figure B1)

Table C1: Severity/consequence of hazard

<table>
<thead>
<tr>
<th>Severity or consequence of hazard descriptor</th>
<th>Definition of severity or consequence of hazard</th>
</tr>
</thead>
<tbody>
<tr>
<td>I  Catastrophic</td>
<td>Multiple deaths, system loss or widespread and severe environmental damage expected</td>
</tr>
<tr>
<td>II  Critical</td>
<td>Death, severe system or environmental damage expected</td>
</tr>
<tr>
<td>III Grave</td>
<td>Severe injury, major system or environmental damage expected</td>
</tr>
<tr>
<td>IV  Serious</td>
<td>Serious injuries but recovery after few days or weeks, serious system or environmental damage likely</td>
</tr>
<tr>
<td>V   Marginal</td>
<td>May result in minor injury, system or environmental damage</td>
</tr>
<tr>
<td>VI  Negligible</td>
<td>Unlikely to result in injury, system or environmental damage</td>
</tr>
</tbody>
</table>

Notes:
1. The definition of the categories of severity/consequence of hazard need to reflect the particular task being analysed, so may need to be amended for particular situations.
2. The levels of frequency/probability of occurrence of the hazard are defined in Table C2 to provide a qualitative indication of the probability that this hazard will occur during the planned life cycle of the system under consideration.

Given in Chapter 4 of CIB Publication 259 Risk assessment and risk communication in civil engineering[27]. This document presents two sets of F–N curves representing different considerations on the boundaries between risks considered to be negligible and those considered as being unacceptable, with the region for the ALARP/SFARP management of risk residing between these two boundaries. The two figures are reproduced below as Figures C3 and C4.

Thus the range of acceptability of risk is represented by two F–N curves as indicated in Figure C3:

- The right-hand curve represents the upper limit above which activities or situations are considered to be not acceptable.
- The left-hand curve represents the lower limit below which risks ‘are generally regarded as insignificant and adequately controlled’, and that the ‘level of risk within this region are comparable to those that people regard as insignificant or trivial in their daily lives’[27]. However HSE guidance[27] does indicate that ‘duty holders must reduce risks wherever it is reasonably practical to do so or where the law requires it’.
### Table C2: Frequency or probability of occurrence of hazard

<table>
<thead>
<tr>
<th>Frequency or probability descriptor</th>
<th>Definition and indicative frequency or probability of occurrence of hazard</th>
</tr>
</thead>
<tbody>
<tr>
<td>A Very Frequent</td>
<td>Likely to occur many times in the life cycle of the building</td>
</tr>
<tr>
<td></td>
<td>(Incident frequency more than 10 per year)</td>
</tr>
<tr>
<td>B Frequent</td>
<td>Likely to occur often in the life cycle of the building</td>
</tr>
<tr>
<td></td>
<td>(Incident frequency between one per year and 10 per year)</td>
</tr>
<tr>
<td>C Common</td>
<td>Will occur a number of times in the life cycle of the building</td>
</tr>
<tr>
<td></td>
<td>(Incident frequency between one per year and one per decade)</td>
</tr>
<tr>
<td>D Likely</td>
<td>Will occur at least once in the life cycle of the building</td>
</tr>
<tr>
<td></td>
<td>(Incident frequency between one per decade and one per century)</td>
</tr>
<tr>
<td>E Occasional</td>
<td>Unlikely, but may possibly occur in the life cycle of the building</td>
</tr>
<tr>
<td></td>
<td>(Incident frequency between one per century and one per millennium)</td>
</tr>
<tr>
<td>F Possible</td>
<td>Unlikely that it will be experienced more than a few times in the life cycle of a high number of similar buildings, eg the same type of building within a community/county</td>
</tr>
<tr>
<td></td>
<td>(Incident frequency less than one per millennium: (10^{-3}))</td>
</tr>
<tr>
<td>G Rare</td>
<td>So unlikely that it might be experienced once in a high number of similar buildings, eg the same type of building within a community/county</td>
</tr>
<tr>
<td></td>
<td>(Incident frequency less than one in 10 millennia: (10^{-4}))</td>
</tr>
<tr>
<td>H Improbable</td>
<td>So unlikely that it can be assumed occurrence will not be experienced, even for a high number of similar buildings</td>
</tr>
<tr>
<td></td>
<td>(Incident frequency less than one in one hundred millennia: (10^{-5}))</td>
</tr>
<tr>
<td>I Deliberate Act (Malicious Act)</td>
<td>Hazard cannot occur unless caused by a deliberate act</td>
</tr>
<tr>
<td></td>
<td>(Incident frequency cannot be determined)</td>
</tr>
<tr>
<td></td>
<td>(NB. Malicious acts are outside the scope of this document)</td>
</tr>
</tbody>
</table>

### Table C3: Risk assessment – template for risk quantification and profiling

<table>
<thead>
<tr>
<th>Frequency/probability of occurrence</th>
<th>Severity/Consequence of hazard</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>I</td>
</tr>
<tr>
<td>C Common</td>
<td>UA</td>
</tr>
<tr>
<td>D Likely</td>
<td>UA</td>
</tr>
<tr>
<td>E Occasional</td>
<td>UA</td>
</tr>
<tr>
<td>F Possible</td>
<td>UA</td>
</tr>
<tr>
<td>G Rare</td>
<td>T</td>
</tr>
<tr>
<td>H Improbable</td>
<td>RAW</td>
</tr>
<tr>
<td>I Deliberate Act (Malicious Act)</td>
<td>RAW</td>
</tr>
</tbody>
</table>

**Key to Table C3: Risk categories**

<table>
<thead>
<tr>
<th>Code</th>
<th>Risk category</th>
<th>Action required</th>
</tr>
</thead>
<tbody>
<tr>
<td>UA</td>
<td>Unacceptable risk</td>
<td>Corrective action essential to reduce the risk</td>
</tr>
<tr>
<td>T</td>
<td>Tolerable only if risk reduction impractical</td>
<td>Review required to determine whether a further reduction of the risk is possible – risk reduction required unless cost is grossly disproportional to benefit gained from bearing risk</td>
</tr>
<tr>
<td>RAW</td>
<td>Risk acceptable with review</td>
<td>Review required to determine whether a further reduction of the risk is reasonable and practicable – implement if cost of reduction less than benefit gained from bearing risk</td>
</tr>
<tr>
<td>RA</td>
<td>Risk acceptable without review</td>
<td>No action required</td>
</tr>
</tbody>
</table>

In the area between the two curves risk-reducing measures should be considered and judged on an economic basis. This is the region for the ALARP/SFARP management of risk as noted above[^27]. Figure C4 illustrates a situation where the boundaries between the zones where the risks are considered to be negligible and those where the risks are considered to be unacceptable are defined by straight lines. As before, the region between these two boundaries is the zone in which risk is managed using the ALARP/SFARP principle. The straight lines in the F–N curve are defined by the following mathematical expression:
Figure C3: F–N curves, where \( F(n) = P(N_d > n \text{ (in one year)} \), and the ALARP region: collapse of a single stack of dwellings (the curve is dashed for small \( n \) as it is not applicable in this region). Reproduced from CIB Publication 259\(^{[73]} \) by permission of CIB.

Figure C4: F–N curves showing the \( F(n) = P(N_d > n < A n^k) \) requirement for one year: collapse of a single stack of dwellings. Reproduced from CIB Publication 259\(^{[73]} \) by permission of CIB.

Key

Potential 'higher limit' estimate of the possible number of fatalities caused by a progressive collapse in a single stack of dwellings (assumed maximum 25 storeys) in an LPS dwelling block without a piped gas supply. The fatalities are assumed to arise from an internal explosion of cylinder gas or other gaseous substance (refer Est 5).

This illustrates the potential influence of abnormally low strength concrete should this be encountered in a significant proportion of the components forming an LPS dwelling block. Abnormally low strength concrete would have a characteristic value appreciably below 20 N/mm\(^2\). It is judged that this factor could increase the probability of a progressive collapse in a single stack of dwellings by about an order of magnitude.

Points X, Y and Z are discussed in the text below Figure C4:

- X = Storebeit project
- Y = Delta works project in Holland
- Z = Channel Tunnel project
\[ F(n) = P(N_g \geq n) > A \times n^{-k} \quad (\text{for } n > 10 \text{ only}) \]

where:
- \( n \) = number of fatalities
- \( F(n) \) = frequency of incidents
- \( k \) = slope of line
- \( A \) = acceptable probability.

The parameter \( A \) is the acceptable probability for \( n = 1 \) and \( k \) determines the slope of the \( F-N \) curve in the log–log diagram. The value of \( A \) may range from 0.001 to 1 and the value of \( k \) from 1 to 2.

- The right-hand line represents the upper limit above which activities or situations are considered to be not acceptable. In this case the value \( k = 1 \), which gives a line with a slope -45°, and the value of \( A = 0.1 \).
- The left-hand curve represents the lower limit below which risks are generally regarded as insignificant and adequately controlled. In this case the value \( k = 2 \) and the value of \( A = 0.01 \).

Some examples of real-life target reliabilities are included in the Figure C4. The engineering examples given are the requirements for the Storebaelt project in Denmark (Point X), the Delta works project in Holland (Point Y) and the Storebaelt project (Point X) represents the lowest number of fatalities which have been considered to be acceptable in these three major engineering schemes. The Storebaelt project (Point X) represents the lowest number of fatalities and the lowest probability of occurrence (ie the safest of the three sets of requirements) and the design of the Channel Tunnel (Point Z) the highest number of fatalities and the highest probability of occurrence (ie the least safe of the three sets of requirements).

In both Figures C3 and C4 the potential ‘higher limit’ estimate of the possible number of fatalities caused by a progressive collapse in a single stack of dwellings in an LPS dwelling block without a piped gas supply (see Est 5) resulting from an accidental internal explosion caused by cylinder gas or other gaseous substances is located below the lowest considered annual risk of \( 1 \times 10^{-4} \). This indicates that the estimated risk of fatalities in LPS dwelling blocks due to this cause is sufficiently low to be ‘off the scale’ in the risk levels typically considered. Thus for an accidental internal explosion caused by cylinder gas or other gaseous substances the estimated risks in the area or close to the area in which risks are generally regarded as insignificant and adequately controlled.

By way of comparison, if considerations of the risk of potential accidental fatalities in LPS dwelling blocks was extended to potential aircraft impacts in the London area as discussed in section B5, Appendix B, such an impact might conceivably demolish an entire LPS dwelling block. If an estimate of the potential maximum number of fatalities is based upon an LPS dwelling block comprising 150 dwellings (say 25 storeys with six dwellings per floor) with an average occupancy rate of 2.3 people per dwelling\(^{29}\), on this basis it is conceivable that there might be 350 fatalities. In general most LPS dwelling blocks are considerably smaller, with occupancies of perhaps 100–150 people when everyone is ‘at home’.

It is estimated in section 13, Appendix B that the annual probability for an aircraft impact upon a block is \( 1 \times 10^{-4} \) or less. However the likelihood that all the flats are ‘fully’ occupied at any moment in time is probably relatively low. This factor would further reduce the consequences of occurrence of the incident, perhaps by an order of magnitude.

The potential ‘higher limit’ estimate of the number of fatalities arising from such an incident is shown in the \( F-N \) curve format in Figure C5. As with the risk of an accidental internal explosion caused by cylinder gas or other gaseous substances, the estimated risk associated with aircraft impact is located at or below the lowest considered annual risk of \( 1 \times 10^{-4} \). Again this suggests that the risks associated with the impact of an aircraft on an LPS dwelling block might be regarded as insignificant and adequately controlled.

### C4.2 Approaches for defining a socially acceptable level of risk: Cost benefit analysis

Evans has discussed safety appraisal criteria\(^{31}\), in particular the use of cost benefit analysis (CBA) techniques in conjunction with the issues of the tolerability of risk (TOR) and the value of preventing a fatality (VFP). He notes that in terms of cost benefit analysis the analysis should take into account, as far as possible, all costs and benefits to whoever they accrue; with the general objective that (subject to budgetary constraints) actions for which the total benefits exceed the total costs arefavoured and those for which the costs exceed the benefits are not.

Evans\(^{31}\) examines the various arguments for and against the use of the CBA approach. However, overall in spite of the limitations the CBA approach is considered to provide a methodology which is understood and agreed and gives results which are sufficiently consistent in different applications to be useful. Evans\(^{16}\) develops these ideas and explores the use of a TOR/ALARP/SFARP/CBA framework for decision making on safety related actions, noting that the official 2003 value of preventing a fatality (VFP) was £1.31 million (Department of Transport, 2004\(^{46}\)).

Figure C6, reproduced from Evans paper\(^{31}\), illustrates individual risk and the cost of preventing a fatality (CPF) for various transport related safety measures.

Observations on Figure C6:

1. **BR–ATP for rail commuters** relates to British Rail’s automatic train protection technology, which, in 1988, British Rail committed itself to implement. The effective CPF for the ATP system was about £15 million at 2003 prices. The probability of death of an individual was estimated to be about \( 1.3 \times 10^{-5} \) per year. This is clearly well into the ‘do not implement’ region. After much discussion, Ministers decide against its adoption in 1995. Thus the safety measure was not implemented.

2. **The inhibit development on 10^{-5} probability line** relates to risk of fatalities contours in airport safety zones (ie a probability of death of \( 10^{-5} \) per year). The point shown indicates where it might be appropriate to
Figure C5: F–N curves showing the $P(N_d > n) < A n^{-k}$ requirement for one year: collapse of an entire LPS dwelling block. Reproduced from CIB Publication 259.[73] by permission of CIB.

Key:
- Points X, Y, and Z are discussed in the text below Figure C4:
  - X = Storebælt project
  - Y = Delta works project in Holland
  - Z = Channel Tunnel project

Figure C6: Individual risk and the cost of preventing a fatality for various transport safety measures. After Figure 9 from Evans’ paper Safety appraisal criteria.[31]. Reproduced by permission of the author and The Royal Academy of Engineering.

Key:
- Individual risk of death (circa $0.1 \times 10^{-9}$) due to progressive collapse in an LPS dwelling block resulting from an internal explosion of cylinder gas or other gaseous substance (refer Esti 6) and the estimated cost of preventing a fatality (perhaps £100,000 to £1 million or possibly more per fatality saved).
inhibit the construction of new housing in the vicinity of an airport due to safety issues. The effective CPF in this situation was about £0.1 million at 2003 prices and the value of implementing the safety measure is marginal.

Example 3. Local road safety for TWMV users relates to a hypothetical person who rides a two-wheel motor vehicle (TWMV) for 500 hours per year (eg about 2.5 hours per workday). Clearly the probability of death of an individual riding a TWMV is very high, being about $2.4 \times 10^{-3}$ per year. The effective CPF for TWMV users was about £0.1 million at 2003 prices and the value of implementing the safety measure is clear.

Example 4. The remove houses on $10^{-4}$ probability line relates to risk of fatality contours in airport safety zones (ie a probability of death of an individual of $10^{-4}$ per year). The point shown indicates where it is necessary to remove existing houses in the vicinity of an airport due to the excessive level of risk to the individuals involved. The effective CPF in this situation was about £42 million at 2003 prices. The safety measure was implemented.

Example 5. TPWS for rail commuters relates to the train protection and warning system that was developed providing a simpler and lower-cost form of train protection than the BR–ATP. The decision to implement TPWS was made again even though it is clearly well into the ‘do not implement’ region. TPWS was installed across the railway network in 2003. The probability of death of an individual was estimated to be about $7 \times 10^{-4}$ per year. The effective CPF for the TPWS system was estimated to be about £11 million at 2003 prices. The safety measure was implemented.

Example 6. The inhibit development on $10^{-4}$ probability line relates to risk of fatality contours in airport safety zones (ie a probability of death of an individual of $10^{-4}$ per year). The point shown indicates where it is economically appropriate to inhibit the construction of new housing in the vicinity of an airport due to safety issues. The effective CPF in this situation was about £0.65 million at 2003 prices. The safety measure was implemented.

Example 7. The speed cameras entry concerns the introduction of speed cameras and relates to the individual risk of all road users who have a probability of death of about $6.1 \times 10^{-5}$ per year. The effective CPF for all road users was about £0.4 million at 2003 prices. The policy of introducing speed cameras as a safety measure has been implemented.

Example 8. Local road safety for all road users concerns the risk to individual road users. Overall all road users each have an overall probability of death of about $6.1 \times 10^{-3}$ per year. The effective CPF for all road users was about £0.1 million at 2003 prices.

Thus, in Examples 1–3, the safety measure was either not implemented or was considered to be of marginal overall benefit. In these instances the CPF ranged from about £0.1 million to £1.5 million at 2003 prices, with the individual risk of death ranging from about $10^{-5}$ to $2.4 \times 10^{-3}$ per year.

In Examples 4–8 the safety measure was implemented. In these instances the CPF ranged from about £0.1 million to £42 million at 2003 prices, with the individual risk of death ranging from about $7 \times 10^{-5}$ to $10^{-1}$ per year.

Recognising that Cost Benefit Analysis is an analytical tool that informs rather than makes the decision, it is perhaps not surprising that some safety related decisions are inconsistent with TOR/CBA appraisal results. Each decision is clearly made within its own specific context which includes a range of social and related aspects.

### C4.3 Cost of preventing a fatality (CPF) in an LPS dwelling block

In the context of the cost of preventing a fatality in an LPS dwelling block due to failure of a loadbearing element under accidental loading arising from a non-piped gas explosion, the estimated cost of preventing a fatality (CPF) has not been evaluated and would therefore require a study to assess its ‘value’ with any degree of certainty. However, a preliminary estimate has been made for some notional situations to seek to establish an indication of the potential range in the value of the CPF in these circumstances. Table C4 presents a simplified estimate of the notional cost of preventing a fatality in an LPS dwelling block considering two approaches; these being the adoption of a remedial works strategy (Approach 1) and the reconstruction of the block using a contemporary form of construction which complies with the requirements of Approved Document A – Structure and, therefore, should not be sensitive to progressive collapse or disproportionate damage (Approach 2).

The analysis undertaken shows that the CPF values obtained varied considerably depending upon the assumptions being made (see Table C4). For example, in the case of Approach 1, undertaking remedial works to dwellings in an LPS dwelling block, CPF values might potentially range from about £2000 to about £100,000 depending on the assumptions made. The CPF values towards the bottom of the quoted range are probably unrealistically low, arising because of the simplicity of the preliminary analysis undertaken. More realistic values are expected to be at the upper end of the range quoted.

In the case of Approach 2, involving the replacement of the block with a new building complying with contemporary Building Regulation requirements, CPF values might potentially range from about £90,000 to about £5 million depending on the assumptions made. Again the CPF values towards the bottom of the quoted range are probably unrealistically low, arising because of the simplicity of the preliminary analysis undertaken. More realistic values are expected to be in the middle of the range quoted (perhaps in the order of £1 million or perhaps somewhat more).

On the basis of the simplified preliminary estimates made, the CPF is perhaps in the range £100,000 to £1 million or more. The action associated with the higher estimated sum would involve demolishing the existing LPS dwelling block and rebuilding it as a more robust contemporary building. The action associated with the lower sum is indicative of the possible cost of a remedial works approach involving strengthening of the affected LPS dwelling block. The annual probability of an individual fatality is below the threshold at which action
**Table C4:** Simplified estimate of the notional cost of preventing a fatality in an LPS dwelling block considering a remedial works strategy (Approach 1) and the replacement of the block (Approach 2) with a new building complying with contemporary Building Regulation requirements (see Note at foot of Table).

<table>
<thead>
<tr>
<th>Approach</th>
<th>Description</th>
<th>Notional cost per dwelling</th>
</tr>
</thead>
<tbody>
<tr>
<td>Approach 1</td>
<td>Assumed cost of remedial works to LPS dwelling block</td>
<td>£20,000 per dwelling</td>
</tr>
<tr>
<td>Approach 2</td>
<td>Assumed cost of reconstruction of LPS dwelling block</td>
<td>£200,000 per dwelling</td>
</tr>
</tbody>
</table>

Assume Approach 1 remedial works to LPS dwelling block can be limited to upper 5 storeys

| Approach 1 | Notional cost of remedial works to a single stack of dwellings in an LPS dwelling block (Max. five storeys – see Note) | £100,000 per dwelling |
| Approach 2 | Notional cost of reconstruction of a single stack of dwellings in an LPS dwelling block (total No. of storeys) | £1,000,000 per dwelling |

Average number of people living in an LPS dwelling: 2.3 (Assumption)

<table>
<thead>
<tr>
<th>Total number of storeys within LPS block</th>
<th>5</th>
<th>25</th>
</tr>
</thead>
<tbody>
<tr>
<td>Max. number of people living in a single stack of dwellings in an LPS dwelling block (based on average of 2.3 people per LPS dwelling)</td>
<td>12</td>
<td>60</td>
</tr>
</tbody>
</table>

Notional cost of preventing a fatality in an LPS dwelling block

<table>
<thead>
<tr>
<th>Approach 1</th>
<th>For remedial works to a single stack of dwellings in an LPS dwelling block</th>
<th>Total number of storeys within LPS block</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Potential number of fatalities</strong></td>
<td>5</td>
<td>25</td>
</tr>
<tr>
<td>Minimum (assuming 1 person in stack at time of failure)</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Notional (assuming 1 person per dwelling)</td>
<td>5</td>
<td>25</td>
</tr>
<tr>
<td>Maximum (assuming 2.3 persons per dwelling)</td>
<td>12</td>
<td>60</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Approach 2</th>
<th>For reconstruction of a single stack of dwellings in an LPS dwelling block</th>
<th>Total number of storeys within LPS block</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Potential number of fatalities</strong></td>
<td>5</td>
<td>25</td>
</tr>
<tr>
<td>Minimum (assuming 1 person in stack at time of failure)</td>
<td>1</td>
<td>1</td>
</tr>
<tr>
<td>Notional (assuming 1 person per dwelling)</td>
<td>5</td>
<td>25</td>
</tr>
</tbody>
</table>

*It has been assumed that the Approach 1 remedial works to an LPS block can be limited to the upper five storeys. Effectively this consideration could apply only to a building without a piped gas supply. This implies that the failure of a loadbearing element under accidental loading would be due to the forces generated during a non-piped gas explosion.

Under both Approach 1 and Approach 2 the marginal increase in safety achieved for LPS dwelling block residents would be small, because the existing levels of risk of death are very small.

### C4.4 Judging the proportionality of measures to reduce risk: Illustration of the application of the ‘value of preventing a fatality’ (VPF)

The conceptual use of cost benefit analysis (CBA) as a way to review whether risks are As Low As Reasonably Practicable (ALARP) is discussed in Appendix 2 of the third edition of the Institution of Structural Engineers’ document Appraisal of existing structures[11]. The following extract from that document presents a simplified illustration of the application of the technique.
Appendix C: Risk Issues

Cost benefit analysis (CBA) is one way to review whether risks are ALARP, and provides a useful model for what is acceptable. However, the Health and Safety Executive (HSE) do not expect that CBA will be used explicitly in most cases. CBA is a useful model to develop understanding, but ‘good practice’ should be considered first. CBA attempts to place the risk and the cost of reducing it on a common scale, often but not necessarily monetary.

To illustrate this, assume that a defect or shortfall in the structure was judged to have a 1 in 1000 risk of causing death. Taking a benchmark value of about £1,000,000 (2001 prices) for the value of preventing a fatality (VPF) as proposed by the HSE, the benefit of removing the risk would be valued at £1000. The employer would be legally bound to remove the risk unless the cost of doing so would be grossly disproportionate to the benefit, i.e., the cost was significantly more than £1000. If the cost was estimated to be, for example, £1500, this would not be considered grossly disproportionate, and the risk should be removed.

The factor for ‘gross disproportion’ is usually taken as at least 2, and can be up to 10 for large risks, i.e., the cost would need to be more than £2000–£10,000 to justify taking no action. If it is not reasonably practicable to remove the whole risk, but only to reduce it, the same test should be applied to the reduction in risk. In many cases, a qualitative argument will be sufficient to show where the balance lies. It is sometimes argued that if a numerical assessment is necessary, the case is sufficiently ‘borderline’ that action is required. It might be unreasonable to expect the appraising engineer to carry out a complete ALARP assessment as part of the appraisal, but any circumstances where ALARP may apply should be brought to the client’s notice.

### C4.5 Judging the proportionality of measures to reduce risk: Application of ‘value of preventing a fatality’ (VPF) to an LPS dwelling block

The potential ‘upper bound’ estimate of the possible number of fatalities caused by a progressive collapse in a single stack of dwellings (assumed maximum 25 storeys) in an LPS dwelling block is 60 people (refer Est 5, Appendix B). The estimate assumes the collapse results from a ‘severe’ explosion involving cylinder gas or other gaseous substances in a single LPS dwelling, this gives an estimated probability of occurrence of $0.7 \times 10^{-6}$ per annum (from Est 3A, Est 3B & Est 4, Appendix B).

This estimate rises to $17.5 \times 10^{-6}$ per annum (from Est 3C, Appendix B) for the situation where an exceptionally low concrete strength is encountered throughout an LPS dwelling block.

The sum taken as the ‘value of preventing a fatality’ (VPF) varies according to circumstances and the organisations involved, is discussed by Evans[31]. However, for the purposes of this illustration VPF is taken to be £1.5 million. So considering the notional ‘worst case scenario’ identified above:

- Using this value of VPF for the estimated 60 fatalities gives £1.5 million $\times 60 = £90$ million.
- The probability of occurrence is generally estimated to be 0.1 to $0.7 \times 10^{-6}$ per annum.
- Taking the highest likelihood of occurrence ($0.7 \times 10^{-6}$ per annum), and on the above basis of the previously outlined VPF philosophy, the sum that might rationally be spent preventing these possible fatalities = £90 million $\times 0.7 \times 10^{-6} = £63$.
- Even allowing 10 times this sum as a notional upper boundary of a proportionate sum which might be spent, the maximum sum which might reasonably be spent preventing a fatality would be $= £63 \times 10 = £630$. Clearly this is very low and it seems highly unlikely that any significant practical measures could be undertaken for such a small amount of expenditure.
- In the situation where an exceptionally low concrete strength is encountered throughout an LPS dwelling block, the probability of occurrence is estimated to be $17.5 \times 10^{-6}$ per annum. On this basis, the sum that could be spent preventing a fatality = £90 million $\times 17.5 \times 10^{-6} = £1575$. Again even allowing 10 times this sum as a notional upper boundary of a proportionate sum which might be spent preventing a fatality, this gives a maximum sum of just $£1575 \times 10 = £16,000$. Clearly this is still a relatively low sum of money for such works.

Thus for an accidental internal explosion caused by cylinder gas or other gaseous substances or an aircraft impact upon an LPS dwelling block, the above estimates developed using the VPF philosophy indicate that the estimated risks would be ‘generally regarded as insignificant and adequately controlled’.
APPENDIX D
OUTLINE HISTORICAL REVIEW OF LPS DWELLING BLOCKS IN THE UK SINCE 1968

D1 INTRODUCTION
This Appendix provides an outline historical review of LPS dwelling blocks in the UK since 1968 dealing briefly with matters such as indicative numbers of the different types of LPS dwelling blocks constructed. An indication is given of the number of dwellings constructed in the UK utilising LPS blocks of (a) less than and (b) more than four storeys in height (ie five storeys and higher).

There are also brief descriptions of some of the various forms of structural testing which were undertaken on some large panel systems shortly after the Ronan Point partial collapse in 1968. These descriptions only give a ‘flavour’ of the work undertaken and effort expended at that time seeking to demonstrate that LPS dwelling blocks were fit for service or what works were required to ensure this.

These items are reported in the following sections:
D2 LPS dwelling block type.
D3 Structural assessment of LPS dwelling blocks for accidental loading.
D4 Historic load testing of LPS structures/components (Concrete Ltd).
D5 Strength testing of cross- and flank-wall joints in Reema blocks.
D6 Strength testing of other LPS dwelling block systems.

D2 LPS DWELLING BLOCK TYPES
Records of construction indicate that approximately 160,000 dwellings were constructed in the UK using large panel systems. A BRE report published in 1986 presents the results of a survey of local authorities, new town development corporations and other bodies within the UK conducted in 1984–1985. However, as 65 English local housing authorities failed to respond to a request for information, the data presented in the report only relate to approximately 70% of the above total.

The total number of LPS dwellings listed above relates to tenanted residential buildings owned at that time by local authorities and other similar bodies. However, a number of LPS systems were designed for other uses. These include army barrack accommodation, student accommodation, small hotels, local community centres, private houses and flats and low-rise offices. BRE currently have no information on the location or total number of these classes of LPS dwelling blocks.

A BRE handbook presents information on a number of LPS low-rise dwellings that were specifically designed for use as privately owned houses and therefore were not included in the BRE report.

The non-traditional LPS dwellings included in the handbook are: Alcrete, AMcK, Bates 4L, BDG, Blackborrow, BRS L-shaped panels, Carlton, Concept4, Fairweather, Falcon, Gregory Industrialised, Kincorth Mk III*, Louden Mk II*, Modus, Natcon*, Reema (Conclad*, Contrad*), SB2, Steward & Partners Type II, Stubbings Industrialised, Token, Wil-Mac. The first floor in the buildings types marked with an asterisk is of traditional timber construction.

The handbook also contains references to other sources of literature (ie Comprehensive Industrialised Building System Annual, Interbuild System Building) which present fairly limited technical information on the classes of system built dwellings designed primarily for the private sector.

Figure D1 shows the variety of systems and the relative numbers of dwellings in blocks of four storeys or less constructed for the public sector.

In light of the above reported limitations on the original data set, Figure D1 should be taken as simply being indicative of the overall numbers and the relative proportion of dwellings constructed using the different types of large panel systems then available on the market for UK publicsector housing.

The relative numbers of dwellings in blocks of five storeys or more constructed in a variety of systems for the public sector is presented in Figure D2. Again this should be taken as being indicative.

D3 STRUCTURAL ASSESSMENT OF LPS DWELLING BLOCKS FOR ACCIDENTAL LOADING
It is estimated that there are still over 700 high rise LPS dwelling blocks (≈ 50,000 dwellings) in the UK; block owners have an ongoing responsibility to manage them. This activity requires periodic inspection and structural assessment of the blocks concerned. In addition, there are also in excess of 1000 low- and medium-rise LPS dwelling blocks in the UK.

The structural assessment requirements stem from the partial collapse of Ronan Point. The incident resulted in changes in the UK Building Regulations, the introduction

---

44 Since the survey data was published an unknown number of low, medium and high-rise LPS blocks have been demolished. Accordingly, the number of 160,000 dwellings should be taken as an indicative total only.
Figure D1: Approximate number of blocks by system type: four storeys or less

Figure D2: Approximate number of blocks by system type: five storeys or more

of the concept of ‘robustness’, as well as an ongoing need for the structural assessment and management of this particular class of UK buildings.

Traditionally structural assessments under accidental loading have been based on the results of a combination of ‘simplified’ calculations, which by their very nature produce conservative results, and engineering judgment. BRE became aware that these issues had unfortunately resulted in a number of inappropriate recommendations for strengthening and, in extreme cases, even demolition and replacement of LPS dwelling blocks by other dwellings. This situation appeared to be creating unnecessary expenditure, social upheaval, environmental impacts, etc.

BRE had believed for some years that LPS dwelling blocks should be stronger than the ‘simplified’ structural assessment calculations could demonstrate, particularly in relation to the forces likely to be associated with certain types of gas explosions. However, in the early 1990s when there was a requirement to undertake structural assessments of many existing LPS dwelling blocks (which were required to remain in-service), there was not a suitable experimentally verified analytical method available to demonstrate that the blocks were likely to be...
able to resist the accidental loading that they might be subjected to by a non-piped gas explosion.

BRE has since gained more experience and knowledge about the behaviour of some types of LPS dwelling blocks under the high levels of loading which might be associated with a non-piped gas explosion. This was obtained from programmes of full-scale testing to failure of selected elements bordering the area of a block under test were undertaken within three LPS dwelling blocks which were nine, 15 and 22 storeys high. These tests enabled certain facets of structural behaviour to be explored and, in all three cases, to demonstrate that adequate reserves of strength existing in these LPS dwelling blocks for the loading situations that they were likely to be exposed to. This evaluation took account of the fact that the quality of the execution of the construction (workmanship) was not perfect and that errors had been made when erecting the buildings (eg some reinforcing bars was not correctly located in joints, erection was not fully compliant with rules for the lapping of bars or for their anchorage, etc.).

Engineers have been unable to take into consideration certain facets of structural behaviour, which if incorporated into the assessment process, could in many instances demonstrate adequate reserves of strength. The recommendations given in previous assessment guidance documents did not consider these forms of behaviour.

D4 HISTORICAL LOAD TESTING OF LPS STRUCTURES/COMPONENTS (CONCRETE LTD)

Following the partial collapse of Ronan Point, Concrete Ltd undertook a programme of 137 load tests upon selected elements of their Bison Wallframe large panel system. The tests were witnessed by independent engineers and representatives from local authorities in an attempt to allay the common concern that appeared to exist at that time which was that all LPS dwelling block systems in the UK might be at a similar risk of collapse. The results of these tests are published in a 1968 report by Concrete Ltd.

The test programme was designed to evaluate the strength and behaviour of wall, floor and ceiling panels under outward, downward and upward loading, respectively; together with the strength of the steel connections under tensile loading.

The tests were undertaken on individual panels and joint mock-ups. Whilst each set of tests was designed to explore one particular form of structural behaviour, it was hoped that by ‘adding’ the results together it would be possible to build-up a picture of how a complete block might behave. It is not clear, however, whether this aspect was evaluated and therefore how closely this aspiration was met.

Whilst Concrete Ltd’s final report provided general details of the type of tests that were undertaken together with a brief summary of the associated results, there appear to be a number of important details missing. These omissions raised a number of questions over the actual loading conditions, restraints, etc., that were achieved. For example, a précis of the results of tests undertaken upon actual buildings in Scotland suggests that the base of the wall was able to accommodate an equivalent lateral overpressure of 34 kN/m². However, Concrete Ltd’s final report did not state where up the height of the block the test was undertaken or whether any uplift was applied to the ceiling above the test room. Since it has not been possible to gain access to the original test data we have not been able to resolve this and other outstanding issues.

The authors of the report on the 137 load tests concluded that ‘in no case were the basic design principles found to be at fault and that had an explosion of the magnitude that occurred in Ronan Point taken place in a Bison Wallframe block, no collapse would have occurred’. It was also concluded that ‘in many cases the tests demonstrated that there were unaccounted reserves of strength within the structure (joint mock-ups)’.

D5 STRENGTH TESTING OF CROSS- AND FLANK-WALL JOINTS IN REEMA BLOCKS

D5.1 Cross wall/floor slab joints

Full-scale tensile tests were undertaken by Reema Construction Ltd on three floor/cross wall panel joint specimens in a laboratory under varying vertical loads. The tests undertaken in September 1969 were witnessed by representatives of The Institution of Structural Engineers. It is suspected that specimens tested were probably constructed and tested carefully, so the results obtained might have been ‘better than the average’ achieved on site.

The test results were regarded as being very satisfactory as the three joint mock-ups exhibited peak and residual strengths in excess of the tensile load that would be imposed upon the joint by the notional 34 kN/m² (5 lb/in²) equivalent static pressure. The tests indicated that only very small movements were required for the tensile capacity of the joint to be mobilised.

Importantly, due to the configuration of the test set-up, the engineers were not able to explore the influence of bending of the floor and wall panels or that of reverse loading on the floor soffit. Therefore whilst these tests were very useful in exploring certain aspects of the behaviour of one type of joint, it must be highlighted that they only provided a partial understanding of the potential behaviour of the Reema LPS system under the stated loading conditions.

D5.2 Flank wall/floor slab joints

Calculations undertaken by the then E W H Gifford and Partners to assess the behaviour of flank wall/floor slab joints in three ‘scissor’ type Reema blocks showed that the potential strength of these joints could not be assessed accurately with an adequate level of confidence.

Therefore a series of tests were subsequently undertaken to determine the likely ultimate strength of this type of joint. BRE believe that these tests are likely to be

---

The programme of load tests undertaken by BRE upon pre-Ronan Point Reema Conclad and Bison Wallframe blocks, in Sandwell, Leeds and Liverpool have confirmed that this assertion is reasonable.
have been carried out in the late 1960s or possibly the early 1970s.

According to the test report the type and configuration of tests were determined after a ‘careful study of the likely behaviour of the adjoining structural elements and a physical examination of the as-built joint detail in one of the existing structures’. It seems that discussions were also held at that time with a representative of the National Building Agency, although there is no indication of the content or outcome of these discussions.

The report emphasised that every attempt was made to construct a joint ‘mock-up’ of not more than average quality and that results for only two joint specimens were obtained. It was also noted that it would be necessary to test a further five joint specimens before it was possible to determine the probability of the strength of any one joint falling below that indicated in MHLG Circular 62/68\[4\].

The report concluded that the flank wall/floor slab joint specimens tested were able to sustain a load twice that of the 2.5 lb/in\(^2\) overpressure criterion (17 kN/m\(^2\)) specified in MHLG Circular 62/68\[4\]. The report stated that compliance with MHLG Circular 62/68\[4\] had been achieved.

D6 STRENGTH TESTING OF OTHER LPS DWELLING BLOCK SYSTEMS

Reports exist describing strength testing carried out in relation to other LPS dwelling block systems, such as the laboratory testing of a full-scale structural model of the Wates LPS dwelling block system comprising three single room tests cells used for static overpressure tests to failure. The study examined various potential influences upon the behaviour and strength of wall and floor panels and the associated joints, including the effect that vertical load in the walls had upon their resistance to lateral loading. Although this and other work is not reported here, it provided insight into the potential behaviour of LPS dwelling block system components and assemblies under accidental loading.
APPENDIX E

BRE TESTS ON LPS DWELLING BLOCKS AND LABORATORY STRUCTURES UNDERTAKEN BEFORE 2000

E1 INTRODUCTION
This Appendix describes various complementary structural and explosion tests undertaken upon laboratory structures and in existing LPS dwelling blocks by BRE between the 1980s and about 2000. These are reported in the following sections:
Section E2. Testing of reinforced concrete panels under static and dynamic loads.
Section E3. Explosion testing in existing LPS dwelling blocks.
Section E4. Full-scale load testing to failure of existing LPS dwelling blocks.

It is interesting to note that earlier programmes of laboratory structural and explosion testing work upon large-scale structural models of LPS dwelling blocks were also undertaken by BRE in the early 1970s. However, whilst detailed results of this work are not available to the current authors, aspects of the outcome of these studies have been passed on by BRE colleagues who were directly involved. This earlier work has informed the tests reported in this Appendix and also the development and implementation of the subsequent studies reported in Appendices F–H.

E2 TESTING OF REINFORCED CONCRETE PANELS UNDER STATIC AND DYNAMIC LOADS
BRE undertook a programme of static and transient load testing at the former BRE Cardington test facility in Bedfordshire. Amongst other goals the aim of these tests was to investigate the accuracy of procedures for calculating the structural response of reinforced concrete panels to gas explosions.

The testing indicated that there were significant problems in predicting actual behaviour of simple uniform reinforced concrete panels even when using a range of analytical techniques. The authors concluded that there is likely to be similar or greater problems when seeking to analyse complex structures, ie LPS dwelling blocks.

Of particular relevance is the different behaviours exhibited by the reinforced concrete test panels under static loading compared with that shown under transient loading (ie gas explosion).

Whilst the researchers appeared to have had leakage problems with the test rig during the static load tests, which limited that ultimate pressure that could be applied, the static strength was estimated to be approximately half that sustained by notionally similar panels under the transient loading associated with gaseous explosions.

Of particular relevance were strain rate effects, demonstrated by the different behaviours exhibited by the reinforced concrete test panels under static loading compared with that exhibited under transient loading (ie gas explosion). The concrete test panels appeared appreciably stiffer (the researchers noted that the displacement at a given applied static load was significantly more than that measured for the same applied load during the explosion test) and stronger under dynamic loading compared with their behaviour under static loading.

Accordingly, the wall and floor panels in a LPS dwelling block are expected to deflect appreciably less and be significantly stronger when subject to loading from a gaseous explosion then when subjected to an equivalent static loading.

E3 EXPLOSION TESTING IN EXISTING LPS DWELLING BLOCKS
Explosion testing was undertaken by BRE in several rooms in a three-storey block of maisonettes constructed using a pre-Ronan Point Reema LPS system. The blocks were located in Chapeltown, Sheffield. These tests were conducted to verify the results obtained previously in a test laboratory. Each test explosion was generated using a ruptured aerosol canister containing butane and an ignition source. The resulting overpressures and boundary wall displacement behaviour were recorded at various positions within each of the test rooms.

The tests seemed to indicate that venting and failure of non-structural walls determined the maximum overpressures reached within each test room. The magnitude of the overpressures generated using 200 ml and 750 ml canisters varied even for the same size canister and identical test set-ups. The pressures generated ranged from 2.6 kN/m² to a maximum of 9.0 kN/m². The maximum lateral deflection of a wall panel bordering a test room was 0.074 mm (associated with an overpressure of 9.0 kN/m²). Importantly, the tests indicated that different areas of the test room experienced slightly different maximum overpressures and that the overpressures recorded at each measurement position appeared to vary with each test explosion.

No structural damage was reported, although damage to the non-load bearing timber partitions was extensive.
The explosions generated during the tests would be classed as ‘moderate’ – see Appendix B.

This form of testing is useful in demonstrating the likely magnitude of the explosion that might arise from a ruptured aerosol canister of the types commonly used in the UK, and provides reassurance that structural damage is unlikely to occur in LPS dwelling blocks when subjected to overpressures of the magnitude recorded during the tests.

**E4  FULL-SCALE LOAD TESTING TO FAILURE OF EXISTING LPS DwELLING BLOCKS**

In the period 1996–1998 BRE undertook a programme of full-scale load tests upon existing LPS dwelling blocks to provide an understanding of the way these buildings might respond to an accidental load arising from a gaseous explosion. The work was sponsored by the then Housing Directorate, Department of the Environment, Transport and the Regions (DETR). The detail, implications and use of results of these tests are briefly set out below: A draft BRE information paper was prepared at that time describing these tests, but was never approved for publication.

Static load testing was performed, firstly, on a high-rise Bison Wallframe LPS dwelling block (kindly made available by Sandwell Metropolitan Borough Council) and, secondly, on a Reema Conclad LPS dwelling block (kindly made available by Leeds City Council). Both LPS dwelling blocks were built prior to the partial collapse of Ronan Point in 1968.

The principal goal behind the design of the loading system was to replicate, as far as was practical and economic, the maximum mid-span bending moment and maximum shear force at the supports which would result from a uniformly distributed load induced by the uniformly distributed pressure arising from a gas explosion within a single room. The loading scheme adopted for these tests was based on a series of hydraulic jacks, adjustable aluminium loading shores and hollow rectangular steel box load distribution beams which were fixed to the elements under test. The test loads were applied simultaneously to the cross and flank walls, and to the floor and ceiling slabs, in the combined wall and floor tests. Separate hydraulic circuits were employed to allow different proportions of load to be applied to the shear and bending load distribution beams. The magnitude and rate at which the loads were applied was controlled by manually operated hydraulic pumps linked together to form four independent loading circuits (i.e. FSB = floor and ceiling slab bending, FSS = floor and ceiling slab shear, WPB = wall panel bending and WPS = wall panel shear). The methodology adopted for these tests was very similar to the overpressure tests undertaken later in 2004 upon a Bison Wallframe LPS dwelling block situated in Liverpool, which is described in section G5 in Appendix G.

Due to the complexity and congestion of the floor and cross/flank wall loading rig and financial constraints, it was considered impractical to load test the spandrel wall panel to assess its behaviour under accidental loading. Accordingly, the combined wall and floor overpressure tests were undertaken with the elevation spandrel wall panel in place. It was not possible to directly check the conjecture that the elevation spandrel wall panel would remain in place during an internal gas explosion, but with the loss of the associated windows and frames.

If the elevation wall panel remained in place during an internal gas explosion, it would be expected to provide a degree of tying between one end of the outer flank wall panel and the adjacent end of the cross wall panel, providing that mechanical ties had been provided at the ends of the elevation wall panel. Such tying would tend to restrict any out of plane movement that might occur at the ends of the cross and flank wall panels. On this basis, the failure of the cross and/or flank walls might be expected to occur at a higher overpressure than that generated during the quasi static load tests conducted by BRE, in the event of an actual internal gas explosion.

Regardless of this short fall in practical evidence during the earlier programme of testing, the more recent load tests (see Appendix G) undertaken by BRE in the Liverpool Bison Wallframe LPS dwelling block demonstrated that the flank and cross wall panels were able to accommodate lateral test loads which were in excess of the notional accidental loading associated with the 17 kN/m² load criterion. This was achieved without the presence of a concrete elevation wall panel to provide a lateral tie as the Liverpool Bison block incorporated a non-load bearing half glazed timber infill elevation balcony panel and not a storey height precast concrete panel.

During the combined wall and floor overpressure tests several forms of protection were employed to prevent excessive movement of the wall and floor panels bordering the test room in the event of premature failure of any of the components and associated joints under test. These measures included, safety propping, steel angle brackets (nuts to anchorage bolts remained loose during testing) and high tensile horizontal tie rods. These features were introduced in a manner so that they did not affect the behaviour of the structure during the tests. The installed bolts, however, could be tightened up after the completion of the BRE testing to allow the LPS dwelling blocks to be left in a safe condition prior to them being demolished.

The physical response of the structure to the applied loads was measured using displacement transducers mounted on small diameter holes cored in the floor and ceiling slabs to the test room. The independent instrumentation frame which passed through small diameter holes cored in the floor and ceiling slabs to the test room. The independent instrumentation frame was mounted on the floor level below the test room, in order that it was not disturbed by the movement of the structure as the testing was undertaken.

Summarising, the following combined wall and floor overpressure tests were undertaken:

- **Bison Wallframe LPS dwelling block, Sandwell:**
  - 19th floor flank wall flat, with the test being carried out in the lounge (long floor span) which was...
situated between the flank wall and the opposing (first) cross wall.

- 19th floor internal flat, with the test being carried out in the kitchen (short floor span) which was situated between two opposing cross walls.

- Reema Conclad LPS dwelling block, Leeds:
  - 8th floor flank wall flat, with the test being carried out in a bedroom (short floor span) which was situated between the flank wall and the opposing (first) cross wall.

The following floor overpressure testing was undertaken (ie no overpressure loads were applied simultaneously to the walls of the test room):

- Bison Wallframe LPS dwelling block, Sandwell:
  - A series of preliminary floor tests had previously been undertaken on the lower floors of the Bison Wallframe LPS dwelling block to gain an understanding of the potential behaviour of the structure and an indication of the ultimate load capacity of the floor slabs. These tests revealed that the floor slabs had a surprisingly high ultimate load capacity, which was demonstrated again when the combined wall and floor overpressure tests were subsequently undertaken.

- Reema Conclad LPS dwelling block, Leeds:
  - This was undertaken in a 4th floor internal flat, with the test being carried out in the lounge (long floor span) which was situated between two opposing cross walls.

Summary details of the various static load tests conducted by BRE are given in Table E1, Parts 1–4.

In calculating the total load to be applied to the floors and walls bordering each test room and its distribution between the (central) bending and (edge) shear loading systems it was necessary to make a number of simplifying assumptions. These related to the boundary support conditions of the floor and wall panels, the likely interaction between adjacent floor panels and the load distribution perpendicular to the span of the loaded elements. The maximum central bending and edge shear loads equivalent to a uniformly distributed load were calculated for both walls and floors, as appropriate. These calculations were based upon the results of a linear finite element analysis undertaken prior to testing of selected elements bordering the test room.

The duration and rate of application of the loads applied to the walls and floors during the load tests were relatively long compared to that which would result from a gas explosion. Consequently, any potential benefits arising from higher strain rate effects were not mobilised during the tests. Thus in an actual internal gas explosion, it is likely that a Bison Wallframe and Reema Conclad LPS dwelling blocks would perform better than the behaviour exhibited during the full-scale combined wall and floor tests which were static load tests.

The test results are considered to have conclusively shown that wall and floor panels located two storeys down from the roof would have been able to resist the specified 17 kN/m² overpressure loading criterion associated with an LPS dwelling block without a piped gas supply.

As such, it was judged that no key vertical load bearing wall elements would have been removed, or damaged sufficiently, for there to have been a need to mobilise alternative load paths. Accordingly, the particular LPS dwelling blocks tested by BRE were judged sufficiently strong to resist the accidental loads associated with typical non-piped gas explosions.

The findings of the load tests conducted by BRE have significant implications for the structural assessment of other Reema Conclad and Bison Wallframe LPS dwelling blocks:

- which were built prior to the partial collapse of Ronan Point in 1968,
- which are without a piped gas supply, and
- in which comprehensive physical investigations have been undertaken that are judged to have demonstrated that a reasonable standard of workmanship has been achieved.

The results confirmed what many engineers undertaking this type of structural assessment have long suspected; that significant lateral frictional/adhesive forces appear to exist in the joint at the bottom of the individual load bearing wall panels.

Some elements of the current guidance for assessment of LPS dwelling blocks indicate that such forces should be disregarded, leading to difficulties in justifying the lateral stability of loadbearing wall panels under the specified accidental overpressure loading criterion. These forces appear to be present even in situations where the uplift forces to the soffit of the upper floor of the test room (located two storeys below roof level) were actually seen to be lifting the part of the LPS dwelling block above the test room (by up to about 3 mm).

The work also demonstrated that, in suitable circumstances, static test loading could potentially be used to assess the safety of particular designs or individual LPS dwelling blocks where analytical methods have not been able to demonstrate an adequate margin of safety for the appropriate accidental overpressure loading criterion (ie 17 kN/m² or 34 kN/m²).

It is possible that the levelling dowels located within the wall panels would also provide some degree of lateral restraint and therefore play some role in restricting lateral out-of-plane movement of the base of the wall panels. However, this has not been established and the magnitude of such a contribution could vary greatly, depending upon local circumstances. It is possible to envisage circumstances where the potential contribution of the levelling dowels to the lateral resistance of the base of the wall panels would be small or possibly even absent. This might occur when the levelling bolts were located in a void at the base of the wall panel and the levelling nuts had been wound down after installation of the mortar dry pack in the joint at the base of the wall panel. In such circumstances the levelling bolts would not be transferring vertical load and, accordingly, would not be expected to mobilise significant lateral frictional force. As such the pre-failure/pre-slip lateral resistance of the levelling
### Table E1: Summary details of static load tests conducted in LPS dwelling blocks by BRE undertaken before 2000

#### Table E1: Part 1  Bison Wallframe LPS dwelling block: Test 1

<table>
<thead>
<tr>
<th>Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>LPS block type and location</td>
<td>Bison Wallframe LPS dwelling block, Sandwell</td>
</tr>
<tr>
<td>LPS block details</td>
<td>22 storey block: see Figure 1</td>
</tr>
<tr>
<td>Type of load test</td>
<td>Room test: combined wall and floor overpressure test</td>
</tr>
<tr>
<td>Load test location</td>
<td>19th floor internal flat: lounge (short floor span)</td>
</tr>
<tr>
<td></td>
<td>Test room situated between two cross walls in the centre of the block</td>
</tr>
<tr>
<td>Dimensions of test room</td>
<td>Span between support walls = 3.21 m; width of test room = 5.3 m</td>
</tr>
<tr>
<td><strong>Floor slab test results:</strong></td>
<td></td>
</tr>
<tr>
<td>End of linear behaviour</td>
<td>Equivalent overpressure for test load – end of linear response = 17 kN/m²</td>
</tr>
<tr>
<td>Maximum applied test load</td>
<td>Equivalent overpressure for maximum applied test load = 24 kN/m²</td>
</tr>
<tr>
<td><strong>Wall panel test results:</strong></td>
<td></td>
</tr>
<tr>
<td>End of linear behaviour</td>
<td>Equivalent overpressure for test load – end of linear response = 14 kN/m²</td>
</tr>
<tr>
<td>Maximum applied test load</td>
<td>Equivalent overpressure for maximum applied test load = 18 kN/m²</td>
</tr>
</tbody>
</table>

#### Table E1: Part 2  Bison Wallframe LPS dwelling block: Test 2

<table>
<thead>
<tr>
<th>Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>LPS block type and location</td>
<td>Bison Wallframe LPS dwelling block, Sandwell</td>
</tr>
<tr>
<td>LPS block details</td>
<td>22 storey block: see Figure 1</td>
</tr>
<tr>
<td>Type of load test</td>
<td>Room test: combined wall and floor overpressure test</td>
</tr>
<tr>
<td>Load test location</td>
<td>19th floor internal flat: kitchen (short floor span)</td>
</tr>
<tr>
<td></td>
<td>Test room situated between two cross walls in the centre of the block</td>
</tr>
<tr>
<td>Dimensions of test room</td>
<td>Span between support walls = 2.28 m; width of test room = 3.46 m</td>
</tr>
<tr>
<td><strong>Floor slab test results:</strong></td>
<td></td>
</tr>
<tr>
<td>End of linear behaviour</td>
<td>Equivalent overpressure for test load – end of linear response = 17 kN/m²</td>
</tr>
<tr>
<td>Maximum applied test load</td>
<td>Equivalent overpressure for maximum applied test load = 24 kN/m²</td>
</tr>
<tr>
<td><strong>Wall panel test results:</strong></td>
<td></td>
</tr>
<tr>
<td>End of linear behaviour</td>
<td>Equivalent overpressure for test load – end of linear response = 14 kN/m²</td>
</tr>
<tr>
<td>Maximum applied test load</td>
<td>Equivalent overpressure for maximum applied test load = 18 kN/m²</td>
</tr>
</tbody>
</table>

#### Table E1: Part 3  Reema Conclad LPS dwelling block: Test 1

<table>
<thead>
<tr>
<th>Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>LPS block type and location</td>
<td>Reema Conclad LPS dwelling block, Leeds</td>
</tr>
<tr>
<td>LPS block details</td>
<td>10 storey block: see Figure 2</td>
</tr>
<tr>
<td>Type of load test</td>
<td>Room test: combined wall and floor overpressure test</td>
</tr>
<tr>
<td>Load test location</td>
<td>8th floor flank wall flat: bedroom</td>
</tr>
<tr>
<td></td>
<td>Test room situated between flank wall and first cross wall</td>
</tr>
<tr>
<td>Dimensions of test room</td>
<td>Span between support walls = 3.21 m; width of test room = 3.77 m</td>
</tr>
<tr>
<td><strong>Floor slab test results:</strong></td>
<td></td>
</tr>
<tr>
<td>End of linear behaviour</td>
<td>Equivalent overpressure for test load – end of linear response = 17 kN/m²</td>
</tr>
<tr>
<td>Maximum applied test load</td>
<td>Equivalent overpressure for maximum applied test load = 27 kN/m²</td>
</tr>
<tr>
<td><strong>Wall panel test results:</strong></td>
<td></td>
</tr>
<tr>
<td>End of linear behaviour</td>
<td>Equivalent overpressure for test load – end of linear response = 17 kN/m²</td>
</tr>
<tr>
<td>Maximum applied test load</td>
<td>Equivalent overpressure for maximum applied test load = 26 kN/m²</td>
</tr>
</tbody>
</table>

#### Table E1: Part 4  Reema Conclad LPS dwelling block: Test 2

<table>
<thead>
<tr>
<th>Description</th>
<th>Details</th>
</tr>
</thead>
<tbody>
<tr>
<td>LPS block type and location</td>
<td>Reema Conclad LPS dwelling block, Leeds</td>
</tr>
<tr>
<td>LPS block details</td>
<td>10 storey block: see Figure 2</td>
</tr>
<tr>
<td>Type of load test</td>
<td>Floor overpressure test</td>
</tr>
<tr>
<td>Load test location</td>
<td>4th floor internal flat: lounge (long floor span)</td>
</tr>
<tr>
<td></td>
<td>Test room situated between two cross walls in the centre of the block</td>
</tr>
<tr>
<td>Dimensions of test room</td>
<td>Span between support walls = 4.56 m; width of test room = 4.03 m</td>
</tr>
<tr>
<td><strong>Floor slab test results:</strong></td>
<td></td>
</tr>
<tr>
<td>End of linear behaviour</td>
<td>Equivalent overpressure for test load – end of linear response = 20 kN/m²</td>
</tr>
<tr>
<td>Maximum applied test load</td>
<td>Equivalent overpressure for maximum applied test load = 36 kN/m²</td>
</tr>
</tbody>
</table>
dowels would be expected to be small. Once significant lateral slip had occurred in the joint at the base of the wall panel, such that mechanical interlock and physical deformation of the levelling bolts was occurring, the levelling bolts would be expected to provide contribution to post-failure/slip lateral resistance of the base of the wall panels.

**E5  INFLUENCE OF LOW CONCRETE COMPRESSIVE STRENGTH**

On a limited number of occasions BRE has found that the compressive strength of the concrete used in an LPS dwelling block was exceptionally low. This may apply to particular localised parts of the block concerned or more rarely, because the concrete was generally of poor quality, to the LPS dwelling block overall. A low compressive strength affects other properties of the concrete and would, for example, reduce the flexural and tensile strength of the precast concrete floor and wall panels affected. Low strength issues can also affect the in-situ joints between precast concrete panels.

In these circumstances the risk of structural failure may extend below the upper storeys of an LPS dwelling block, which are traditionally considered to be most at risk from the overpressure loading associated with an internal gas explosion. In such circumstances it may therefore be necessary to give consideration to the potential effect and implications of a ‘severe’ internal gas explosion occurring at any level within the LPS dwelling block concerned.
Plate 1: Direct stress contours in Leeds LPS 4th floor lounge floor slab for an applied uniformly distributed overpressure load (1 kN/m²): as associated with an internal gas explosion

Plate 2: Direct stress contours in Leeds LPS lounge floor slab from applied test load
Plate 3: Linear finite element model (only lower five storeys of block shown) showing predicted deflections in the vertical (z) direction (kitchen–lounge spine wall present)

Plate 4: Linear finite element model (only lower five storeys of block shown) showing predicted deflections in the vertical (z) direction (kitchen–lounge spine wall absent)
Plate 5: Linear finite element model (only lower three storeys of block shown) showing predicted deflections in the vertical (z) direction (kitchen–lounge spine wall present)

Plate 6: Linear finite element model (only lower three storeys of block shown) showing predicted deflections in the vertical (z) direction (kitchen–lounge spine wall absent)
Plate 7: Solid element model of half of a lounge test floor slab (symmetry assumed about mid-span of test floor)

Plate 8: General view of finite element model showing cast-in voids in the precast concrete lounge floor slabs
Plate 9: Type and distribution of steel reinforcement in finite element model of the precast concrete lounge floor slabs

Yellow: R6 bars at top
Red: T13 bars at bottom
Blue: R14 Bars at bottom
Green: T16 bars at bottom
Orange: R6 bars at bottom
Plate 10: Vertical displacement pattern predicted by the numerical analysis for the flank wall flat lounge floor slab (symmetry assumed about mid-span of test floor)

Plate 11: Stress distribution (von Mises) as predicted by numerical analysis at concrete face immediately below floating screed for the flank wall flat lounge floor slab (symmetry assumed about mid-span of test floor)
Plate 12: Development of cracks in the soffit of the flank wall flat lounge floor slab (symmetry assumed about mid-span of test floor) – low load intensity: Partition present

Plate 13: Development of cracks in the soffit of the flank wall flat lounge floor slab (symmetry assumed about mid-span of test floor) – high load intensity: Partition present
Plate 14: Development of cracks in the soffit of the flank wall flat lounge floor slab (symmetry assumed about mid-span of test floor) – low load intensity: Partition absent

Plate 15: Development of cracks in soffit of floor slabs – high load intensity: Partition absent
APPENDIX F

OVERVIEW OF FINITE ELEMENT ANALYSES AND CALIBRATION EXERCISES FOR THE 1990s’ BRE LOAD TESTS ON LPS DWELLING BLOCKS

F1  INTRODUCTION
Section E4, in Appendix E gives details of the 1990s programme of BRE full-scale load tests on LPS dwelling blocks. The static load testing was undertaken to establish the ultimate load capacity of selected parts of LPS dwelling blocks subject to an applied test loading regime designed to simulate the uniform overpressure loading associated with internal gas explosions.

The combined wall and floor overpressure tests were not just a test of the strength of the wall and floor components making up the LPS dwelling block, but were an evaluation of the potential behaviour of these components working as a three-dimensional structural system. The full-scale load tests were taken as close to the point of structural failure as seemed prudent as each test was performed.

The full-scale load tests were performed on a high-rise Bison Wallframe LPS dwelling block (kindly made available by Sandwell Metropolitan Borough Council) and on a Reema Conclad LPS dwelling block (kindly made available by Leeds City Council). Section E4 in Appendix E gives details of these tests.

The LPS dwelling blocks were essentially tested as found, apart from the fact that any supplementary strengthening which had been installed previously in the vicinity of the test locations (ie post-Ronan Point strengthening) was removed. Non-loadbearing partitions which formed part of the original building, such as those acting as a divider between the kitchen and lounge, were not removed. It was considered that such components could potentially contribute to the strength of the building in the event of an internal gas explosion occurring, and therefore should be retained. Thus, the combined wall and floor overpressure tests sought to evaluate the behaviour and ultimate strength of the as constructed/as found LPS dwelling blocks.

F2  METHODOLOGY FOR CALCULATING ‘EQUIVALENT’ UNIFORMLY DISTRIBUTED LOAD
For practical reasons the applied test loads could not be uniformly distributed on the surfaces of wall and floor components. Thus, it was necessary to obtain an estimate of the uniformly distributed overpressure load which is ‘equivalent’ to the static test load applied to each wall or floor component using the system of shear and bending load distribution beams (see section G4 in Appendix G for further details).

This exercise was undertaken by means of finite element analyses carried out using the general-purpose finite element program ANSYS. Whilst more realistic ‘equivalent’ uniformly distributed loads (UDL) could potentially have been calculated by undertaking analyses that considered both geometric and material nonlinearities, project resource constraints did not permit this. Thus the comparison and calibration exercises undertaken and explained here were carried out using linear elastic finite element analysis. Accordingly it should be recognised that the use of a linear elastic approach, as opposed to a nonlinear approach, is likely to result in a less accurate determination of the actual equivalent UDL’s for the maximum applied test load (by which stage the behaviour of the wall and floor components had generally become highly non-linear).

In the numerical analyses the response of the structure was calculated for a nominal test load of 100 kN. This load was apportioned between shear and bending in a manner similar to that adopted for the combined wall and floor overpressure test being considered. These results were compared with those for an analysis assuming a uniformly distributed (overpressure) load; allowing an overpressure load (UDL100) ‘equivalent’ to the applied static test load to be estimated.

For example, in the case of the fourth floor lounge – floor overpressure test carried out in the Reema Conclad LPS dwelling block in Leeds, it was estimated that a uniformly distributed (overpressure) load of about 7 kN/m² would give the same magnitude of stresses and support reactions as a total applied test load of 100 kN. Thus, in this case UDL100 = 7 kN/m². A variety of stress and support reaction parameters were considered in the evaluation. These gave a range of estimates for the ‘equivalent’ uniformly distributed (overpressure) load (UDL100) ranging from 5.6 to 8.6 kN/m².

The requirement was to estimate ‘equivalent’ uniformly distributed (overpressure) loads for two test load conditions, namely:
1. The point at which the response of the wall and floor components to the applied test load became nonlinear or more markedly so. This point was termed the lower bound uniformly distributed load, UDL_L.
2. The maximum test load applied to the wall and floor components. This point was termed the upper bound uniformly distributed load, UDL_U.
The lower bound (UDLLB) and upper (UDLU) estimates for the ‘equivalent’ UDLs were calculated using the relevant loads determined from the responses of the wall and floor components measured during the particular load test. Summarising, the results of these calculations (discussed further below) were given in terms of the following:
1. The ‘equivalent’ uniformly distributed load (UDL100) which provides the same maximum response (at either the same or similar close locations) as a total test load of 100 kN.
2. The test loads corresponding to the onset of more prominent nonlinear behaviour. This corresponds to the ‘equivalent’ lower bound uniformly distributed load (UDLLB) value.
3. Upper values that are assumed to correspond to the maximum applied test load. This corresponds to the ‘equivalent’ upper bound uniformly distributed load (UDLU) value.

The largest values in the range of estimates for the ‘equivalent’ uniformly distributed loads (UDLs) give an indication of the maximum loads which the wall and floor components might sustain, based upon idealised linear behaviour. However, as only linear analysis of the wall and floor components was carried out, it was considered more appropriate to take the conservative lower range values.

**F3 METHOD OF ANALYSIS: COMBINED WALL AND FLOOR OVERPRESSURE TESTS**

During the combined wall and floor overpressure tests the cross and flank walls and upper and lower floor slabs were loaded simultaneously. In order to accurately model the behaviour of the floor and wall it would be necessary to simultaneously consider the whole structural system. With this approach any in-plane effects would be taken into consideration. However, project resource constraints did not permit this and it was only possible to carry out linear elastic analyses in which the supports were assumed to be rigid. In this approach the walls and floors were analysed separately without considering the interaction between them.

As linear elastic analysis was employed, both the test room floor and ceiling slabs could be evaluated using the same finite element model; but attention was required to ensure that the test loading was being applied in the appropriate direction (ie upward or downward).

For the testing undertaken in flank wall flats it was only necessary to model the behaviour of the flank wall panels within the test room, as the presence of the doorway in the adjacent cross wall meant that the cross wall tended to be less highly stressed than the flank wall. This occurred because of the nature of the overpressure test loading arrangement employed, which necessitated a load reaction column being positioned in the room doorway.

In the case of testing undertaken in internal flats (ie situated away from the flank walls of the LPS dwelling block), the combined wall and floor overpressure tests were undertaken between two cross walls. For the reasons noted above, again it was only necessary to model one of the cross walls.

**F3.1 Floor slab analyses**

In modelling the floor slabs, a smeared approach was used with respect to the reinforcements and, where appropriate, hollow slab cross sections. The floor slabs were considered as either a 3-layer or a 5-layer material with appropriate equivalent properties. The top and bottom layers represented the solid concrete and reinforcements therein. The middle layer represented the partially hollow region.

The Shell-91 element of ANSYS was used in the finite element modelling. The analyses assumed concrete elastic modulus values between 20 GPa and 30 GPa and a steel elastic modulus of 210 GPa. The presence of the reinforced hollow slab with equivalent properties made it necessary to consider the slab as an orthotropic one.

In some cases it was possible to take advantage of the symmetry of the floor slabs, in which case only one quarter of each floor slab was analysed by considering relevant symmetrical boundary conditions. In other cases it was necessary to model the whole floor slab. In all cases the floor slab was assumed to be ‘simply supported’ on rigid wall panels. Because of the presence in the middle of the test room of the unsupported and unreinforced joint between the two floor slabs, the boundary between them was taken as being unstressed.

**F3.2 Wall panel analyses**

The finite element modelling of the flank wall was carried out in a manner similar to that described above for the floor slabs, except that the wall panels were treated as being unreinforced solid walls.

The boundary conditions along the upper and lower edges of the flank wall varied continuously throughout the overpressure load test. This was as a result of the initiation and development of cracking along the horizontal junctions between the wall panels and floor slabs. As a result two main sets of boundary conditions were considered:
1. Where the wall panels were continuously pin jointed along their upper and lower edges.
2. Where the wall panels were laterally restrained by a number of dowel bars (number varied), with the upper and lower edges of the wall panel being treated as either pinned or clamped (ie either free to rotate or were restrained against rotation).

**F4 METHOD OF ANALYSIS: FLOOR OVERPRESSURE TESTS**

The same methodology was employed for the finite element modelling of the floor overpressure tests as has been described for the combined wall and floor overpressure tests in section F3 above.

**F5 REEMA CONCLAD LPS DWELLING BLOCK, LEEDS**

**F5.1 Floor slab analyses: 4th floor lounge: floor overpressure test**

Plate 1 shows the contours of direct stress in the top of the floor slab for an applied uniformly distributed (overpressure) load of 1 kN/m². Plate 2 shows the contours...
of this stress under a total applied load of 8.5 kN (for the whole test floor) apportioned between shear and bending in a manner similar to that adopted during the load test.

As explained above, the finite element analyses results allowed the following ‘equivalent’ uniformly distributed values to be calculated for a number of stress and support reaction parameters.

1. The ‘equivalent’ 100 kN test load value, UDL₁₀₀.
2. The ‘equivalent’ lower bound uniformly distributed load value corresponding to the end of linear behaviour, UDLLₜₕ. The end of linear behaviour occurred at a test load of about 350 kN.
3. The ‘equivalent’ upper bound uniformly distributed load value corresponding to the maximum applied test load, UDLᵤ. The maximum applied test load was about 640 kN.

It is the second two parameters which are of principal interest in terms of the interpretation of the applied test loads as ‘equivalent UDL’ overpressure values. In the following these values are taken conservatively at the lower end of the calculated ranges. Table F1 presents the ‘equivalent UDL’ values for the Reema Conclad LPS dwelling block, Leeds fourth floor lounge: floor overpressure test.

Thus the estimated equivalent overpressure values are as follows:
(a) End of linear behaviour (350 kN test load) at about 20 kN/m²: see Figures F1–F4.
(b) The maximum applied test load (640 kN) of about 36 kN/m²: see Figures F1–F4.

Figures F1 and F2 present the measured load-deflection graphs for the upward movement of the two central ceiling slabs under the applied test load. Figures F3 and F4 present the measured load-deflection graphs for the downward movement of the two central floor slabs in the test room. Figures F5 and F6 illustrate the locations of the

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Calculated range</th>
<th>Equivalent UDL</th>
</tr>
</thead>
<tbody>
<tr>
<td>‘Equivalent’ 100 kN test load value, UDL₁₀₀</td>
<td>5.6 to 8.6 kN/m²</td>
<td>Not applicable</td>
</tr>
<tr>
<td>‘Equivalent’ lower bound uniformly distributed load value corresponding to the end of linear behaviour, UDLLₜₕ</td>
<td>19.6 to 26.3 kN/m²</td>
<td>20 kN/m²</td>
</tr>
<tr>
<td>‘Equivalent’ upper bound uniformly distributed load value corresponding to the maximum applied test load, UDLᵤ</td>
<td>35.8 to 55.0 kN/m²</td>
<td>36 kN/m²</td>
</tr>
</tbody>
</table>

Figure F1: Upward deflection of ceiling slab in Leeds LPS 4th floor lounge floor load test, Sensor Group 1. The respective data points are shown in the key.
Figure F2: Upward deflection of ceiling slab in Leeds LPS 4th floor lounge floor load test, Sensor Group 2. The respective data points are shown in the key.

Figure F3: Downward deflection of floor slab in Leeds LPS 4th floor lounge floor load test, Sensor Group 1. The respective data points are shown in the key.
Figure F4: Downward deflection of floor slab in Leeds LPS 4th floor lounge floor load test, Sensor Group 2. The respective data points are shown in the key.

Figure F5: Plan of ceiling slab showing location of upward deflection monitoring transducers (Floor T+1): Leeds LPS 4th floor lounge floor load test.

Figure F6: Plan of floor slab showing location of downward deflection monitoring transducers (Floor T): Leeds LPS 4th floor lounge floor load test.
deflection monitoring transducers for the ceiling and floor slabs for the fourth floor lounge floor load test.

The 20 kN/m² and 36 kN/m² 'equivalent' values are significantly in excess of the 17 kN/m² overpressure value used in structural assessments for accidental loading in LPS dwelling blocks without a piped gas supply. This suggests that the fourth floor lounge floor would have been easily able to resist the forces associated with a 'severe' non-piped gas explosion.

F5.2 Floor slab analyses: 8th floor bedroom: Combined wall and floor overpressure test

Table F2 presents the 'equivalent UDL' values for the Reema Conclad LPS dwelling block, Leeds 8th floor bedroom floor, for the combined wall and floor overpressure test (room test).

Thus the estimated equivalent overpressure values are as follows:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Calculated range</th>
<th>Equivalent UDL</th>
</tr>
</thead>
<tbody>
<tr>
<td>'Equivalent' 100 kN test load value, UDL_{100}</td>
<td>8.6 to 12.3 kN/m²</td>
<td>Not applicable</td>
</tr>
<tr>
<td>'Equivalent' lower bound uniformly distributed load value corresponding to the end of linear behaviour, UDL_{LB}</td>
<td>16.5 to 18.5 kN/m²</td>
<td>17 kN/m²</td>
</tr>
<tr>
<td>'Equivalent' upper bound uniformly distributed load value corresponding to the maximum applied test load, UDL_{UB}</td>
<td>21.3 to 30.5 kN/m²</td>
<td>27 kN/m²</td>
</tr>
</tbody>
</table>

(a) End of linear behaviour (150 kN test load) at about 17 kN/m²: see Figures F7–F11.
(b) The maximum applied test load (248 kN) of about 27 kN/m²: see Figures F7–F11.

Figures F7 and F8 present the measured load-deflection graphs for the upward movement of the two ceiling slabs under the applied test load. Figures F9 and F10 present the measured load-deflection graphs for the downward movement of the two floor slabs in the test room. Figures F11 and F12 illustrate the locations of the deflection monitoring transducers for the ceiling and floor slabs for the 8th floor bedroom room load test.

The maximum applied test load 'equivalent' value of 27 kN/m² is significantly in excess of the 17 kN/m² overpressure value used in structural assessments for accidental loading in LPS dwelling blocks without a piped gas supply. This suggests that the 8th floor bedroom floor and ceiling slabs would have been able to resist the forces associated with a 'severe' non-piped gas explosion.

Table F2: ‘Equivalent UDL’ values for the Reema Conclad LPS dwelling block, Leeds 8th floor bedroom: Combined wall and floor overpressure test: Floor slab

Figure F7: Upward deflection of ceiling slab in Leeds LPS 8th floor bedroom: Room test, Sensor Group 1. The key identifies the data points for the structural element being load tested.
APPENDIX F  OVERVIEW OF FINITE ELEMENT ANALYSES AND CALIBRATION EXERCISES FOR THE 1990’s BRE LOAD TESTS

Figure F8: Upward deflection of ceiling slab in Leeds LPS 8th floor bedroom: Room load test, Sensor Group 2. The key identifies the data points for the structural element being load tested.

Figure F9: Downward deflection of floor slab in Leeds LPS 8th floor bedroom: Room load test, Sensor Group 1. The key identifies the data points for the structural element being load tested.
F5.3 Wall panel analyses: 8th floor bedroom: Combined wall and floor overpressure test

Table F3 presents the ‘equivalent UDL’ values for the Reema Conclad LPS dwelling block, Leeds 8th floor bedroom flank wall, for the combined wall and floor overpressure test (room test).

Thus the estimated equivalent overpressure values are as follows:
(a) End of linear behaviour (90 kN test load) at about 17 kN/m²: see Figures F13 and F14.
(b) The maximum applied test load (138 kN) of about 27 kN/m²: see Figures F13 and F14.
Figure F13 shows the measured load-deflection graphs for the flank wall, with Figure F14 showing the results for the associated opposing cross wall. Figures F15 and F16 illustrate the locations of the displacement monitoring transducers for the flank wall and the opposing cross wall for the 8th floor bedroom room load test.

The maximum applied test load ‘equivalent’ value of 26 kN/m² is significantly in excess of the 17 kN/m² overpressure value used in structural assessments for accidental loading in LPS dwelling blocks without a piped gas supply. This suggests that the 8th floor bedroom flank wall and opposing cross wall would have been able to resist the forces associated with a ‘severe’ non-piped gas explosion.

Table F3: ‘Equivalent UDL’ values for the Reema Conclad LPS dwelling block, Leeds 8th floor bedroom: Combined wall and floor overpressure test: Flank wall

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Calculated range</th>
<th>Equivalent UDL</th>
</tr>
</thead>
<tbody>
<tr>
<td>‘Equivalent’ 100 kN test load value, $UDL_{100}$</td>
<td>18.5–32.6 kN/m²</td>
<td>Not applicable</td>
</tr>
<tr>
<td>‘Equivalent’ lower bound uniformly distributed load value corresponding to the end of linear behaviour, $UDL_{LB}$</td>
<td>16.7–29.3 kN/m²</td>
<td>17 kN/m²</td>
</tr>
<tr>
<td>‘Equivalent’ upper bound uniformly distributed load value corresponding to the maximum applied test load, $UDL_{U}$</td>
<td>25.5–45.0 kN/m²</td>
<td>26 kN/m²</td>
</tr>
</tbody>
</table>

Figure F13: Lateral deflection of flank wall in Leeds LPS 8th floor bedroom: Room load test. The key identifies the data points for the structural element being load tested.

Table F4 presents the ‘equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 8th floor bedroom: Combined wall and floor overpressure test (room test).

F6.1 Floor slab analyses: 19th floor lounge: Combined wall and floor overpressure test

Table F4 presents the ‘equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 19th floor lounge floor, for the combined wall and floor overpressure test (room test).

Thus the estimated equivalent overpressure values are as follows:
(a) End of linear behaviour (240 kN test load) at about 17 kN/m²: see Figures F17–F21.
(b) The maximum applied test load (316 kN) of about 24 kN/m²: see Figures F17–F21.
**Figure F14:** Lateral deflection of cross wall in Leeds LPS 8th floor bedroom: Room load test. The key identifies the data points for the structural element being load tested.

**Figure F15:** Elevation of flank wall showing location of displacement monitoring transducers: Leeds LPS 8th floor bedroom: Room load test.

**Figure F16:** Elevation of cross wall showing location of displacement monitoring transducers: Leeds LPS 8th floor bedroom: Room load test.

### Table F4: ‘Equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 19th floor lounge: Combined wall and floor overpressure test: Floor slab

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Calculated range</th>
<th>Equivalent UDL</th>
</tr>
</thead>
<tbody>
<tr>
<td>‘Equivalent’ 100 kN test load value, $UDL_{100}$</td>
<td>7.5–11.6 kN/m²</td>
<td>Not applicable</td>
</tr>
<tr>
<td>‘Equivalent’ lower bound uniformly distributed load value corresponding to</td>
<td>18.0–27.8 kN/m²</td>
<td>17 kN/m²</td>
</tr>
<tr>
<td>the end of linear behaviour, $UDL_{\text{L}}$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>‘Equivalent’ upper bound uniformly distributed load value corresponding to</td>
<td>23.7–36.7 kN/m²</td>
<td>24 kN/m²</td>
</tr>
<tr>
<td>the maximum applied test load, $UDL_{u}$</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Figure F17: Upward deflection of ceiling slab adjacent kitchen (Floor T+1) in Sandwell Bison LPS 19th floor lounge: Room load test. The key identifies the data points for the structural element being load tested.

Figure F18: Upward deflection of ceiling slab adjacent elevation wall/window panel (Floor T+1) in Sandwell Bison LPS 19th floor lounge: Room load test. The key identifies the data points for the structural element being load tested.
Figure F19: Downward deflection of floor slab adjacent kitchen (Floor T) in Sandwell Bison LPS 19th floor lounge: Room load test. The key identifies the data points for the structural element being load tested.

Figure F20: Downward deflection of floor slab adjacent elevation wall/window panel (Floor T) in Sandwell Bison LPS 19th floor lounge: Room load test. The key identifies the data points for the structural element being load tested.
Figure F21: Downward mid-span deflection of adjacent floor slabs (Floor T) in Sandwell Bison LPS 19th floor lounge: Room load test. The key identifies the data points for the structural element being load tested.

Figure F22: Plan of floor and ceiling slabs showing location of downward and upward deflection monitoring transducers (Floors T & T+1) in Sandwell Bison LPS 19th floor lounge: Room load test.
Figures F17 and F18 present the measured load-deflection graphs for the upward movement of the two ceiling slabs under the applied test load. Figures F19 and F20 present the measured load-deflection graphs for the downward movement of the two floor slabs in the test room. Figure F21 shows the downward mid-span deflection of adjacent floor slabs, indicating that there is virtually no differential vertical displacement between the two adjacent floor slabs. This suggests that the two floor slabs are effectively working as single floor slab within the dwelling. Figure F22 illustrates the locations of the deflection monitoring transducers for the ceiling and floor slabs for the 19th floor lounge room load test.

The maximum applied test load ‘equivalent’ value of 24 kN/m² is significantly in excess of the 17 kN/m² overpressure value used in structural assessments for accidental loading in LPS dwelling blocks without a piped gas supply. This suggests that the 19th floor lounge floor and ceiling slabs would have been able to resist the forces associated with a non-piped gas explosion.

### F6.2 Wall panel analyses: 19th floor lounge: Combined wall and floor overpressure test

Tables F5 and F6 present the ‘equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 19th floor lounge flank wall, for the combined wall and floor overpressure test (room test). Two analyses are reported because the behaviours of the two panels (i.e. (a) kitchen end panel and (b) window end panel) forming the flank wall were markedly different.

Thus the estimated equivalent overpressure values are as follows:

- (a) End of linear behaviour (180 kN test load) at about 14 kN/m²: see Figures F23–F25.
- (b) The maximum applied test load (237 kN) of about 18 kN/m²: see Figures F23–F25.

Figures F23 to F25 present the measured load–deflection graphs for the lateral out-of-plane movement of the flank wall and cross wall panels under the applied test load. Figure F26 illustrates the locations of the deflection monitoring transducers for the flank wall and cross wall panels for the 19th floor lounge room load test.

The maximum applied test load ‘equivalent’ value of 18 kN/m² is just in excess of the 17 kN/m² overpressure value used in structural assessments for accidental loading in LPS dwelling blocks without a piped gas supply. This suggests that the 19th floor lounge flank wall would have been able to resist the forces associated with a non-piped gas explosion.

**Table F5: ‘Equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 19th floor lounge: Combined wall and floor overpressure test: Flank wall – Kitchen end**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Calculated range</th>
<th>Equivalent UDL</th>
</tr>
</thead>
<tbody>
<tr>
<td>‘Equivalent’ 100kN test load value, UDL&lt;sub&gt;100&lt;/sub&gt;</td>
<td>7.8–23.4 kN/m²</td>
<td>Not applicable</td>
</tr>
<tr>
<td>‘Equivalent’ lower bound uniformly distributed load value corresponding to the end of linear behaviour, UDL&lt;sub&gt;LB&lt;/sub&gt;</td>
<td>14.0–42.1 kN/m²</td>
<td>14 kN/m²</td>
</tr>
<tr>
<td>‘Equivalent’ upper bound uniformly distributed load value corresponding to the maximum applied test load, UDL&lt;sub&gt;U&lt;/sub&gt;</td>
<td>18.5–55.5 kN/m²</td>
<td>18 kN/m²</td>
</tr>
</tbody>
</table>

**Table F6: ‘Equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 19th floor lounge: Combined wall and floor overpressure test: Flank wall – Window end**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Calculated range</th>
<th>Equivalent UDL</th>
</tr>
</thead>
<tbody>
<tr>
<td>‘Equivalent’ 100kN test load value, UDL&lt;sub&gt;100&lt;/sub&gt;</td>
<td>7.6–16.2 kN/m²</td>
<td>Not applicable</td>
</tr>
<tr>
<td>‘Equivalent’ lower bound uniformly distributed load value corresponding to the end of linear behaviour, UDL&lt;sub&gt;LB&lt;/sub&gt;</td>
<td>13.7–29.2 kN/m²</td>
<td>14 kN/m²</td>
</tr>
<tr>
<td>‘Equivalent’ upper bound uniformly distributed load value corresponding to the maximum applied test load, UDL&lt;sub&gt;U&lt;/sub&gt;</td>
<td>18.0–38.4 kN/m²</td>
<td>18 kN/m²</td>
</tr>
</tbody>
</table>
Figure F23: Lateral deflection of flank wall adjacent elevation wall/window panel in Sandwell Bison LPS 19th floor lounge: Room load test. The key identifies the data points for the structural element being load tested.

Figure F24: Lateral deflection of flank wall adjacent kitchen in Sandwell Bison LPS 19th floor lounge: Room load test. The key identifies the data points for the structural element being load tested.
F6.3 Wall panel analyses: 19th floor mid-kitchen: Combined wall and floor overpressure test

Table F7 presents the ‘equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 19th floor mid-kitchen cross walls, for the combined wall and floor overpressure test (room test).

Thus the estimated equivalent overpressure values are as follows:
(a) End of linear behaviour (180 kN test load) at about 36 kN/m²: see Figures F27 and F28.
(b) The maximum applied test load (180 kN) of about 36 kN/m²: see Figures F27 and F28.

Figures F27 and F28 present the measured load-deflection graphs for the lateral out-of-plane displacement of the 19th floor mid-kitchen cross walls under the applied test load. Figure F29 shows the locations of the deflection monitoring transducers for the 19th floor mid-kitchen room load test.

The behaviour of the 19th floor mid-kitchen cross walls was somewhat unusual in as much that the behaviour of most of the deflection gauges was linear with increasing applied test load up to the point at which lateral failure of one of the cross walls occurred. There were no prior indications that lateral failure of the cross...
wall would occur at the point it did. One deflection gauge (transducer location Wall 28) did however exhibited non-linear behaviour from an early stage of the test, but that was not the location at which the lateral (out-of-plane) failure of the bedroom/kitchen cross wall initiated. In this instance the apparent almost total recovery of the cross walls at locations of maximum lateral (out-of-plane) movement (transducer location Wall 23 and Wall 28) after unloading seems a little strange, some degree of permanent set/lateral displacement of the wall would have been expected.

The maximum applied test load ‘equivalent’ value of 36 kN/m² is considerably in excess of the 17 kN/m² overpressure value used in structural assessments for accidental loading in LPS dwelling blocks without a piped gas supply. This suggests that the 19th floor mid-kitchen cross walls would have easily been able to resist the forces associated with a non-piped gas explosion.

### Table F7: ‘Equivalent UDL’ values for the Bison Wallframe LPS dwelling block, Sandwell 19th floor mid-kitchen: Combined wall and floor overpressure test: Cross walls

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Calculated range</th>
<th>Equivalent UDL</th>
</tr>
</thead>
<tbody>
<tr>
<td>‘Equivalent’ 100 kN test load value, UDL₁₀₀</td>
<td>20 kN/m²</td>
<td>Not applicable</td>
</tr>
<tr>
<td>‘Equivalent’ lower bound uniformly distributed load value corresponding to the end of linear behaviour, UDL₁₈</td>
<td>36 kN/m²</td>
<td>36 kN/m²</td>
</tr>
<tr>
<td>‘Equivalent’ upper bound uniformly distributed load value corresponding to the maximum applied test load, UDLₚ</td>
<td>36 kN/m²</td>
<td>36 kN/m²</td>
</tr>
</tbody>
</table>

**Figure F27**: Lateral deflection of bedroom/kitchen cross wall in Sandwell Bison LPS 19th floor mid-flat kitchen: Room load test. The key identifies the data points for the structural element being load tested.
Figure F28: Lateral deflection of lounge/kitchen cross wall in Sandwell Bison LPS 19th floor mid-flat kitchen: Room load test. The key identifies the data points for the structural element being load tested.

Figure F29: Elevations of cross walls showing location of displacement monitoring transducers: Sandwell Bison LPS 19th floor mid-flat kitchen: Room load test.
G1  INTRODUCTION – BACKGROUND AND AIMS OF THE LOAD TESTING

The load testing previously undertaken by BRE in a Bison Wallframe and a Reema Conclad block, as outlined in Appendix E of this document, provided unique information on the full-scale behaviour of selected wall and floor panels (combined wall and floor tests) within each of the two blocks under static loading. Until these tests, such information had not been available.

An initial programme of invasive investigations of the two blocks, together with material testing, was undertaken to evaluate the quality of workmanship, tying and reinforcement provision and material properties. With this information it was possible to judge whether the blocks were likely to be typical or atypical of others blocks of the same system type. An LPS dwelling block was required with satisfactory standard of construction, but not ‘perfect’ workmanship. All the LPS dwelling blocks tested by BRE were empty and awaiting demolition as part of site social redevelopment plans (see Figure G1).

In 2003 a consortium including BRE, six consulting engineering practices, the UK office of an analytical software development company, thirteen local authorities and housing associations, the Northern Ireland Housing Executive, a non-destructive test equipment manufacturer and a Midlands based material test laboratory, were brought together. The group was subsequently able to secure funding from the Department of Trade and Industry via the 2003 Partners in Innovation (PII) scheme.

The following sections describe the nature and outcome of the various load tests, and provide an overview of the subsequent non-linear finite element analysis and calibration exercise that was undertaken as part of the research project.

There were three main phases of work in the Liverpool LPS dwelling block test programme, namely:

Phase 1: Structural investigations to establish details of construction and the quality of workmanship.

Phase 2: Static load testing in the lounge of a flank wall flat on the 14th floor in the chosen block to simulate overpressure loading applied simultaneously to wall and floor elements within a single room. This work was supported by structural calculations, finite element modelling of structural behaviour, etc. This operation is referred to variously as the combined wall and floor overpressure tests and as the full room test – see Figures G2 and G3.

Phase 3: An element removal operation involving the kitchen-lounge spine wall of a flank wall flat on the 1st floor in the chosen block. This work was supported by structural calculations, finite element modelling of structural behaviour, etc. – see Figures G2 and G4.

The load testing previously undertaken by BRE upon the Bison Wallframe (Sandwell) and the Reema Conclad (Leeds) LPS dwelling blocks had involved only two phases of work; the Phase 1 structural investigations and the Phase 2 static load testing at selected locations in the blocks concerned.

G2  SCOPE OF OVERPRESSURE LOAD TESTING AND ELEMENT REMOVAL PROGRAMME

The goal of the static overpressure load testing and the element removal programme in the Liverpool LPS dwelling block was to establish the:

- performance of a selected area of the building relative to the currently employed notional loading criterion of 17 kN/m² (this was taken to act as an equivalent static pressure applied simultaneously to the surfaces of all structural elements bounding the enclosed space containing the notional explosion), and
- ability of the structure to develop alternative load paths by bridging over areas of damage.

Summaries of the results of the invasive investigations, the subsequent load tests and the element removal operation are presented in sections G3–G5.
within at least some of the spine wall/floor slab joints was variable. It was found that significant areas of in-situ concrete were missing or poorly compacted and that levelling dowels cut off and/or bent to aid subsequent construction. Figures G5 and G6 show two examples of two in-situ reinforcement details found during the physical investigations. Examples of poor workmanship had been observed in other Bison blocks previously inspected by BRE.

Figure G5 shows incorrectly installed reinforcement in a flank wall–floor joint, where the bent-up reinforcing bar from the floor slab (at the bottom of the image) only penetrates a very short distance into the flank wall–floor joint. As such there would be limited anchorage to the bar and low pull-out resistance.

Figure G6 shows correctly installed reinforcement in a cross wall–floor joint, where the looped reinforcement extending from the ends of opposing floor slabs are located over a vertical dowel bar projecting from the top of the underlying cross-wall. This squat dowel bar provides mechanical coupling for the looped bars.

The presence of significant voids within the spine wall/floor slab joints did not appear to have had any visible affect upon the behaviour of the surrounding structural components under normal downward/lateral loading (i.e., dead, wind loads, etc.). The voids did, however, prevent the safe transfer of the existing load within the kitchen–lounge spine wall from above the test area into the steel shoring system that had been installed adjacent the spine wall as part of the element removal test. This structure, which comprised SGB soldiers, flat jacks and steel RHS box needles, was intended to act as a safety structure and to carry the weight of the structure above the

G3 INVESTIGATION TO ASSESS QUALITY OF CONSTRUCTION OF THE LPS DWELLING BLOCK

The quality and consistency of workmanship within the structural joints examined by BRE were, in the main, found to be reasonably good. Investigations undertaken as a result of localised failures of the kitchen and lounge floor slabs, which occurred during one of the load tests, indicated that the quality of workmanship achieved
kitchen–lounge spine test wall while the primary (in-wall) hydraulic jacking system was installed.

In summary, the quality of construction observed in the Liverpool Bison Wallframe dwelling block appeared to broadly comparable with that generally observed in other Bison Wallframe blocks and, more specifically, with that of the Sandwell Bison Wallframe load tested by BRE in February 1997.

G4  OVERPRESSURE TESTS: COMBINED WALL AND FLOOR OVERPRESSURE TESTS

Two combined wall and floor load tests were undertaken in a flank wall flat situated two storeys down (ie 14th storey) from the roof of the 16-storey block (Figures G2 and G3). This location was chosen as it represented the most critical location in terms of the response of the building to an internal gas explosion. Thus this was the location in the building considered to be most vulnerable to the effects of an internal gas explosion. Accordingly, other locations in the building would be expected to be more resilient to the effects of an internal gas explosion (ie they should perform better, all other factors being equal).

The combined wall and floor load tests were undertaken at different locations to those which had been subjected to the detailed physical investigations to establish the nature of construction of the Liverpool LPS dwelling block and to assess the achieved quality/workmanship (see section G3).

The combined wall and floor load tests were undertaken in the lounge of the flank wall flat as shown in Figure G3, and had the following dimensions. The test floor had a clear span of 3.82 m between the flank wall and the opposing cross-wall and an overall width of 4.52 m. The test room floor and ceiling slabs each comprised two sets of precast concrete floor slabs. The floor to ceiling height of the test room was 2.45 m. The flank wall of the test room was formed from two precast concrete wall panels, but the cross-wall between the lounge and the bedroom was formed from a single precast concrete wall panel.

Outside the test flat on the main elevation of the LPS dwelling block there was a balcony zone. This was formed by a separate concrete slab spanning between the flank wall and the adjacent cross-wall. To avoid cold-bridging an expanded polystyrene insulated joint was incorporated between the balcony floor slab and the adjacent test room floor slab, being situated on the line of the inside face of the lightweight glazed and timber framed panel forming the elevation wall. This joint made the balcony and test room floor slabs structurally independent. The flank wall and the cross wall extended beyond the test room to support the balcony floor slab. Cold-bridging into the dwelling was avoided by the application of expanded polystyrene insulation beneath a rendered finish on the outer face of these wall panels. This joint created a structural discontinuity between the floor/ceiling slabs within the dwelling and the adjacent balcony floor slab. In these circumstances notionally no load sharing occurred between the floor/ceiling slabs within the dwelling and the adjacent balcony floor slab.

Figure G7 shows schematically the nature of the loading created by an internal gas explosion, which acts as an overpressure applied simultaneously to the walls and floors of the enclosure bounding the site of the explosion. Figure G8 presents a schematic diagram indicating where the test loads were applied simultaneously to the walls and floors as bending moments and shear loads. The pattern of loading seeks to approximate the maximum bending moments and shear loads that might be produced by the uniformly distributed overpressure loading associated with an internal gas explosion.

Figure G9 shows a schematic of the hydraulic loading rig used to apply test loads to the wall and floor slabs in a single room, with the set-up shown being for the application of test loading to two adjacent pairs of wall panels. Finally, Figure G10 shows the general form of the room loading rig for the simultaneous application of the test loads to the wall and floor/ceiling slabs within a single room.
The first test was conducted with all the mechanical connections within the cross and flank wall / floor panel joints intact. The types of mechanical connections in question are shown in Figures G5 and G6.

The loading and instrumentation systems were identical to those used for the previous overpressure load tests undertaken in the Sandwell Bison Wallframe and the Leeds Reema Conclad LPS dwelling blocks (see section E4).

Figure G11 shows a schematic representation of the location of the movement transducers used to monitor the behaviour of key structural components during the combined wall and floor overpressure load tests (room overpressure load tests). A total of 18 channels were used to monitor vertical deflections during the tests and a further 21 channels were used to monitor horizontal deflections. All the deflections were monitored relative to a reference frame mounted on the 13th floor, one floor below the test room. A plan and a vertical section showing the location of the test room are presented as Figures G3 and G2, respectively.

Following the successful completion of the first load test cycle (Test 1), two of the looped tie connections within the cross wall–floor panel joint were cut. The type of mechanical connection in question is shown in Figure G6. The available drawings indicated that four of this type of mechanical connection should have been...
present within the cross wall–floor panel joint tested (three to the floor panel adjacent the balcony and one to the floor panel adjacent the kitchen). In the second load test cycle (Test 2), the floor and wall test loads were reapplied up to the maximum achieved during the first load test cycle (Test 1).

The aim of this aspect of the combined wall and floor overpressure load tests was to replicate the situation where the projecting floor loops had not been correctly placed over the associated wall dowels (refer Figure G6) and to see if this made an appreciable difference to the ultimate load behaviour.

Both tests demonstrated that the wall and floor panels were able to accommodate the forces associated with the specified overpressure loading criterion of 17 kN/m² without gross distortion of the panels and accompanying joints. A considerable amount of cracking developed in both the wall and floor slabs during the overpressure loading test. The applied load levels were taken to what was judged to be close to the ultimate load/structural failure condition.

There was little discernable change in behaviour when all the mechanical connections were intact during the first load test cycle (Test 1) and subsequently during the second load test cycle (Test 2), when two of the mechanical connections in the cross wall–floor panel joint were cut.

Figures G12 to G15 illustrate the response of the floor slabs and wall panels in the test room to the combined wall and floor overpressure test loads (Tests 1 and 2). Figures G12 and G13 show the measured responses for the first load test cycle (Test 1); with the floor slab response being shown in Figure G12 and that for the wall panels in Figure G13, respectively.

For Test 1 the mid-span of the floor slabs exhibited maximum load displacements of about 27 mm (for the floor slab which was subject to downward movement) and about 33 mm (for the ceiling slab which was subject to upward movement). After Test 1 the mid-span of the floor slabs exhibited residual displacements of about 15 mm (downward for the floor slab which was subject to downward movement) and 9 mm (upward for the ceiling slab which was subject to upward movement). The wall panels exhibited maximum measured lateral displacements of about 2 mm and residual lateral displacements of less than 1 mm.

Similarly Figures G14 and G15 show the measured responses for the second load test cycle (Test 2); with the floor slab response being shown in Figure G14 and that for the wall panels in Figure G15, respectively. The displacement transducers were not re-set (zeroed) between the first load test cycle (Test 1) and the second load test cycle (Test 2). Thus the total cumulative movements are shown in Figures G14 and G15.

For Test 2 the mid-span of the floor slabs exhibited maximum load displacements of about 33 mm (for the floor slab which was subject to downward movement) and about 41 mm (for the ceiling slab which was subject to upward movement). The response of the floor slabs during Test 2 was stiffer than recorded during Test 1, as indicated in Table G1.

On the completion of Test 2 the mid-span of the floor slabs exhibited residual displacements of about 25 mm (downward for the floor slab which was subject to downward movement) and 27 mm (upward for the ceiling slab which was subject to upward movement). The wall panels exhibited maximum measured lateral displacements of about 2 mm and residual lateral displacements of less than 1 mm.
Figure G13: Liverpool Bison Wallframe combined wall and floor test: wall panel displacements: Test 1

Figure G14: Liverpool Bison Wallframe combined wall and floor test: floor panel displacements: Test 2
Note: The displacement transducers were not reset to zero between Test 1 and Test 2; accordingly the values presented for Test 2 commence from the recovery position after Test 1 and are therefore cumulative values including any ‘permanent’ set.

The outcome of the Liverpool combined wall and floor panel tests were broadly consistent with those obtained from nominally identical tests undertaken in the Sandwell Bison Wallframe block as described in Appendix E (albeit that the floor spans were different in the two blocks tested). Taken together this suggests that, unless there are gross deficiencies in the construction, Bison Wallframe LPS dwelling blocks are generally likely to be sufficiently strong to accommodate the overpressure forces associated with a ‘severe’ non-piped gas explosion (or less onerous explosion).

65 ABILITY TO MOBILISE ALTERNATIVE LOAD PATHS: SPINE WALL TEST
The element removal test was aimed at exploring the behaviour of the block to the simulated loss of a particular...
steel channels, which in turn are meant to be welded to the tension reinforcement embedded in the two floor slabs. Figure G16 illustrates a correctly formed version of this detail.

To break the existing load path in the spine wall, it was decided to remove the top of the spine wall at first floor level, thereby forcing the existing load in the spine wall into an alternative path. This transfer was carried out slowly by means of a hydraulic jack/loading system. It was not practicable to do this transfer rapidly, as would be the case in a gas explosion, for a number of reasons. Firstly, control of the test had to be maintained at all time to ensure the safety of the test team and those who were to subsequently demolish the block after testing was completed. Secondly, the financial resources available for the project were limited. Thirdly, there was the difficulty that explosions cannot necessarily be closely controlled and the extent of the damage that might have been caused could have exceeded that desired. This could potentially have caused unforeseen disturbance to the existing structure, which could have affected the development of alternative load paths.

Table G1: Overview of floor slab mid-span deflections and recovery values for two room overpressure tests (Test 1 and Test 2)

<table>
<thead>
<tr>
<th>Test no. and loading direction</th>
<th>Max mid-span displacement (mm)</th>
<th>Recovery upon unloading (mm)</th>
<th>% Recovery upon unloading</th>
<th>Span/max mid-span displacement (Ratio)</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1 – Up</td>
<td>32.6</td>
<td>23.7</td>
<td>73%</td>
<td>117</td>
</tr>
<tr>
<td>T1 – Down</td>
<td>27.4</td>
<td>12.9</td>
<td>47%</td>
<td>139</td>
</tr>
<tr>
<td>T2 – Up</td>
<td>27.2</td>
<td>13.4</td>
<td>49%</td>
<td>140</td>
</tr>
<tr>
<td>T2 – Down</td>
<td>16.6</td>
<td>8.0</td>
<td>48%</td>
<td>230</td>
</tr>
</tbody>
</table>

The kitchen–lounge spine wall within the flank wall flats in the Liverpool Bison Wallframe block, forms the separating wall between the kitchens and lounges. It supports one end of the kitchen floor slab above. Each kitchen floor slab spans orthogonally to the adjacent lounge floors, which are supported by the cross and flank walls. The kitchen and lounge floor slabs are intended to be tied together at one location within the width of the interconnecting door way. A typical tie normally comprises a relatively short square steel bar welded to the load bearing wall element. The wall selected for ‘removal’ was located between the kitchen and lounge in the flank wall flats. The test location was situated at first floor level, with fourteen floors of the building above (Figures G2 and G4). The test was included in the programme to enable the modelling/calibration of a different form of behaviour to that examined during the earlier set of load tests. The test also addressed existing concerns over the potential behaviour of an LPS dwelling block in the event of the loss of this type of wall during an explosion. Such a loss, potentially resulting from either base shear failure and/or flexural failure of the kitchen–lounge spine wall, could occur during a ‘severe’ gas explosion.

Figure G15: Liverpool Bison Wallframe combined wall and floor test: wall panel displacements: Test 2

Note: The displacement transducers were not reset to zero between Test 1 and Test 2; accordingly the values presented for Test 2 commence from the recovery position after Test 1 and are therefore cumulative values including any ‘permanent’ set.
It was also uncertain how the kitchen–lounge spine wall might perform when subject to overpressure in the event of a significant gas explosion. There was a question as to the nature of the damage which might occur. For example, would the wall crack at mid-height, but remain in place by virtue of compression membrane/arching effects, or be completely displaced/destroyed by the applied lateral loading. It was hypothesised that failure was likely to involve a combination of flexural and base shear failure. So in order to gain some understanding of possible whole building behaviour it was necessary to simplify the test and event scenario.

It is relevant to note that strain rate effects, which are expected to occur during a gas explosion, could also act to increase the chance that the kitchen–lounge spine wall would survive such an explosion. This behaviour is discussed in section E3 in Appendix E.

The kitchen–lounge spine wall test set-up consisted of several component parts. A load bearing safety structure (Figures G17 and G18) comprising twinned SGB steel soldiers, steel box (RHS) ‘needles’ and flat jacks was installed adjacent the kitchen–lounge spine wall. This structural system was designed to pick-up the existing load in the kitchen–lounge spine wall at second floor level, so it could safely carry this load in the event of failure of the primary (in-wall) hydraulic jacking system or related parts of the kitchen–lounge spine wall. The system also had the function of minimising disturbance to the adjacent structural elements/load paths prior to the start of the element removal test. But primarily it was to allow the
existing load in the kitchen–lounge spine wall to be safely transferred into the primary (in-wall) jacking system once that had been safely installed. The latter system consisted of a series of long throw hydraulic jacks and steel bearing plates positioned in narrow cut-outs formed in the top of the kitchen–lounge spine test wall.

An instrumentation system similar to that used during the combined wall and floor tests (see section G4) was installed within the kitchen to monitor the behaviour of the surrounding structure.

Once the safety and instrumentation systems had been installed and the load taken into the safety structure, the top 100 mm or so of the kitchen–lounge spine wall was gradually broken away using hand held concrete breakers.

Measurements of vertical displacements of the kitchen floor slab showed that the structure above the spine wall under test moved downwards only a fraction of a millimetre once the support provided by the top of the kitchen–lounge spine wall panel had been removed and the primary and safety system hydraulic jacks retracted.

The test clearly demonstrated that the section of the building situated above a damaged or ‘missing’ kitchen–lounge spine wall could potentially develop alternative load paths. Such a mechanism should enable the structure to bridge over areas of damage which might occur during a gas explosion within the lounge or kitchen of the flank wall flats in certain types of Bison Wallframe dwelling blocks which had caused severe damage to, or the failure of, a kitchen–lounge spine wall.

On the basis of the observed behaviour, a Bison Wallframe dwelling block would not be expected to suffer progressive collapse or disproportionate damage in the event of the loss of a spine wall panel between the kitchen and lounge in a flank wall flat as a result of a non-piped gas explosion.

Plates 3–6 illustrate the form of the finite element models developed by BRE together with the predicted response of the wall and floor elements bordering the kitchen–lounge spine wall in these circumstances.

Once this had been achieved the full load of the structure above to be taken by the primary (in-wall) hydraulic jacking system, the load transfer process associated with element removal operation could then be instigated.

The rams of the hydraulic jacks forming the primary (in-wall) load-carrying system (Figure G18) were then lowered in stages to enable the behaviour of the section of the building above to be monitored.

Measurements of vertical displacements of the kitchen floor slab showed that the structure above the spine wall under test moved downwards only a fraction of a millimetre once the support provided by the top of the kitchen–lounge spine wall panel had been removed and the primary and safety system hydraulic jacks retracted.

The test clearly demonstrated that the section of the building situated above a damaged or ‘missing’ kitchen–lounge spine wall could potentially develop alternative load paths. Such a mechanism should enable the structure to bridge over areas of damage which might occur during a gas explosion within the lounge or kitchen of the flank wall flats in certain types of Bison Wallframe dwelling blocks which had caused severe damage to, or the failure of, a kitchen–lounge spine wall.

On the basis of the observed behaviour, a Bison Wallframe dwelling block would not be expected to suffer progressive collapse or disproportionate damage in the event of the loss of a spine wall panel between the kitchen and lounge in a flank wall flat as a result of a non-piped gas explosion.

Plates 3–6 illustrate the form of the finite element models developed by BRE together with the predicted response of the wall and floor elements bordering the kitchen–lounge spine wall in these circumstances.

Figure G18: Close-up of primary jack (two jacks each side of wall) and safety loading shore (left) arrangement prior to removal of head of spine wall
OVERVIEW OF FINITE ELEMENT ANALYSES AND
CALIBRATION EXERCISES FOR THE LIVERPOOL
BISON WALLFRAME LPS DWELLING BLOCK
TESTED BY BRE

H1 INTRODUCTION

The numerical analysis work was carried out using the commercially available finite element program, DIANA 9, developed by TNO, Holland. DIANA was used to undertake finite element modelling of selected areas of the Liverpool Bison Wallframe LPS dwelling block prior to carrying out the load testing and element removal operation activities. The information obtained from these exercises provided guidance upon the likely behaviour of key elements within the test block and gave insight into potential mode(s) of failure.

The linear and non-linear finite element analysis (FEA) modelling and associated calibration exercises were also used to explore the relative influence of varying the nature of the models employed and the associated input parameters. This gave some understanding of the sensitivity of the FEA models to variations in the input data. Both linear and non-linear FEA methods were used to model the lounge floor slab tested in a flank wall flat during the simulated room overpressure test to failure (see section G4 in Appendix G). Finite element modelling (FEM) undertaken for the kitchen–lounge spine wall element removal operation (see section G5 in Appendix G) was restricted to a linear FEA because of the number of finite elements/degrees of freedom involved.

These activities allowed some degree of ‘calibration’ of the finite element models against the experimental results. The intention was not to find the most accurate model representation per se, but rather to develop sufficiently accurate models within the limitations imposed by the available computing resources, time and funding constraints. Thus this activity involved a sensitivity analysis of a number of the variables influencing the FEA modelling outcomes.

DIANA 9 provided a large number of finite element options and, when decisions were made on matters such as element type and mesh density, consideration was given to numerous factors. These factors included the required numerical accuracy, the available computing resources and the specific characteristics of the problem being modelled; such as its dimensions, composition, materials involved, loading and the potential degree of non-linearity in the behaviour of the structure (for the flank wall flat lounge floor model).

H2 MODELLING OF LPS DWELLING BLOCK
RESPONSE TO THE REMOVAL OF A
KITCHEN–LOUNGE SPINE WALL ELEMENT

A number of different finite element models were employed to investigate the overall behaviour of the Liverpool Bison Wallframe LPS dwelling block before and after the kitchen–lounge spine wall element removal operation; and the behaviour of the structural elements local to the site of the spine wall element removal in particular. These investigations considered the behaviour of the structure above and below the first floor kitchen–lounge spine wall element removal location. The lower floors were modelled in most detail. For example, one detailed FEA model examined the ground, first and second floor storeys of the building, with associated dead loads. Another detailed FEA model replicated the LPS dwelling block structure from ground to fourth floor level, taking into account the influence of the dead loads arising from the remainder of the LPS dwelling block above. Other less detailed models considered a larger proportion of the overall LPS dwelling block.

Some 13 detailed FEM runs were carried out to explore the influence of concrete strength, finite element type, Young’s modulus, joint element and connection type upon the predicted behaviour of the LPS dwelling block structure.

Table H1 presents a summary of the range of material properties used and assumptions made during the various behaviour investigation and calibration exercises.

The linear finite element analyses were undertaken prior to the actual kitchen – lounge spine wall element removal operation to help inform the judgement which had to be made about the likely behaviour of the

<table>
<thead>
<tr>
<th>Table H1: The range of material properties used and assumptions made – modelling of removal of kitchen – lounge spine wall element</th>
</tr>
</thead>
<tbody>
<tr>
<td>Property</td>
</tr>
<tr>
<td>Characteristic compressive strength of wall/floor/joint concrete</td>
</tr>
<tr>
<td>Young’s modulus of floor/wall elements</td>
</tr>
<tr>
<td>Floor elements</td>
</tr>
<tr>
<td>Wall elements</td>
</tr>
<tr>
<td>Horizontal and vertical joints</td>
</tr>
</tbody>
</table>
structure in these circumstances and whether the test could be conducted safely. In particular this concerned whether the building would be able and likely to develop alternative load paths for the 14 storeys of building above the test location, and hence bridge over the missing kitchen – lounge spine wall, or suffer local failure and potential progressive collapse. The judgement was based upon a number of factors including the estimated stresses, deflection and potential overstress of structural elements.

The results of the modelling runs (Plates 3 to 6) suggested that overstress of structural elements would not be excessive and that the downward vertical displacement of the part of the kitchen–lounge spine wall above the element removal location was likely to be less than 1 mm. The modelling employed a number of different element configurations, which sought to replicate expected variations in element joint behaviours. The behaviour of the of the kitchen–lounge spine wall and associated parts of the LPS dwelling block above the element removal location appeared to be consistent, regardless of the variations in the choice of joint shells or their respective input parameters.

Accordingly, the linear finite element analyses suggested that the LPS dwelling block structure should be able to bridge over the ‘missing’ kitchen–lounge spine wall based upon the magnitude of the predicted displacements and the estimated stresses. This result gave confidence to proceed with the test.

Plate 3 illustrates the behaviour of a five storey section of the LPS dwelling block with the kitchen–lounge spine wall present, showing the contour plots of the estimated vertical deflection of the structure under dead loading. The largest deflections are associated with the downward deflection of the lounge floor slab under dead load, occurring in the middle of the slabs. Plate 4 illustrates the behaviour of a five storey section of the LPS dwelling block with the kitchen–lounge spine wall absent. Thus Plates 3 and 4 provide an overview of the predicted overall response of the section of the LPS dwelling block being considered. This work was carried out as part of the first phase of the linear FEM.

Plates 5 and 6 provide more detail of the predicted behaviour, being produced in a second phase of the FEM which concentrated upon the lower three storey section of the LPS dwelling block. Plate 5 shows the expected behaviour with the kitchen–lounge spine wall present and Plate 6 shows the anticipated behaviour with it absent, with both figures showing the estimated vertical deflection of the structure under dead loading.

H3 MODELLING THE BEHAVIOUR OF THE FLANK WALL FLAT LOUNGE FLOOR SLABS UNDER OVERPRESSURE AND OTHER FORMS OF LOADING

Both linear and non-linear finite element methods were used to model the behaviour of the lounge floor slab tested in a flank wall flat during the simulated room overpressure test to failure in the Bison LPS dwelling block (see section G4 in Appendix G).

It was concluded that the most appropriate option for modelling non-linear behaviour of the lounge floor slabs was to use either ‘shell’ or ‘solid’ elements. A layered 8-node quadrilateral isoparametric curved shell element was adopted for the ‘shell’ model and a 20-node isoparametric brick element for the ‘solid’ model. It was decided not to build finite element models using linearly interpolated elements as these were considered to be less suitable for non-linear analyses.

Whilst ‘solid’ elements are the most appropriate type for accurately modelling the complex geometry of the floor slabs, the use of this type of element produces large systems of equations which require appreciable computer processing power for their solution. Due to limitations on the available computing resources it was decided to use ‘solid’ elements for only a limited number of analysis runs. This way of working allowed a comparison to be made between the performance of finite element models constructed using ‘shell’ elements and those constructed with ‘solid’ elements.

Plates 7 to 9 illustrate various aspects of the general form of construction of the finite element model of the flank wall flat lounge floor slabs, which includes modelling of the reinforcement provision and of the horizontal void distribution within the precast concrete floor slab components.

It should be noted that ‘layered’ shell elements, as opposed to a ‘normal’ shell elements, were employed as the former type can be modified so that they are better able to describe the ultimate behaviour of the reinforced concrete components.

The reinforcing bars were modelled as embedded, ie the reinforcing elements lie within the boundaries of the basic elements. Both smeared and discrete reinforcement representation methods were employed. The discrete bars were positioned in accordance with BRE’s understanding of the layout of the lounge floor reinforcement established through both invasive and non-invasive investigations undertaken previously by BRE.

A number of refinements were made to some of the FEM runs in order to maximise the accuracy of the ‘shell’ model and to improve the efficiency of the computations.

The most likely non-linear effects upon the behaviour of the reinforced concrete floor were taken into consideration when building the finite element models. The effects considered include multi-axial compressive response, tensile cracking of concrete and yielding of the steel reinforcement. However, the precise choice of yield criterion was found to be of minor importance, as the numerically predicted response of the lounge floor slab was found to be dominated by tensile cracking of the concrete.

Table H2 presents a summary of the range of material properties used and assumptions made, during the calibration exercise.

BRE carried out over 50 FEM runs using two element types and a range of input parameters associated with representative material properties and behaviours. Clearly, it would have been impractical to explore the effects of all possible combinations of input parameters for the two selected element types upon the predicted structural response of the lounge floor slab. It was, therefore, necessary to use engineering judgment to select the most
MPa = MegaPascal = 1 N/mm²

high applied load intensity, respectively. Plates 14 and present under low applied load intensity and under slabs with the non-load bearing half-glazed partition orientation of cracking in the soffit of the lounge floor timber partition to the balcony is absent.

the floor slabs when the (external elevation wall) glazed flat lounge floor slab. Plate 10 shows the predicted predicted by the numerical analysis for the flank wall BRE. Plate 10 shows the vertical displacement pattern of the finite element modelling runs undertaken by some of the calibration runs.

construction was introduced into the finite element model idealised and simplified version of the glazed partition associated with an internal gas explosion. To do this an lounge floor slab under accidental overpressure loading if any, such a notionally non-load bearing glazed timber door. It was therefore important to explore what effect, the adjoining balcony was gained via a fully glazed timber (in which the room overpressure load test was carried (outside elevation wall) flat lounge floor slab. Plate 11 shows the predicted distribution and orientation of cracking under the same two applied load intensities, but with the non-load bearing half-glazed partition absent.

Figures H1 to H3 present a series of graphs which provide a comparison between the load/deflection behaviour of the flank wall flat lounge floor slab predicted by various FEM runs and that measured during the combined wall and floor overpressure load test. The deflections were measured at mid-span at either side of the joint between the two precast floor panels forming the lounge floor. The first load cycle is shown as a solid line and the second as a dashed line. This convention is applied to both test locations for which recorded deflection results are presented.

The finite element models developed for this programme of work utilised measured material properties where known. In cases where certain parameters were not known it was necessary to use engineering judgment to select the most appropriate initial input values, including those for material behaviour.

In Figures H1 to H3 the uniformly distributed overpressure load of 17 kN/m² is indicated on the load/ deflection graphs. This has simply been calculated on the basis of the total jack loads divided by the total area of the floor slabs being loaded. The FEM results presented in Figure H1 suggests that this provides a reasonable first order approximation of the equivalence to the between the patch loading applied during the load test and the uniformly distributed overpressure associated with an internal gas explosion.

Figure H1 examines the effect of load distribution upon slab deflections and on the load/deflection behaviour of the flank wall flat lounge floor slab. Two types of loading are considered; these being the patch loading applied during the combined wall and floor test load (see section G4 in Appendix G) and the uniform overpressure loading created by an internal gas explosion. The analysis results indicate that the load/ deflection behaviour is similar for the two types of loading considered. This in turn suggests that the test loading regime provides a reasonable representation of the forces associated with the overpressure loading created by an internal gas explosion. The FEM runs achieve a good agreement with the behaviour measured during the combined wall and floor overpressure load test in the vicinity of the maximum load, which was judged to be close to the ultimate/failure load for the flank wall flat lounge floor slab.

Figure H2 examines the effect of support conditions upon slab deflections and on the load/deflection behaviour of the flank wall flat lounge floor slab. This demonstrates the potential influence on the stiffness and load capacity of the flank wall flat lounge floor slab which could arise from the presence of a glazed timber partition separating the lounge from the balcony. The glazed timber partition is notionally non-loadbearing (ie gravity loads). However, a strong partition, if present, could significantly increase the stiffness and load-carrying capacity of the flank wall flat lounge floor slab subject to accidental loads. This result indicates the potential wider importance of internal or external elevation non-loadbearing partition

<table>
<thead>
<tr>
<th>Material properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Characteristic compressive strength of concrete</td>
<td>30 MPa</td>
</tr>
<tr>
<td>Young's modulus</td>
<td>31.8 GPa</td>
</tr>
<tr>
<td>Compressive failure strain</td>
<td>3.5 mm/m (0.0035)</td>
</tr>
<tr>
<td>Tensile strength of concrete</td>
<td>2–4 MPa</td>
</tr>
<tr>
<td>Tensile ultimate strain</td>
<td>0.002</td>
</tr>
<tr>
<td>Shear retention factor</td>
<td>0.2–1.0</td>
</tr>
<tr>
<td>Reinforcement yield stress</td>
<td>355–470 MPa</td>
</tr>
</tbody>
</table>

Notes:
1 Estimated from strength test results from nine cores obtained from selected floor slabs.
2 Estimated from compressive strength using formulae given in CEB 95[81].
3 Maximum permitted compressive strain in analysis.
4 Limiting characteristic values according to CEB 95[81].
5 Ultimate strain for tensile resistance.
6 Constant shear resistance factor for closed cracks (simulates degree of aggregate interlocking).
7 Yield strength of the various reinforcing bars. Test results provided by Birmingham City Laboratories.

MPa = MegaPascal = 1 N/mm²
GPa = GigaPascal = 1000 N/mm² = 1 kN/mm²

appropriate input values and to adopt the most realistic assumptions with respect to material behaviour for the modelling runs undertaken.

Output from the initial finite element analyses were used to establish the changes required to the input parameters/assumptions in order to reduce the error between the measured and predicted structural behaviours. Appropriate adjustments were made to selected parameters and the finite element models were then re-run.

The external elevation wall of the flank wall flat lounge (in which the room overpressure load test was carried out) comprised timber studding faced with plywood with full width half-height glazed windows. The glazed timber partition separated the lounge from the balcony. Access to the adjoining balcony was gained via a fully glazed timber door. It was therefore important to explore what effect, if any, such a notionally non-load bearing glazed timber partition might have on the predicted behaviour of the lounge floor slab under accidental overpressure loading associated with an internal gas explosion. To do this an idealised and simplified version of the glazed partition construction was introduced into the finite element model for some of the calibration runs.

Plates 10 to 15 present graphically some of the results of the finite element modelling runs undertaken by BRE. Plate 10 shows the vertical displacement pattern predicted by the numerical analysis for the flank wall flat lounge floor slab. Plate 11 shows the predicted distribution of stress and orientation of cracking in the sofit of the lounge floor slabs with the non-load bearing half-glazed partition present under low applied load intensity and under high applied load intensity, respectively. Plates 14 and 15 illustrate the expected distribution and orientation of cracking under the same two applied load intensities, but with the non-load bearing half-glazed partition absent.

Figures H1 to H3 present a series of graphs which provide a comparison between the load/deflection behaviour of the flank wall flat lounge floor slab predicted by various FEM runs and that measured during the combined wall and floor overpressure load test. The deflections were measured at mid-span at either side of the joint between the two precast floor panels forming the lounge floor. The first load cycle is shown as a solid line and the second as a dashed line. This convention is applied to both test locations for which recorded deflection results are presented.

The finite element models developed for this programme of work utilised measured material properties where known. In cases where certain parameters were not known it was necessary to use engineering judgment to select the most appropriate initial input values, including those for material behaviour.

In Figures H1 to H3 the uniformly distributed overpressure load of 17 kN/m² is indicated on the load/ deflection graphs. This has simply been calculated on the basis of the total jack loads divided by the total area of the floor slabs being loaded. The FEM results presented in Figure H1 suggests that this provides a reasonable first order approximation of the equivalence to the between the patch loading applied during the load test and the uniformly distributed overpressure associated with an internal gas explosion.

Figure H1 examines the effect of load distribution upon slab deflections and on the load/deflection behaviour of the flank wall flat lounge floor slab. Two types of loading are considered; these being the patch loading applied during the combined wall and floor test load (see section G4 in Appendix G) and the uniform overpressure loading created by an internal gas explosion. The analysis results indicate that the load/ deflection behaviour is similar for the two types of loading considered. This in turn suggests that the test loading regime provides a reasonable representation of the forces associated with the overpressure loading created by an internal gas explosion. The FEM runs achieve a good agreement with the behaviour measured during the combined wall and floor overpressure load test in the vicinity of the maximum load, which was judged to be close to the ultimate/failure load for the flank wall flat lounge floor slab.

Figure H2 examines the effect of support conditions upon slab deflections and on the load/deflection behaviour of the flank wall flat lounge floor slab. This demonstrates the potential influence on the stiffness and load capacity of the flank wall flat lounge floor slab which could arise from the presence of a glazed timber partition separating the lounge from the balcony. The glazed timber partition is notionally non-loadbearing (ie gravity loads). However, a strong partition, if present, could significantly increase the stiffness and load-carrying capacity of the flank wall flat lounge floor slab subject to accidental loads. This result indicates the potential wider importance of internal or external elevation non-loadbearing partition
walls above and below the site of an explosion to enhance the ability of floor/ceiling slabs to accommodate the forces and loadings associated with the overpressure created by an internal gas explosion.

The FEM for the weak partition case achieved a good agreement with the behaviour measured during the combined wall and floor overpressure load test in the vicinity of the maximum load (see Figure H2). This suggests that the glazed timber partition separating the lounge from the balcony in the flank wall flat lounge (ie in the flats above and below the site of an explosion) was relatively weak (ie in terms of gravity loads) and would therefore have only had a modest influence upon the measured stiffness and load capacity of the flank wall flat lounge floor slab tested.
Figure H3: Effect of concrete tensile strength on the predicted maximum deflection at mid-span of the flank wall flat lounge floor slab – at the joint between the two floor slabs.

Figure H3 examines the effect of concrete tensile strength upon slab deflections and on the load/deflection behaviour of the flank wall flat lounge floor slab. The FEM results indicate that the stiffness and ultimate load capacity of the flank wall flat lounge floor slab should increase substantially as the tensile strength of the concrete increases from 2 MPa to 4 MPa. The FEM runs achieve a good agreement with the behaviour measured during the combined wall and floor overpressure load test in the vicinity of the maximum load when the tensile strength of the concrete is 3 MPa.

The data presented in Figures H1–H3 demonstrate that a non-linear finite element analysis, carried out with appropriate skill and care in association with a subsequent calibration exercise, can predict, with a reasonable degree of accuracy, the general behaviour of a lounge floor slab under uniformly distributed load and its ultimate load-carrying capacity.
APPENDIX I
STRENGTHENING OPTIONS

I1  INTRODUCTION
This Appendix presents summary details for a number of strengthening options which have either been employed or considered for use within LPS dwelling blocks. The principal advantages, limitations and circumstances of use for each strengthening method are briefly indicated. The details presented should be taken to be indicative, since they do not address all potential strengthening or remedial options or variations upon the basic approaches. The details presented also give an insight into most of the strengthening measures which may be encountered within an existing LPS dwelling block. Methods for the protection and repair of concrete structures, such as making interventions to enhance the durability of an LPS dwelling block are not included in this Appendix.

Strengthening options for components/panels:
- Bonded steel plates.
- Externally-bonded fibre reinforced polymer (FRP) composites.
- Supplementary bearing steelwork.
- Replacement loadbearing partitions.

Strengthening method – Dry pack replacement or reinstatement:
- Reinstatement of defective or missing dry packing at the base of wall panels.

Strengthening options involving supplementary tying or load bearing structural elements:
- Vertical tying.
- Horizontal tying.
- Cast in-situ internal concrete columns (flank wall).
- External steel frame (flank wall).
- Intermediate strong floors.

Strengthening options for joints between components/panels:
- Steel angles at base and/or head of wall panels.
- Resin bonded anchors.

I2  OVERVIEW OF SOME STRENGTHENING OPTIONS
Whilst in most instances there will be only a limited requirement for strengthening works to undertaken within the majority of LPS dwelling blocks within the UK, particular deficiencies in construction, material strength or degradation in tying provision (ie corrosion), identified in some LPS dwelling blocks may require the provision of particular forms of strengthening. The various principles and options for the key strengthening techniques are outlined in the following sections. Table I1 included below presents the advantages, limitations and circumstances for use for each strengthening method.

I3  VERTICAL EXTENT OF STRENGTHENING REQUIRED
It will generally be necessary to achieve a balance between the extent of strengthening works undertaken in an LPS dwelling block; their cost and the disruption to residents that is associated with major works of this type. In this process consideration also has to be given to the perceived marginal improvement in safety that would be achieved through the proposed works; providing that the overall level of safety being achieved is ‘adequate’.

The full-scale load tests undertaken by BRE in a Reema Conclad LPS dwelling block and two Bison Wallframe LPS dwelling blocks have demonstrated that in a ‘standard’ LPS dwelling block, that the wall and floor panels situated one storey down from the top storey (and below) are likely to be able to survive a ‘severe’ or less onerous accidental non-piped gas explosion, albeit in a damaged state. It is expected that such internal non-piped gas explosions are unlikely to generate overpressures exceeding 17 kN/m². In addition, strain rate effects and venting through failure of windows and/or internal partitions, which typically reduces the maximum overpressure developed, are expected to enhance the chances of an LPS dwelling block surviving such an explosion.

In LPS dwelling blocks selective strengthening works are normally restricted to the components and joints in those parts of the block perceived to be at the greatest risk of damage or disruption from an accidental internal gas explosion. In LPS dwelling blocks without a piped gas supply, load bearing wall panels in the upper five storeys are commonly perceived to be at greatest risk and therefore these are therefore typically the focus for assessing the potential need for strengthening works. Floor slabs throughout an LPS dwelling block are notionally exposed to the same probability of an internal gas explosion and it is the longer spanning floor slabs, such as

---
47 A block in which nominally identical wall panels are situated directly above one another as opposed to being laterally staggered on successive floor levels.
Table II: Overview of a number of potential strengthening options

<table>
<thead>
<tr>
<th>Option (Section ref.)</th>
<th>Circumstances for use</th>
<th>Advantages*</th>
<th>Limitations/Issues to consider*</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Strengthening options for components/panels</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bonded steel plate (I4.2.1)</td>
<td>Generally applied to the soffit or top of floor slabs to enhance flexural and shear capacity.</td>
<td>Fairly quick installation time – no drilling required, low profile so minimal intrusion into room.</td>
<td>Requires sound substrate, limited ability to enhance shear capacity of section, poor behaviour of adhesives in fire so fire protection required, cosmetic surface cover required when applied to floor soffits, potential long term durability issues with adhesive. Requires partial removal of screed to expose floor slab. Cannot be used to enhance flexural capacity of flank wall panels as access cannot be gained to outer face of inner leaf. Disruption to residents.</td>
</tr>
<tr>
<td>Bonded FRP composites (I4.2.2)</td>
<td>Applied to the top and/or bottom of floor slabs to enhance flexural (tensile) capacity.</td>
<td>Fairly quick installation time – no drilling required, low profile so minimal intrusion into room.</td>
<td>Requires sound substrate, unable to enhance shear capacity of section, poor behaviour of FRP and adhesive in fire so fire protection required, cosmetic surface cover required when applied to floor soffits. Susceptible to impact damage and fire if left exposed. Requires partial removal of screed to expose floor slab. Cannot be used to enhance flexural capacity of flank wall panels as access cannot be gained to outer face of inner leaf. Disruption to residents during installation.</td>
</tr>
<tr>
<td><strong>Additional bearing steelwork</strong> (I4.2.3)</td>
<td>To enhance the bearing of slabs where deficiencies have been identified.</td>
<td>Quick installation, fixing bolts can be proof tested.</td>
<td>Fire protection required/boxing-in usually needed to improve visual appearance. Potential durability issues where steelwork is installed in areas of high humidity. Limited disruption to residents.</td>
</tr>
<tr>
<td>Replacement loadbearing partitions (I4.2.4)</td>
<td>Installation of load bearing partitions to enhance load-carrying capacity of ceiling slabs.</td>
<td>Replacement walls can have the same ‘foot print’; as existing partitions. Can be designed to provide vertical tying at certain locations within a block.</td>
<td>Limited opportunities to apply this method in most LPS dwelling blocks due to layout of rooms. Consideration needs to be given to implications for floor slabs and existing reinforcement provision. Works are disruptive to residents.</td>
</tr>
<tr>
<td><strong>Strengthening method – dry pack replacement</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reinstate defective or missing dry packing (I5)</td>
<td>To reinstate defective or missing dry pack at base of load bearing wall panels.</td>
<td>Replacement can be done in stages, quality checks by coring.</td>
<td>Disruption to residents. Ideally requires access to both sides of wall to ensure good degree of compaction. Problems may arise if electrical cabling/services are located in screed/face of joint at base of wall panels.</td>
</tr>
<tr>
<td><strong>Strengthening options involving supplementary tying or load bearing structural elements</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Install or enhance vertical tying (I6.1.1) (I6.2.1)</td>
<td>Vertical strapping enables block to develop alternative load paths in the event of the loss of a key element.</td>
<td>Provides clear/established mechanism for achieving structural continuity</td>
<td>Can be unsightly/need to box-in. Installation results in significant disruption to residents. Has been used to enhance effective dead load in upper parts of a block to resist uplift forces present in a gas explosion. Storeys can also ‘hang’ above location of a damaged wall panel. Consider in conjunction with horizontal tying.</td>
</tr>
<tr>
<td>Install or enhance horizontal tying (I6.1.2) (I6.2.2)</td>
<td>Horizontal straps provide supplementary horizontal tying where original provision is deficient/absent.</td>
<td>Provides clear/established mechanism for achieving structural continuity</td>
<td>Significant disruption to residents during works. There can be noticeable local reductions in floor to ceiling height where a supplementary beam is introduced below the floor slab soffit. However, where straps are used that can be concealed within screed thickness (assuming a screed is present) there will be no loss in room height. Consider in conjunction with vertical tying.</td>
</tr>
<tr>
<td>In-situ concrete columns (I7.2.1)</td>
<td>Provides building with opportunity of developing alternative load paths.</td>
<td>None perceived.</td>
<td>Significant disruption to residents during formation of columns, problems associated with transporting and casting wet concrete within building, care need in providing sufficient tying between column and wall and floor panels.</td>
</tr>
</tbody>
</table>

* Costs of remedial options have not been considered in this summary.
the lounge floor slabs, that are commonly perceived to be at greatest risk. These therefore are usually the focus for any potential strengthening works and, of course, occur at all levels throughout the building.

However, it must be accepted that where an explosion occurs in one of the rooms on the top storey of a block, there is a chance that the wall and roof panels forming the top storey flat will not survive the explosion intact. This is primarily due to the relatively low dead loads that are developed at the base and head of the wall panels at this storey level, and in particular in the flank wall panels, which control the likelihood of a sliding failure in the loadbearing wall panels. In these circumstances the wall and roof panels in top storey flats might be disrupted and potentially collapse during a ‘severe’ internal gas explosion, perhaps in the manner portrayed in Figures B6 and B7 (see Appendix B). In this case the damage to the six-storey LPS dwelling block at Hulme, Manchester was caused by a ‘very severe’ piped gas explosion. Figures B6 and B7 show the wall and roof panels that, following the internal gas explosion, collapsed onto the floor panels of the top storey flat concerned. The floor slab successfully withstood the associated impact. On this basis, the wall and roof panels of top storey flats might be regarded as being sacrificial, with the potential gas explosion damage considered as likely to be limited to the single top storey flat concerned, albeit that there is the possibility of damage to the adjacent top storey flats. It may, of course, be considered prudent to provide limited strengthening in a top floor flat. This extent of damage is expected to be considered acceptable within the requirements of the Building Regulations, as applied to a new build situation.

Nevertheless, the implications of the loss of wall and roof panels of a top floor flat in which a non-piped gas explosion may occur needs to be considered. The debris loading on the floor slabs below arising from the collapse of the roof and adjacent wall panels is expected to be significant. The floor slab needs to be able to survive the associated impact(s), albeit in a damaged state. There is also the possibility that if a gas explosion occurs in a top floor flat then it must be accepted that there will be a risk to the public and users of the building arising from any roof and/or wall panels that may become detached from the building.

However, BRE consider that generally there may be limited merit in undertaking extensive strengthening works to the wall and roof panels on the top storey of LPS dwelling blocks. This is due to the nature and extent of strengthening that are likely to be required to prevent any of the top wall and roof storey panels from becoming detached from the building. The decision would be based upon the degree of assurance required and the potential costs versus the benefits which may be gained. Appendix C, which is concerned with risk issues, discusses related matters and outlines some aspects of cost benefit assessment.

<table>
<thead>
<tr>
<th>Option (Section ref.)</th>
<th>Circumstances for use</th>
<th>Advantages*</th>
<th>Limitations/Issues to consider*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Externalseel frame (flank wall) I7.2.2</td>
<td>Provides external bracing to flank walls.</td>
<td>All works external, limited disruption to residents.</td>
<td>Need to provide sufficient protection against corrosion, possibly also fire. Acceptable external visual appearance of block needs to be considered/maintained.</td>
</tr>
<tr>
<td>Intermediate strong floors</td>
<td>Divides block in attempt to limit vertical damage within block.</td>
<td>Theoretically could be used to eliminate need to strengthen all floors.</td>
<td>Approach unlikely to be practical. Consideration needs to be given to the load capacity/strength of supporting walls. Results in low head room, significant difficulty in transporting and positioning steelwork in block. Careful design of all aspects required.</td>
</tr>
<tr>
<td>Provision of steel angles at base and/or head of wall panels I8</td>
<td>Addition of steel angles to increase sliding resistance at base and/or head of wall panels, plates to tie adjacent panels together.</td>
<td>Relatively simple installation procedure, fixing bolts can be proof tested, angles and plates can be fixed to either side of joint (designed as ‘push’ or ‘pull’ types).</td>
<td>Angles/plates are unsightly, these require boxing in/setting into screed. Works cause disruption to residents during installation. Option relies on strength of fixings.</td>
</tr>
<tr>
<td>Resin bonded anchors I8</td>
<td>Resin bonded single/double inclined threaded anchors (wall/floor)</td>
<td>Inclined ties are hidden from view.</td>
<td>Requires high accuracy drilling using jig. Relies on integrity of in-situ joint for effectiveness. Floor voids can prevent effective installation. Proof loading difficult/impossible to achieve. Requires access to both sides of wall for fully effective tie (assumes a pair of opposing ties). Option relies on strength of bond to concrete and complete resin filling of drilled hole – past experience suggest this not always achieved.</td>
</tr>
</tbody>
</table>

* Costs of remedial options have not been considered in this summary.
I4 COMPONENT STRENGTHENING

I4.1 Principle
Where an assessment has indicated that a wall or floor component is unlikely to be able to accommodate normal or accidental loads (this situation could arise due to the identification of particularly low concrete strength, inadequate or poorly positioned reinforcement or an enhanced loading criterion), then it may be considered necessary to enhance the load carry capacity with respect to shear and/or flexure in order for the element to meet the necessary assessment criterion. It may also be necessary to reduce deflections of a floor slab, although this situation is unlikely to be a problem in LPS dwelling blocks when subjected to normal gravity loading conditions.

Consideration may also need to be given to the performance of joints between wall or floor components (see section I5).

I4.2 Options

I4.2.1 Bonded steel plates
Plate bonding is a well established concept whereby thin steel plates are applied to plane concrete surfaces using epoxy resin adhesive and sometimes bolted fixings.

Steel plates or strips can be bonded to either the soffit of a floor slab (primarily to enhance flexure capacity under downward loading), or following the removal of the complete or strips of screed, to the top surface of the slab (primarily to enhance flexural capacity under upward loading). The steel plates/strips can either be bolted to the slab using appropriately designed fixing bolts and/or two component epoxy adhesives.

It may not always be practical to introduce steel plates onto hollow core floor slabs and particular care is likely to be required for such applications, depending upon circumstances. This is especially so if there are concerns about the corrosion of reinforcement embedded in slabs with, for example, cast-in chlorides. In such instances reinforcement corrosion can cause cracking and delamination of the flange zone concrete at the junction with integral webs, which can be difficult to detect.

General design guidance on steel plate bonding is given by the Highways Agency document which covers the use of this technique for flexural strengthening of reinforced concrete. However, further guidance can be obtained from design engineers who have gained wide experience in the design and implementation of this form of strengthening over a number of years.

Whilst the steel plates used to strengthen the floor slabs in upward flexure can be hidden within the depth of the existing floor screed, those fixed to the slab soffit will need to be hidden behind a false ceiling due to the unsightly nature of the steelwork. Consideration, however, must be given to the long term durability of the plates in areas of high humidity or where there is a risk of the plates coming into contact with water (ie bathroom floors). There may also be a long term durability issue with the adhesives. Furthermore, there is concern over the behaviour of the plates and adhesive in a fire.

I4.2.2 Externally-bonded FRP composites
The use of fibre composites, including carbon, glass or aramid fibres, for the repair/strengthening of concrete structures has increased rapidly over the past few years. Numerous types of structures have been strengthened using FRP composites. These have included bridges, commercial and industrial buildings, staircases, car parks, marine structures and large panel structures.

FRP composites have been used for example, to increase the flexural capacity of Bison floor slabs in cases where the bottom face tension reinforcement has been misplaced within the depth of the slab, or laterally such that certain lengths of the reinforcement run within the cast-in voids.

Unprotected lengths of this material (and adhesive) have a poor performance in fire. Accordingly, in some instances (ie floor soffits) supplementary fire protection should be provided. This may take the form of a false ceiling or a coating of an intumescent material. This material is also particularly susceptible to impact damage and vandalism is left exposed.

A major revision of The Concrete Society’s Technical Report 55 was published in November 2005. TR 55 is widely accepted as the industry standard for FRP strengthening and it includes a review of applications of FRP strengthening in the UK and abroad. It provides guidance on the strengthening concrete structures, including techniques such as near-surface-mounted reinforcement and shear strengthening. Reference is also be made to methods for strengthening other types of structures, such as masonry and metallic structures.

Technical Report 57, February 2003, provides further information on the subject. Document TR55 should be used in conjunction with TR57.

I4.2.3 Supplementary bearing steelwork
When there is a question over the shear capacity of the bearing nibs to certain floor panels under the specified loading, appropriate actions will need to be taken to provide an adequate level of safety. Remedial solutions could take the form of supplementary steel angles bolted to the head of the wall panels, or by the addition of steel plates bonded to the soffit of the floor panels immediately adjacent the face of the wall panel.

There are no perceived general limitations or shortcomings associated with this type of remedial solution apart from its unsightly appearance and durability where there are high levels of moisture. One solution to this problem might be to install a false ceiling, comprising plasterboard on softwood timber battens. A second approach would be to provide a decorative cornice moulding. Whichever option is selected, care must be taken in selecting the appropriate grade of steel for the prevailing environmental conditions. The use of appropriate grades of corrosion resistant steel components should be considered, for example for use in kitchens and bathrooms with their higher moisture levels. It may be appropriate to use galvanised or stainless steel components in such or other locations where enhanced longer-term durability is required.
I4.2.4 Replacement load bearing partitions
This option may provide a structural solution in certain circumstances. However, the design and internal layout of most LPS dwelling blocks presents potential problems in that generally there are very limited opportunities for installing additional walls. However, in certain circumstances it may be possible to replace the existing non-load bearing partitions by small section walls incorporating a loadbearing steel framework or similar.

Areas where this form of construction could be utilised include, the external partially glazed partition between the lounge and balcony, the separating partition between bedrooms and adjacent hallways, and the enclosures that often house a heating unit situated at the junction between the kitchens, lounges and hallways.

I5 DRY PACKING/IN-SITU JOINT ENHANCEMENT

I5.1 Principle
Dry packing uses a very dry concrete mix that is rammed into the temporary gap between the base of a wall panel and top of the in-situ joint situated beneath the wall panel in many LPS dwelling block types. It therefore forms a key ‘feature’ that facilitates the transfer of imposed, wind and dead loads down through a LPS dwelling block to the foundation level. It also, however, forms a potential plane of weakness at the base of the wall panels. It is identified as a possible contributory factor in the potentially sliding failure at the base of wall panels that are subjected to significant lateral loading from an internal gas explosion.

Therefore to minimise the risk of sliding failure at the base of wall panels it is important to provide a sound and strong bearing for each panel. Defects such as poorly compacted, low strength or missing dry packing will understandably serve to lower the threshold at which sliding failure will occur.

Shortcomings in the quality and provision of dry packing have frequently been found in cases where access to install the dry pack was only possible from one side of the wall panel. These situations include both the flank and elevation walls. The dry pack beneath spine walls bordering bathrooms (particularly those immediately adjacent baths) has often been found to be of poor quality. Relatively long lengths of dry pack have also been found to be missing at positions where vertical electrical conduits had been taken up through some wall to floor slab joints (the in-situ concrete was also found to be absent from these areas which sometimes left the mechanical floor/wall connections exposed).

Where these types of defects are found to be widespread within an LPS dwelling block, or are present in areas where an assessment indicates that there is a significant risk of sliding failure at overpressure loads that are less than the assessment criterion, then it would be prudent to undertake remedial repairs. Deficiencies in the dry packing and, as noted above, the in-situ concrete where electrical conduits had been taken up through some wall to floor slab joints may also provide a potential route for smoke and fire transmission.

I5.2 Options
Repairs to the dry packing could take a number of forms. However, restrictions on noise and air borne dust could well limit the available practical options. One or two approaches may be adopted.

Test drillings of the dry pack, using a small diameter drill bit, should be undertaken at regular intervals along the base of the wall panel to identify any missing or weak areas. Any suspected ‘buttered up’ dry packing should be removed to reveal the full extent of the missing or poor quality material.

Depending on what the test drillings and subsequent visual inspection reveal, it might be appropriate to undertake localised patch repairs using a low shrinkage concrete repair material. In extreme cases where appreciable lengths of dry packing were missing and/or are of particularly poor quality (ie highly friable), it might be more appropriate to replace all of the material. This task would be undertaken in a phased manner with alternate short lengths of dry pack being replace using an underpinning type of working methodology. The intervening sections would only be cut out and replaced once the initial repaired sections had cured and achieved sufficient strength.

The use of resin injection might also provide an alternative repair method but there is an appreciable risk that significant quantities of repair material may be ‘lost’ into adjoining rooms, flats or floor voids (where present) if the correct choice of resin viscosity is not made. It is recommended that test trials are undertaken at several locations to assess the viability of this repair technique, before a major programme of work is commissioned. The use of such materials may also introduce subsequent problems with the emission of volatile organic substances into the internal environment, which may create health and safety and wellbeing issues.

I6 SUPPLEMENTARY TYING

I6.1 Principle

I6.1.1 Vertical ties
Any perceived shortcomings in the vertical tying provision within a LPS dwelling block can be rectified by installing segmental ties attached to individual wall panels forming a vertical ‘run’ of walls. However, the type and configuration of the tie steelwork will depend upon the degree of tying requiring within a block together with the reinforcement provision within each wall panel. A further consideration is whether this form of strengthening is designed to allow the section of a building above the site of an explosion to ‘hang’, or to act as a downward tie thus preventing uplift on the structure under accidental loading.

The design of one form of vertical ‘tie’ examined by BRE would seem to suggest that it was intended to have a dual role. Acting partially in compression (small section thick walled hollow box section), and partly in tension (box sections in upper/lower storeys bolted together).
I6.2 Options

I6.2.1 Vertical tie options
These types of ties can comprise storey height steel straps face bolted to the wall panels. Vertical continuity between storeys being achieved by site welding joining strips threaded through slots cut through the ceiling and floor slabs. This modular/site welded approach is easily able to accommodating variations in floor to ceiling height.

Alternatively rectangular box sections with welded end plates can be used to provide a similar function, and can also sustain a certain degree of compression load.

In addition, cast in-situ internal concrete columns can also provide a vertical tie function, depending upon the continuity of the vertical reinforcement provided (see section I7.2.1).

I6.2.2 Horizontal tie options
Long high tensile horizontal tie bars can be threaded through voids in the floor panels; the ties situated adjacent the flank walls being installed from the outside of the building.

Alternatively, high tensile rods can be threaded through holes drilled adjacent the head of the flank and cross wall panels and notionally attached to the sofit of the floor slabs (a false ceiling can be used to mask the tie rods) or they can be embedded in slots cut in the floor screed. The ends of these types of ties are generally secured using resin filled expanding sock anchors or steel anchor plates.

Long high tensile steel rods can also be introduced into hollow core concrete floor slabs in conjunction with cementitious filled sock anchors, which provide effective bonding and anchorage for the bars in these circumstances.

I7 SUPPLEMENTARY LOAD BEARING STRUCTURAL ELEMENTS

I7.1 Principle
If it is demonstrated that it is prohibitively expensive or impossible to strengthen individual wall panels and associated joints to accommodate the forces associated with either the 17 kN/m² or 34 kN/m² overpressure assessment criterion, then there are only a limited number of options available.

One approach which could be adopted as part of an overall strengthening scheme is the installation of a supplementary structural system. This could consist of either an internal or external steel or concrete frame.

I7.2 Options

I7.2.1 Cast in-situ internal concrete columns
This approach was employed to strengthen existing LPS dwelling blocks following the collapse of Ronan Point. It involves the introduction of cast in-situ reinforced concrete columns adjacent both flank walls. Whilst we have no record of the design philosophy adopted, it is believed that the columns were intended to provide an alternative load path. Thus the flank wall panels (and possibly the adjoining floor panels) situated above the site of an explosion, would be supported by the columns in the event of the loss of one or more flank wall panels.

Small section reinforced concrete columns were cast against the inside face of the flank walls to a number of Fram Russell blocks. Vertical continuity was ensured by inserting continuity reinforcement through holes formed in the solid floor slabs and attaching this to the reinforcement cage of the columns in the storey below. It has not, however, been possible to establish whether any shear bolts/reinforcement was provided between each column and the flank wall panel behind, although localised excavations revealed the presence of steel plates, whose purpose is not known.

I7.2.2 External steel frame
Another approach that has been successfully adopted by at least one housing authority is the installation of an external steel frame on a medium rise development constructed using the HSSB system. A steel frame was installed immediately adjacent, and connected to, the flank wall panels at each end of the individual blocks. It is understood that the frames were used to increase the robustness the structure. The frame would also have enhanced its lateral racking resistance. A decorative masonry flank wall was used to hide the steelwork.

I8 STRENGTHENING OF JOINTS BETWEEN COMPONENTS

I8.1 Principle
The strength of wall/wall and wall/floor panel joints can be compromised for a variety of reasons. These include poor quality dry packing, as well as missing or incorrectly placed site reinforcement. The integrity of some joints may also be affected by differential movement between adjacent panels which might occur as a result of a fire or explosion. There may also be a requirement to enhance the sliding resistance at the head or base of a wall panel where an assessment has shown that the load run-down within a wall type is insufficient at a given height in the building to provide adequate frictional restraint to lateral (out-of-plane) movement. Localised strengthening can offer a convenient solution to this problem.

I8.2 Options
There are three basic methods that can be employed to improve the integrity or local strength of both wall/wall and wall/floor panel joints. The available methods include steel stitch plates, steel angles and resin bonded single/double inclined high tensile threaded anchors.
APPENDIX J

LPS DWELLING BLOCK ASSESSMENT CASE STUDIES

J1  INTRODUCTION
Appendix J presents details about the following five LPS dwelling block assessment case studies. These have been made anonymous, but details are given of the LPS dwelling block type and of the general area of the UK in which the blocks concerned are situated.

J2  Case Study A: Pre-Ronan Point Bison Wallframe block: Midlands

J3  Case Study B: Pre-Ronan Point Jesperson: London Borough

J4  Case Study C: Reema Scissor Type Block: South Coast

J5  Case Study D: Pre- and post-Ronan Point Fram Russell: North-West England

J6  Case Study E: Pre- Ronan Point Bison Wallframe: North-East England

J2.1  Introduction
There was a requirement to assess the behaviour of a high-rise pre Ronan Point Bison Wallframe LPS dwelling block against the accidental loading criterion associated with non-piped gas explosions. This followed on from a comprehensive programme of invasive investigations carried out in two flats situated at different levels within the block. The purpose of these investigations was to ascertain whether the LPS dwelling block had been constructed in accordance with the design intentions.

The LPS dwelling block was rectangular on plan. Concrete cores were obtained from a selection of the wall and floor panels and tested for compressive strength. The testing indicated that the characteristic compressive strengths for both the wall panels and floor slabs were very low, indeed the lowest that had been reported to BRE for a Bison Wallframe LPS dwelling block.

J2.2  Floor panels
The results of the simplified calculations undertaken by BRE indicated that the floors failed to satisfy the 17 kN/m² assessment criterion for upward flexure associated with overpressure loading from a non-piped gas explosion in the flat below the floor in question. However, the floors satisfied the 17 kN/m² assessment criterion for downward flexure (associated with overpressure loading from a non-piped gas explosion in the flat above the floor in question).

It was, therefore, decided to undertake a non-linear finite element analysis of both the short and long span floor slabs under both upward and downward loading to see if this approach was able to show that these floor slabs satisfied the 17 kN/m² assessment criterion (again this was concerned with overpressure loading associated with a non-piped gas explosion).

The programme of non-linear finite element analysis runs was undertaken using the strength data derived from the concrete cores taken from the floor panels. These indicated that the long and short span floors should be able to resist both the downward and upward forces associated with an assessment overpressure loading of 17 kN/m² applied separately to the top and to the soffit of the floor slabs, respectively.

J2.3  Wall panels
The simplified analyses of the flank and cross wall panels indicated that the majority of these elements up to the top few storeys would resist base sliding failure for the assessment criterion for accidental loading of 17 kN/m². However, the cross and flank wall panels on all but the lower two floor levels failed to meet this assessment criterion for flexural resistance. Indeed the calculated maximum tensile stress in the outer fibre of the inner leaf to some flank wall panels was over twice that recommended in MHLG Circular 71/68[5], which was one of the guidance criteria used to inform this assessment.

Clearly if failure of a flank wall panel occurred then the whole of the flank wall and adjoining long span floor slabs might become destabilised. Due to the absence of vertical ties (which were not required at the time of construction of pre-1968 Bison Wallframe LPS dwelling blocks), this situation could potentially lead to progressive collapse in the flank wall flats and potentially to a disproportionate degree of damage.

The apparently high incidence of predicted flexural failure arose from the very low characteristic tensile strength obtained for the concrete wall panels sampled. The tensile strength value was derived from the estimated characteristic compressive strength of the concrete in the precast panels.

A difficult position had been reached, whereby the wall panels failed to meet the assessment overpressure criterion of 17 kN/m² for a non-piped gas explosion, but that the floor slabs satisfied this assessment criterion. This led to the conclusion that, in the highly unlikely event
of a ‘severe’ non-piped gas explosion occurring, the wall panels could fail in flexure potentially leading to a progressive collapse and possibly to a disproportionate degree of damage to the LPS dwelling block.

The initial response to this result was to condemn the building on structural grounds. However, the LPS dwelling block concerned was very popular with the residents and, accordingly, there was a desire to retain it in service, if at all possible. As a result consideration was given to alternative ways of dealing with or managing the problems that had been identified. Some of the approaches could be taken to act together as a potential package of measures, whereas it was also possible for some of the approaches to be implemented in isolation.

The potential approaches considered included:

- Active management of the situation in order to reduce the ongoing probability of an internal gas explosion. The gas safety education measures would have taken the form of warning the residents of the block, security staff and area wardens of the risks associated with using and storing bottled gas or any other potentially explosive materials, together with the potential implications should a gas explosion occur in the block. It was noted that this warning should take the form of clear and concise notices positioned at eye level on doors in key circulation areas at all floor levels in the LPS dwelling block. Similar warning notices were proposed for community centres/housing centres used by the residents. It was recognised that these warnings would need to be supplemented at regular intervals by direct circulars to the residents posted through their letterboxes.

- Obtaining and testing further concrete cores from the walls in the LPS dwelling block to see if the additional results led to an increase in the estimated characteristic compressive strength (it was noted that such testing could also lead to a reduction in the estimated characteristic compressive strength). This approach, although relatively costly, had previously been used by another local authority to justify a higher concrete strength value. In this instance the consultant was able to demonstrate that at least 95% of the wall panels would be expected to survive a ‘severe’ non-piped gas internal explosion. This was a risk that was considered by the client to be acceptable, taking into account the extremely low probability of the occurrence of an internal non-piped gas explosion.

- Strengthening all ‘high risk’ wall panels using carbon fibre reinforcing strips. Whilst strengthening of the cross walls could be readily undertaken as access could be gained to both faces, this was not the case with the flank wall panels due to the presence of the non-loadbearing outer leaf and associated insulation layer. The presence of these components prevented access to the outer face of the wall panel, which would be put into tension during an internal gas explosion. The only option available would have been to remove the outer leaf and insulation layer of the flank wall panels and to bond carbon fibre strips to the exposed face of the inner leaf. The flank walls could then have been overlaid with a proprietary cladding system or the original outer leaf could have been reinstated with the existing or newly cast concrete replacement units. However, this approach would result in a reduced load run-down in the flank wall panels thereby increasing their risk of failure due to out of plane sliding.

- Decant the block in the medium to long term, demolish and re-develop or sell the site.

- Move from the deterministic analytical approach that had been used previously, to one that was fully probabilistic in nature. However, since no formal methodology existed at the time for formulating an analysis of this type, it was necessary to undertake a research study to develop the approach. This involved gathering the necessary statistical data on the occurrence of explosions in buildings, gas pressure loadings and how these vary naturally. It was also necessary to establish the potential effect of venting upon the overpressure loadings that could be generated during an internal gas explosion. The overpressure is influenced by the relative proportion of ‘ventable areas’ in the surfaces bounding and enclosing the locations of potential internal gas explosions. A key factor is the relative proportion of glazing to solid concrete panels. As glazing typically fails at a lower overpressure than the solid concrete panels, a larger percentage of glazing reduces the overpressure generated. Data were also needed upon the variation in concrete strength in the panels forming the building. Armed with these data, the necessary probability distributions were derived to allow the level of ‘risk’ to be estimated for appropriate combinations of these conditions and parameters.

Consideration was also given to mitigating circumstances. This action was a short-term first step to gain time whilst a satisfactory longer-term solution was developed. Not surprisingly, the local authority concerned raised the question as to ‘whether it was “safe” for the residents to remain within the building’.

This question is perhaps better posed in a different way as follows: ‘Are the safety risks, which will always be present in any situation and cannot be avoided, manageable and considered as acceptable to various stakeholders and more broadly to society?’

An overview of the above issues is presented in the following sections.

### J2.4 Probabilistic risk assessment

#### J2.4.1 Overview

In view of the impasse outlined above the consultant was commissioned to conduct a probabilistic structural assessment of the LPS dwelling block. This considered the risks related to structural collapse caused by an internal gas explosion. An outline of the assessment process adopted, together the resulting recommendations, is presented below.

The structural risk assessment considered issues that could cause a progressive collapse in the LPS dwelling block, such as the loss of one or more of the load bearing walls within the block. The LPS dwelling block contained...
two types of flats and various other element configurations that needed to be considered. It was anticipated that the highest risks might arise in one of the corner flats in the LPS dwelling block. The walls of particular interest were the cross and flank walls, which provide primary support to the lounge and kitchen floors and were locations where an internal gas explosion could occur.

Three types of criteria were used in the structural assessment to judge whether the safety of the LPS dwelling block was acceptable. This required the simultaneous satisfaction of three conditions which were as follows.

**Criterion 1.** The reliability of structural components had to be acceptable relative to levels adopted in current structural design codes.

It was specified that the probability of progressive collapse in the LPS dwelling block must be less than the generally accepted safety level for a structural component, as defined in structural reliability codes of practice. ISO 2394 defines an acceptable probability of failure of $7.2 \times 10^{-6}$, which is approximately $1/10,000$ for a 50 year period.

On the basis of the conservative assumption that all the uncertainty is associated with the time-varying loads, the above value equates to probability of failure of $1/500,000$ for a period of one year. The basis for this calculation is not strictly correct because there are other factors which do not vary significantly with time, such as the strength of the various panels in bending and sliding (unless for example deterioration is taking place and is taken into account). However, the calculation is, however, a safe upper bound on the correct answer. Thus the probability of failure in $n$ years may be considerably less than approximately $n$ times the probability in one year.

**Criterion 2.** Safety risks to an individual. These were to be compared with what is perceived to be acceptable by the society, and for occupational health and safety purposes, the levels defined as being acceptable by the HSE.

In the absence of other appropriate guidance, the risk of fatality to individuals may be based upon the HSE requirements for tolerable risk in industrial situations. Accordingly, this risk was required to be lower than the HSE tolerable limit of the risk of death of any member of the public from an industrial operation of $1/10,000$ per year. Consideration was also to be given to another HSE criterion, that of the level of tolerable risk of the death of any member of the public from nuclear operations; where a risk of death of $1/100,000$ per year is considered acceptable. Satisfaction of this additional requirement was considered to be desirable, but not essential.

**Criterion 3.** Safety risks in terms of the significance of an event and its consequences, together with its acceptability as established in terms of public perception. This criterion recognises that society appears to be prepared to accept only a very low probability of occurrence of accidents where there is a large consequence/number of casualties, but that society is willing to accept a higher frequency of occurrence of lower consequence/casualty accidents.

Compliance with socially acceptable risk levels was established by reference to the ISO 2394 ‘F–N diagram’. In addition, it was desired that HSE’s related criterion developed for workplace and industrial situations should be satisfied, as set down in the CIB Report 259 Risk assessment and risk communication in civil engineering. Thus if any one of these three criteria were not met, then the LPS dwelling block would be considered to have ‘failed’ the ‘safety’ evaluation.

The probabilistic studies used material properties, structural details and loads other than the overpressure as used in previous deterministic calculations undertaken by the consultant. However, the concrete compressive strengths for the concrete cores were statistically re-analysed so that they were in a suitable form for the probabilistic assessment methodology. The statistics on internal gas pressures were derived from past data collected by the consultant. As with the concrete strength data, the data on gas pressures were re-analysed and translated to a form suitable for probabilistic analysis. Justifiable assumptions and simplifications were carried out as necessary.

The safety assessments undertaken involved simplified structural analysis combined with component reliability analysis (using the program COMREL) and system reliability analysis (using the program SYSREL). The following are important assumptions made in relation to the risk assessments about the potential performance and likely failure modes of the wall and floor slab panels involved. That:

- As this pre-Ronan point building had been retrofitted with mechanical connections between the walls and floor slabs, the loadbearing wall would not suffer sliding failure during an internal non-piped gas explosion.
- The mechanical connections between the walls and floor slabs would help to maintain frictional forces upon the horizontal joints in the walls, by reducing the effect of the uplift force transmitted via the ceiling slab.
- The bolts in the retrofitted mechanical connections between the walls and floor slabs were anchored into the concrete members using adhesive anchorages, but that these connections could not be expected to mobilise alternative load paths in the case of a loss of a load bearing wall panel. Accordingly, it was (conservatively) assumed that the loss of a single load bearing wall panel would be sufficient to cause initiation of a progressive collapse in the LPS block.
- The floors and walls were strong in shear, making flexural failure precede any shear failure.

The risk assessments considered the safety of the LPS dwelling block in terms of structural reliability of the wall panels and the potential number of fatalities. Whilst these factors were the main concern of the building owner, economic risks could also have been considered if desired.
J2.4.2 Results of the probabilistic risk assessments

The outcome of the risk assessment, in relation to the three criteria mentioned above, is given below. The building was seen to be sufficiently safe. However, it was recommended that the gas safety measures mentioned above should continue in order to keep the probability of a gas explosion low.

1. **The reliability of structural components**: The maximum fifty-year probability of a progressive collapse due to an internal gas explosion anywhere within the LPS dwelling block was estimated to be $0.86 \times 10^{-4}$. This is lower than the HSE’s tolerable limiting value for the probability of failure associated with an industrial operation of $1/10,000 = 1.0 \times 10^{-4}$ for this period. This value is also compatible with the criteria for structural reliability employed in the structural Eurocodes (refer Criterion 1 established above). On the basis of the conservative (ie adverse) assumptions made with respect to the ‘simplified’ structural model used to represent the behaviour of the building, this outcome was considered to be acceptable.

2. **Safety risks to an individual**: The annual individual risk level was estimated to be just below $0.7 \times 10^{-4}$ per year, which is less than the HSE limit of tolerable risk of death of any member of the public from an industrial operation of $1.0 \times 10^{-4}$ per year (refer Criterion 2 established above).

3. **Compliance with socially acceptable risk levels**: The estimated number of fatalities associated with the potential progressive collapse of parts of the LPS dwelling block for a non-piped gas explosion are shown in the F–N diagram of Figure J1. This was calculated on the basis of assumed occupancy levels of two occupants on average in a one-bedroom flat and three occupants on average in a two-bedroom flat, and that all occupants stay indoors permanently (24 hours a day and seven days every week); the results indicate that the risk level is acceptable. Even for the higher-risk situation of four occupants in each two-bedroom flat, the risk level was considered to remain acceptable (see Criterion 3 established above).

J3 **CASE STUDY B: PRE-RONAN POINT JESPERSON BLOCK: LONDON BOROUGH**

J3.1 **Introduction**

A number of medium-rise pre-Ronan Point Jesperson 12 M blocks were located in this north London Borough. The five- and six-storey blocks were constructed prior to the Ronan Point partial collapse in 1968. Balanced flue gas boilers were present in the kitchen of each flat and the LPS dwelling blocks had not been previously strengthened to satisfy the post Ronan Point structural assessment requirements.

J3.2 **Structural assessment**

The consultant was instructed to assess the behaviour of one five-storey and one six-storey block against the appropriate accidental loading criterion associated with an internal gas explosion.

Construction information relevant to the structural assessment was obtained during a visual inspection of an

---

**Key**

- **X**: Storebælt project
- **Y**: Delta works project in Holland
- **Z**: Channel Tunnel project
- Points X, Y and Z are discussed in the text below Figure C4 (Annex C).

*Figure J1*: The $F(n) = P(N_d > n) < A n^{-k}$ requirement for one year.
adjacent block that was being demolished and by means of invasive investigations of four vacant maisonettes within the five- and six-storey LPS dwelling blocks.

Since both LPS dwelling blocks contained a piped-gas supply, it was appropriate to assess them in relation to accidental loads on the basis of a 34 kN/m² overpressure applied to all surfaces bordering the location of a prospective explosion. However, since a decision might be taken to remove the piped gas supply, it was decided that the LPS dwelling blocks should also be assessed on the basis of a 17 kN/m² overpressure.

Concrete cores were obtained from a selection of wall and floor panels and subsequently tested to estimate the compressive strength of the concrete.

### J3.3 Results of structural assessment

From the ‘simplified’ deterministic assessment undertaken it was concluded that the flank, spine and cross wall panels and floor slabs would be expected to fail under an accidental loading overpressure of 34 kN/m². It was also concluded that in the event of an internal explosion generating this magnitude of overpressure there was a significant risk that the LPS dwelling block would suffer from a progressive collapse, and potentially, disproportionate damage.

In contrast the LPS dwelling block was expected to behave significantly better during an internal explosion in which the maximum overpressure generating was no more than 17 kN/m². However, the flank and cross walls on the upper two storeys (and possibly the next storey down due to the variable quality of the dry pack) were not expected to survive a ‘severe’ gas explosion if it generated an internal overpressure of 17 kN/m².

The consultants recommended that, if the existing piped gas supply within the LPS dwelling block was to be retained, remedial strengthening works should be undertaken to provide a more robust structure. The proposed strengthening scheme required the installation of horizontal perimeter ties at each floor level and continuous back-to-back vertical strap ties fixed to each wall panel. The floor slabs also needed to be strengthened to improve their behaviour under upward loading associated with an internal gas explosion in the flat on the floor below the slab concerned.

Alternatively, it would have been possible to introduce an independent steel or concrete framework within the LPS dwelling block blocks to provide an alternative means of support should one or more key load bearing elements be lost. The floor panels also needed to be strengthened to resist accidental upward loading associated with an internal gas explosion.

It was recommended that the client instigate further gas safety and contingency measures if the gas supply was to be retained in the short/medium term. These measures included the fitting of safety cut-off valves to the gas supply pipes and the provision of sealed enclosures around the boilers within each flat (the gas supply mains ran up the external face of the LPS dwelling blocks). In addition, it was suggested that a gas monitoring system could be installed within each of the flats to provide early detection and warning of the build-up of a significant level of gas from leaks from the piped supply. Potentially such a system could automatically turn-off the gas supply to a dwelling if there were a build-up of a significant level of gas in the dwelling.

The consultant highlighted that these measures were not an alternative to the need to comply with current guidance for the assessment of LPS dwelling blocks.

No feedback was received from the client on what actions were taken to address the issues that arose from the study.

### J4 CASE STUDY C: REEMA SCISSOR TYPE BLOCK: SOUTH COAST

#### J4.1 Introduction

A severe fire occurred in one of the upper floor flats in this Reema LPS dwelling block situated near the south coast. Smoke and heat from the fire affected a number of flats on the upper two floors as well as the adjoining access corridor. The fire is estimated to have lasted between 30 and 45 minutes. It broke-out of the flat that was the seat of the fire into adjoining parts of the LPS dwelling block.

Following initial discussions and a preliminary site visit, the consultant was engaged to undertake an assessment of the fire damaged structure.

#### J4.2 Assessment

The structural assessment included the following tasks:

- Review of the archive information and potential other information sources.
- Site and laboratory evaluation of materials directly affected by high temperatures (i.e. in the vicinity of the fire).
- Site evaluation of elements and joints between components affected by movements and thermal expansion (i.e. remote from the seat of fire).
- Assessment of affected parts of block for normal loads.
- Assessment of affected parts of LPS dwelling block for accidental overpressure loads due to internal gas explosions.
- Definition of repair objectives and the selection of potential repair options.

The tasks were grouped in the following manner.

#### J4.2.1 Visual inspection and materials testing

The primary aim of the visual inspection of selected areas of the block was to establish the extent of the potential damage, both direct and indirect. Materials testing was undertaken to determine the compressive strength of cores obtained from the fire damaged floor slab, petrographic examination of the heat affected surface zone, and the estimated tensile strength of the heat affected reinforcement.

#### J4.2.2 Structural evaluations

The structural evaluation of the wall and floor panel elements affected by direct heating from the fire considered the ability of the elements to support normal dead loads from the building elements and the associated
domestic imposed loads. The evaluation was made broadly on the basis of seeking to achieve compliance with the requirements of BS 8110: 1985: Structural use of concrete[61].

A structural evaluation was made for the principal load bearing elements in the LPS dwelling block for accidental loading arising from an internal gas explosion. Since the block did not contain a piped gas supply, the assessment was undertaken against an overpressure loading criterion of 17 kN/m².

**J4.2.3 Repair options**

The repair objectives were examined by the consultant and an evaluation undertaken to develop the most practical and effective ways of delivering these, whilst seeking to minimise cost and inconvenience to the block’s residents.

**J4.3 Findings**

The assessment indicated that the fire had caused significant damage to some floor slabs and wall panels within the upper two floor levels of the LPS dwelling block, such that the load capacity of the elements was estimated to have been impaired to varying degrees. It was concluded that structural strengthening would be required to reinstate some of the fire-damaged floor panels. In addition, remedial works were also required to the wall panels including resin injection of some cracked cross-wall panels and the provision of dry-lining. The resin grouting of some horizontal joints between wall panels was suggested to ensure satisfactory vertical load transfer through the dry packing affected by cracking and movements.

**J5 CASE STUDY D: PRE- AND POST-RONAN POINT FRAM RUSSELL BLOCKS: NORTH-WEST ENGLAND**

**J5.1 Introduction**

The local authority concerned owns four 17-storey pre-Ronan Point (Type A blocks) and five post-Ronan Point (Type B blocks) Fram Russell LPS dwelling blocks. Both Type A and B blocks had been strengthened, but the form of the strengthening measures employed were different for the two types of LPS dwelling block.

Several years earlier the blocks had been appraised for their potential behaviour under accidental overpressure loading associated an internal gas explosion. BRE were engaged (as part of a wider programme of work) to review the recommendations made by the consultants.

The summaries presented below include the main aspects of the structural assessments and subsequent recommendations for possible alternative remedial works and management strategies.

From our review of the reports provided it was not clear how the consultants had arrived at certain recommendations. BRE considered it necessary to obtain additional information on the form and quality of construction for both types of Fram Russell Type blocks not examined by the consultants. These investigations were carried out prior to undertaking an assessment of the potential behaviour of these types of LPS dwelling block for accidental overpressure loading associated with an internal gas explosion.

**J5.2 Fram Russell Type A blocks**

An initial inspection was undertaken of selected structural joints between elements in a sister Type A block that was to be demolished for non structural reasons. BRE was also on site during its deconstruction, which provided further information on the joint configuration and detailing.

The strengthening works which had been carried out previously varied the height of the blocks but generally comprised steel angles bolted across the joints between the flank wall/floor, spine wall/flank wall, and elevation wall/flank wall. Steel plates were also bolted across the flank wall/flank wall joints up to the 7th floor level. Steel angles were bolted at the base of the cross wall/elevation wall joints. In addition, back-to-back steel RSJ’s had been installed between the head of the cross and flank wall panels and were located immediately adjacent and either side of the head of the lounge/kitchen partition wall. No strengthening had been undertaken in the top storeys.

None of the original construction drawings were available of the Fram Russell Type A LPS dwelling blocks.

Since the piped gas supply had been disconnected from the Type A blocks it was appropriate to evaluate their strength in respect of accidental loads on the basis of a 17 kN/m² overpressure applied to all the surfaces of the room or enclosure bounding the site of a prospective explosion.

The process of assessment adopted by BRE involved:

1. A review of information available for the Fram Russell Type A LPS dwelling blocks.
2. Comparison of this information with that available for the Reema and Bison LPS dwelling blocks subjected to load testing by the BRE.
3. Materials testing programme to determine concrete compressive strength and reinforcement yield strength.
4. Consideration of the structural evaluation calculations undertaken by BRE for the blocks which were load tested and also for the Fram Russell Type A LPS dwelling blocks.

To undertake the specified assessment the approach adopted essentially sought to make an evaluation of the 'general condition/form' of the LPS dwelling block by considering a number of selected wall and floor element types, and the connections (joints) between these. The main types of joints included:

1. Flank wall.
2. Internal cross walls.
4. Lounge, kitchen and bedroom floors.
5. Flank wall – floor joint.
6. Flank wall – flank wall joint.
7. Flank wall – spine wall joint.
8. Internal cross wall – floor joint.
10. Internal cross wall – elevation wall joint.
The results from the deterministic spreadsheet analysis indicated that the majority of the main structural elements should be able to resist the forces associated with an overpressure of 17 kN/m², albeit possibly suffering some degree of damage.

These deterministic spreadsheet calculations indicated that the long span floor slabs to the kitchens and lounges failed to meet the assessment criterion of 17 kN/m² by a significant margin. It was therefore considered necessary to undertake a non-linear finite element analysis of these floor slabs. The finite element analysis took into consideration the contributions made by horizontal tying and two-way spanning behaviour, as well as non-linear behaviour due to materials and effects such as cracking of concrete. In adopting this approach it was anticipated that it would be possible to obtain a more accurate prediction of the actual behaviour of these floor slabs when subject to upward loading associated with a gas explosion in the flat on the floor below.

The analyses undertaken subsequently indicated that the floor slabs should be able to survive an accidental overpressure loading of at least 17 kN/m² without suffering significant structural distress or excessive cracking of the concrete.

Whilst some of the spine wall panels might be ‘lost’ or damaged by a gas explosion, the consequences of such an outcome were expected to be fairly limited. This was because the spine walls notionally do not provide support to the adjacent floor slabs and also because of the presence of nearby wall and floor elements expected to facilitate the development of alternative load paths at these particular locations.

**J5.3 Fram Russell Type B blocks**

A similar programme of invasive investigations, materials testing, evaluation and calculations to that developed for the Fram Russell Type A LPS dwelling blocks, was adopted for the Fram Russell Type B LPS dwelling blocks.

From our review of the Fram design check calculations (prepared for a Fram Russell LPS dwelling block situated elsewhere in the UK) it appears as though the Type B LPS dwelling blocks were designed to bridge over areas of damage in the event of the loss of one key load bearing wall panel. However, the design features described in the Fram design check calculations appear to be appreciably different from those present in three out of the five Fram Russell LPS dwelling blocks owned by this local authority. From the limited documentation available it was not possible to establish the reason for these, potentially significant, differences.

**J5.3.1 Behaviour of existing structures WITHOUT a piped gas supply (17 kN/m² criterion)**

The deterministic spreadsheet calculations indicated that the majority of the main structural elements in the five LPS dwelling blocks were expected to be able to resist the forces associated with an overpressure of 17 kN/m², albeit possibly having suffered some degree of damage. However, some of the floor panels were judged not to be able to resist these forces on the basis of the ‘simplified’ spreadsheet analyses performed.

However, the results of a previous programme of non-linear finite element analyses of a typical lounge floor within the Fram Russell Type A LPS dwelling block previously reported, indicated that both the short and long span floor slabs were expected to meet the 17 kN/m² overpressure assessment criterion for both upward and downward loading cases.

The presence of existing strengthening steelwork (viz. substantial steel angles and vertical strap ties, and in the case of the flank wall panels, in-situ concrete columns and anchor plates) was expected to enhance the sliding behaviour of the cross and flank walls, in addition to restraining the elevation wall panels.

Accordingly, disproportionate damage was not expected to occur in the five Fram Russell Type B LPS dwelling blocks for an assessment overpressure of 17 kN/m².

**J5.3.2 Behaviour of existing structures WITH a piped gas supply (34 kN/m² criterion)**

The ability of the main structural elements and associated joints to resist the forces associated with an overpressure of 34 kN/m² was examined. The evaluation criteria used were similar to those used in the assessment for the 17 kN/m² overpressure loading.

The cast-in-situ columns, which were understood to be present at all floor levels within the five blocks examined, were expected to limit the extent of damage occurring in the flank wall in the event of the loss of one or more of these components. The presence of bolted steel connection plate(s) sandwiched between the inside face of some flank wall panels and the rear face of the adjoining column, if present at all levels within the five LPS dwelling blocks, would be expected to assist in limiting the number of flank wall panels lost in the event of an explosion. Accordingly, it was concluded that disproportionate damage of the flank wall was not be expected to occur.

The floor panels failed to satisfy the assessment overpressure criterion in flexure and shear for both upward and/or downward loading cases, by a large margin. Even if the results of a previous non-linear finite element analysis of a similar floor slab (Fram Russell Type A LPS dwelling block lounge floor) and associated ‘simplified’ spreadsheet analysis are considered together, it was not possible to provide adequate assurance that the floor slabs would be able to survive a ‘very severe’ internal gas explosion.

Therefore, it was concluded that in the unlikely event of a ‘very severe’ piped gas explosion occurring in one of the five Fram Russell Type B LPS dwelling blocks, there was a significant risk that the floors could become either partially or fully detached from the surrounding structure. It was anticipated that this could lead to progressive collapse and perhaps disproportionate damage of the building.

On the basis of these findings it was concluded that the nature of the strengthening measures present in the Fram Russell Type B LPS dwelling blocks being considered appeared to be in line with other Fram LPS dwelling blocks built elsewhere within the UK at the time, and...
hence in accordance with the available guidance that was relevant at the time.

The LPS dwelling blocks were considered to be sufficiently strong to cope with the forces associated with normal loading.

J6.4 Alternative strategies for the future management of risk
It was also concluded that there could be a potential risk of vertical progressive collapse of the floor panels should a ‘very severe’ piped-gas explosion occur in one of the LPS dwelling blocks, albeit that such explosions are extremely rare occurrences. Therefore measures were recommended to minimise the risk of piped gas leakage and associated explosion, with the attendant risk of progressive collapse. A number of alternative management strategies were proposed by BRE for both the short, medium and long term to address the issues.

In the short-term it was proposed that this be achieved by actively managing the risks by utilising the gas safety education procedures described previously. In the longer-term the preferred option was to remove the source of the risk. The option of demolition was also included as a possible long-term solution, if there proved to be little justification in retaining the LPS dwelling blocks on social or economic grounds.

J6 CASE STUDY E: PRE-RONAN POINT BISON WALLFRAME BLOCK: NORTH-EAST ENGLAND

J6.1 Introduction
A structural assessment was conducted upon three post Ronan Point LPS dwelling blocks which varied in height from 18 to 20 storeys. The blocks were understood to have been constructed in 1967 and 1968, prior to the Ronan Point incident. The piped gas supply was removed from the blocks shortly after construction.

Records seemed to indicate that the precast panels were supplied by Concrete Northern Ltd (Bison). From an external inspection of the blocks they seemed to be of typical Bison LPS dwelling block construction. However, invasive investigations undertaken by the consultants indicated that the flats were not typical Bison Wallframe LPS dwelling blocks, but a variation of the system.

The assessment was undertaken as a precursor to a programme of refurbishment works that included the installation of a new pitched roof and the addition of overcladding. It was also undertaken to confirm the results of a previous assessment of the stability of the blocks undertaken several years earlier and in response to a recommendation to assess the blocks every five years.

The visual inspection indicated that the flank wall to floor slab joints in the top three or four storeys of each block had been strengthened. No strengthening steelwork was reported to have been present in the lower storeys of the three blocks.

J6.2 Review of existing documentation
The results of an earlier programme of materials testing (ie carbonation depth, chloride content and concrete compressive strength testing) and invasive investigations were made available to the consultant. These data indicated a wide variation in many of the parameters measured such as the thickness of the wall leafs, dry pack distribution, floor nib bearing area and thickness of the lower flange of the floor slab panels.

J6.3 Assessment
Since some of the variations noted in section J6.2 above could have a significant influence upon the behaviour of the wall panels and floor slabs under accidental loading, it was decided to analyse these data statistically and to use the results in a programme of simplified deterministic analyses to try and ‘bracket’ the likely behaviour of the wall and floor panels. Typical results of the statistical analysis undertaken are included below, as an example of the variability found in some of the measured parameters.

The following estimated concrete compressive strengths were used in the calculations. These were:

J6.3.1 Floors
- 5th percentiles: Concrete strength 18.0 N/mm², Element thickness 150 mm
- Means: Concrete strength 28.9 N/mm², Element thickness 150 mm
- 95th percentiles: Concrete strength 39.7 N/mm², Element thickness 150 mm

J6.3.2 Walls
The combined ‘normal’ distributions of concrete compressive strength and inner leaf thickness were adopted as these were expected to lead to more realistic predictions of potential behaviour.
- 20th percentiles: Concrete strength 27.9 N/mm², Inner leaf thickness 141 mm
- Means: Concrete strength 36.2 N/mm², Inner leaf thickness 145 mm
- 80th percentiles: Concrete strength 44.5 N/mm², Inner leaf thickness 150 mm

Whilst this ‘exploratory’ approach to the assessment of the behaviour of the wall panels and floor slabs under accidental loading was interesting, it resulted in a range of inconsistent predicted behaviours. In such circumstances it is difficult to make a judgement as to the most likely behaviour. Such judgements can of course have appreciable economic and social consequences.

J6.4 Results of assessment
Hand calculations undertaken by the consultant with respect to the behaviour of the floor slabs under normal downward loading indicated that there was an insufficient reserve of strength in flexure. It was therefore recommended that carbon fibre strips should be bonded to the soffit of selected floor slabs to enhance their strength. BRE understand that no strengthening was undertaken on the upper surface of the floor slabs.

The structural assessment calculations using data from the lower confidence interval and mean of the probability distribution indicated that the majority of the floors did not meet the 17 kN/m² overpressure loading assessment
criterion for accidental loading. However, consideration of the behaviour of the previous LPS dwelling blocks load tested by BRE suggested that many of the floors within the three LPS dwelling blocks should probably perform satisfactorily in upward flexure under the maximum accidental loads associated with non-piped gas explosions.

This opinion was supported by the results of non-linear finite element analyses that had previously been undertaken upon a similar design of floor panel undertaken for other clients. Whilst the results of the ‘simplified’ calculations previously undertaken indicated that the floors would fail the assessment criteria for upward flexure by an appreciable margin; consideration of the influence of other factors such as two-way spanning behaviour, top reinforcement, contributions made by ‘non-loadbearing’ stud partitions and non-linear behaviour enabled the consultants to conclude that the long span floors assessed should be able to accommodate maximum accidental loads associated with non-piped gas explosions, albeit that they were likely to suffering some structural damage as a result.

However, the situation concerning the performance of long span floors within the blocks for upward loading was less clear. In this case the overload factors associated with input data derived from the lower confidence limit and mean values were 3.0 and 2.16, respectively. The results derived from data from the upper confidence limit were not included in the calculations as these were unlikely to be representative of the actual behaviour of the floor slabs.

Accordingly, with the limited material test or characterisation information available, it was not possible to provide any reassurance that the long span floors could accommodate the maximum accidental loads in upward flexure associated with non-piped gas explosions.

### J6.5 Alternative remedial options

A number of possible alternative options/solutions to this problem were proposed. These included:

- Do nothing by way of strengthening the top of the long span floor slabs and accept that in the statistically unlikely event of a non-piped gas explosion occurring in a lounge during the remaining life of the blocks, that there was a risk that the flank wall panels could become destabilised. This could result in progressive collapse, and possibly disproportionate damage, to the structure.
- Undertake a non-linear finite element analysis of the long span floor slabs to evaluate potential behaviour under accidental loading. This would have necessitated compiling and running a suite of different finite element models to assess the sensitivity of the long span floor slabs to variations in the configurations of the voids and top reinforcement. Recommendations were also made for invasive investigations to establish the reinforcement provision in the top of the floor slabs and to determine the variations in void geometry, as well as in the top and lower flange configurations of the floor slabs.
- Strengthen the top of the long span lounge floor slabs in selected flats using carbon fibre strips. It was argued that these works could be restricted to the floor slab adjacent the elevation wall as the adjacent floor slab was partially restrained above and below by the ‘non-loadbearing’ stub partition between the lounges and kitchens. Such works would have required strips of floor screed to be removed to expose the top of the structural slab.
- Undertake a full-scale load test of the lounge floor within a minimum of two first floor flats, allowing the floor to the test flat to be propped from beneath to enable the ceiling to the test flat to be load tested. It would have been necessary to carry out post-test invasive investigations and material tests to enable a judgement to be made as to whether the test floor was likely to typical or atypical of similar elements within the three LPS dwelling blocks. These tests would have demonstrated the likely static behaviour of the long span floors; in an actual non-piped gas explosion the floors would be expected to behave better than during the static load tests because of beneficial strain rate effects.

### J6.6 Durability assessment

A limited number of concrete dust samples were taken from the wall panels and floor slabs in all three LPS dwelling blocks. Laboratory testing indicated that a proportion of the samples contained significant levels of cast-in chlorides up to a maximum of 1.4% chloride ion by weight of cement. No remedial actions were recommended by the consultant to prevent current or future corrosion of the reinforcement embedded in the floor slabs from occurring.

### J6.7 Outcome

BRE understand that the over-roofing and over-cladding works were successfully completed and that the floor slab strengthening works proposed by the consultant were also implemented.
APPENDIX K

BRE TRIALS TO DETERMINE COEFFICIENT OF FRICTION AT BASE OF WALL PANELS

K1  INTRODUCTION
BRE experiments were undertaken to determine the coefficient of friction between a formed concrete surface (i.e., friction between two concrete surfaces) and another surface comprising dry-pack (i.e., friction between concrete and dry-pack surfaces).

These trials were undertaken by BRE to confirm and expand upon the results of a limited series of tests that were carried out by BRE in the early 1990s to determine an approximate angle of friction for concrete surfaces that were in intimate contact.

The recent trials considered the slip behaviour of a concrete cube with a surface finish that was considered to be representative of the base of a precast concrete LPS wall panel, when placed on samples of concrete and dry-pack with varying surface roughness. The 300 mm square samples were fabricated using a moderate strength concrete and four ‘grades’ of dry-pack. (See Figures K1–K6.)

K2  SPECIMENS

K2.1 Concrete specimen
A maximum 20 mm aggregate Portland cement concrete was used to produce a single moderate strength concrete test sample. Sufficient water was added to the mix to make the concrete workable. The concrete was placed in a mould and hand tamped; no vibration was employed as this form of compaction/consolidation was unlikely to have been used on site. The mix was designed for a compressive strength in the region of 15 to 20 N/mm².

K2.2 Dry-pack specimens
Four dry-pack mortar specimens were formed using a mix comprising one part cement and 2.5 parts concreting sand. Sufficient water was added to the mix to produce a mortar that was just cohesive enough to stick together while being moulded into a ball with the hands.

Figure K1: Estimated circa 15–20 N/mm² site mix concrete (representative of that used as a bed for wall panels, for example in the Fram Russell system)
APPENDIX K  BRE TRIALS TO DETERMINE COEFFICIENT OF FRICTION AT BASE OF WALL PANELS

Figure K2: Dry pack – introduced from top of mould – lightly hand compacted using timber tamper, finished level with steel trowel, textured using stiff brush (based upon the approach employed for the Bison Wallframe LPS form of construction)

Figure K3: Dry pack (as for A09/9197/1) – introduced from the top of the mould – lightly hand compacted using timber tamper, minimal finished with steel trowel (based upon the approach employed for the Bison Wallframe LPS form of construction)
Figure K4: Dry pack (as for A09/9197/1) mix prepared by firm side ramming in increments by small section hand-held timber compactor. This taken to represent a ‘reasonable’ level of workmanship (based on the approach used for the Bison Wallframe LPS form of construction).

Figure K5: Dry pack (as for A09/9197/1) mix prepared by light/inconsistent side ramming in large increments by small section hand held timber compactor. This is considered to represent a ‘poor’ level of workmanship (based on the approach used for the Bison Wallframe LPS form of construction). The dry pack mix was allowed to air dry for 30 minutes prior to use.
APPENDIX K  BRE TRIALS TO DETERMINE COEFFICIENT OF FRICTION AT BASE OF WALL PANELS

The test arrangement adopted by BRE is shown in Figure K7. The concrete cube was placed on the concrete/dry-pack specimen and correctly orientated relative to guide marks. The gradient of the support steelwork was gradually increased using a scissor jack until the concrete cube slipped. The angle at which slippage occurred was measured using a digital inclinometer. The face of the concrete cube presented to the test specimen was changed and the slippage test repeated. The test was repeated using three faces of the test cube to enable an average angle at which slippage occurred to be calculated. These tests were undertaken for the upper (hand finished) and lower surface (cast face) of each of the five test specimens.

K4 SUMMARY TEST RESULTS
The mean angle of slippage for the upper and lower faces of each of the five test specimens is included in Table K1. The results are presented for increasing levels of surface roughness (visual assessment only)/voidage.

K5 FRICTION VALUES GIVEN IN CODES OF PRACTICE FOR DESIGN
Guidance on friction values to be employed is given or can be derived from the information presented in a number of codes of practice for design including the CEB–FIP Model Code 1990, EN 1992-1 and FIB Model Code 2010. Various circumstances and surface characteristics are considered.
Table K1: Test results – Measured static friction angle at which slippage occurred

<table>
<thead>
<tr>
<th>Mix reference (Material type)</th>
<th>Angle of slippage (Degrees)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper surface</td>
<td></td>
</tr>
<tr>
<td>A09/9178/1 (Concrete)</td>
<td>35.1</td>
</tr>
<tr>
<td>A09/9179/1 (Dry pack)</td>
<td>33.6</td>
</tr>
<tr>
<td>A09/9179/2 (Dry pack)</td>
<td>37.2</td>
</tr>
<tr>
<td>A09/9179/3 (Dry pack)</td>
<td>36.8</td>
</tr>
<tr>
<td>A09/9179/4 (Dry pack)</td>
<td>36.3</td>
</tr>
<tr>
<td>Lower surface (cast face)</td>
<td></td>
</tr>
<tr>
<td>A09/9178/1 (Concrete)</td>
<td>30.3</td>
</tr>
<tr>
<td>A09/9179/1 (Dry pack)</td>
<td>35.0</td>
</tr>
<tr>
<td>A09/9179/2 (Dry pack)</td>
<td>33.6</td>
</tr>
<tr>
<td>A09/9179/3 (Dry pack)</td>
<td>36.6</td>
</tr>
<tr>
<td>A09/9179/4 (Dry pack)</td>
<td>34.1</td>
</tr>
<tr>
<td>Minimum friction angle measured</td>
<td>30.3</td>
</tr>
<tr>
<td>Average friction angle measured</td>
<td>34.9</td>
</tr>
<tr>
<td>Maximum friction angle measured</td>
<td>37.2</td>
</tr>
</tbody>
</table>

Figure K7: General view of static friction test set-up

Increasing surface roughness/voidage
INDEX

Note: Figures and Tables are indicated by italic page numbers, definitions and glossary terms by emboldened numbers, and footnotes by the suffix ‘n’

A
Abbeystead water pumping station, valve house explosion 66, 108
abbreviations xxi
acceptable risk xxvii
accidental design situation xxi
accidental external gas explosions 21, 23, 110
accidental internal gas explosions 21, 22–23, 106–110
multi-room explosions 23, 32, 110
overpressure loading associated with 23, 31–33, 61
single-room explosions 23, 32, 33, 109
statistics 108, 110
accidental loading, assessment of block under xxx, 67, 72–75
accidental loads and actions xxi, 10
factors influencing behaviour under 37–45
forms 21–25
fires 21, 24–25
gas explosions 21, 22–23
vehicle impacts 21, 23–24
accidental vehicle impacts 21, 23–24, 110–112
aircraft impacts 21, 24, 111–112
road vehicle impacts 21, 23–24, 110
train impacts 21, 24, 111, 112
action xxi
effect of xxiv
active venting 45
aerosol canisters
explosions involving 23, 25, 32, 33, 63, 77, 132, 133
propellant gas in 26, 29, 32, 63
aircraft impacts 21, 24, 111–112
annual probability of occurrence 111, 123
risk issues 29, 62
ALARP (as low as reasonably practicable) principle ivn1, xxi, 16, 27–28
applications 76, 78, 102, 120, 122, 126–127
alternative load paths, mobilising xxvii, 2, 12, 14, 20, 42, 45, 46, 63, 72, 78, 96, 98
after severe fire 57, 59
spine wall test for iv, 35, 160–163
Amagasaki [Japan], train derailment 24, 111
assessment xxi–xxiii
assessment criteria 61–62
see also overpressure assessment criteria
assessment requirements, changes with time 9, 99
asset management ivn2

B
basement xxi, 38
assessment overpressure used 5, 61
‘bath-tub’ performance curve 18, 19
benchmarking of risks xxi, 116–117
Bijlmermeer, Netherlands, aircraft crash 111
Bison Wallframe LPS dwelling block(s) 1
assessment case study (Midlands) 175–178
floor slabs 175
probabilistic risk assessment 176–178
wall panels 175–176
assessment case study (north-east England) 182–183
joints at base of wall panels 41, 42, 43–44
load testing of components 130
number in UK 129
structural load test (Liverpool) 4, 34, 155–163, 164–168
combined wall and floor overpressure tests 157–160
finite element analyses 164–168
overpressure tests 157–160
phases of work in test programme 155
quality-of-construction investigation 156–157
scope of overpressure load testing and element removal programme 155, 156
spine wall test 160–163
structural load test (Sandwell) 4, 34, 133–134, 135, 145–154
combined wall and floor overpressure test 133–134, 135, 145–154
floor overpressure test 134
boiling liquid expanding vapour explosions 110
bonded FRP composites, strengthening by 169, 170, 172
bonded steel plate, strengthening by 169, 170, 172
bottled gas, use in building/repair work 77, 80
BR-ATP (British Rail’s automatic train protection) system 123, 124
BRE definitions
collapse resistance 15
robustness 14
definitions xxii–xxviii

deflagration xxiii

Delta works project, Netherlands [risk assessment curve] 122, 123, 178
demolition 59–60, 64
progressive collapse during 3, 59, 64, 84
design life xxiii
deterioration of buildings xxiii
factors affecting 17, 49, 63
deterministic linear elastic analysis xxx, 20, 67, 72–73, 73–74
deterministic non-linear finite element analysis xxx, 20, 75
detonation xxiii
diagnosis xxiii–xxiv
differentiation of parts of buildings 38
direct action xxii
disintegration xxiv
disproportionate damage xxiv
diversity xxiv
‘domino’ form of progressive collapse 12
dry-pack mortar in joints 41–42, 49
friction tests 184–187, 188
replacement/enhancement of 169, 170, 173
DTI (Department of Trade and Industry) funded research project 3, 4, 155
ductility xxiv
durability, concrete components 48–57
durability assessment 63–64, 183
durability inspection and monitoring 48, 75
durability investigation and assessment procedure 53, 55

E

effect of action xxiv
electricity, risks associated with 83
environment xxiv
equivalent uniform static pressure xxiv
EU (European Union) Workplace Directive 103
explosion incidents
risks associated with 26
statistics on 25, 106–110
explosion testing 23, 132–133
explosions xxiv
classification of 22, 107
external gas explosions 23, 110
risk issues 29, 62
see also internal gas explosions
external steel frame [flank wall], strengthening by 169, 177, 174

F

F–N curves 16, 29, 62, 102, 119–123, 177, 178
failure probability 17
fib (Fédération Internationale du Béton), definition of ‘robustness’ 14
finite element analysis
deterministic non-linear xxx, 20, 67, 75
for full-scale load tests 137, 164
for room overpressure test 165–168
for spine wall test 164–165
finite element modelling 35
fire 21, 24–25
risk issues 29, 62
statistics 112, 113
structural assessment after 57–59
structural stability affected by 25
flank wall/floor slab joints, strength testing of in Reema LPS blocks 130–131
flank walls xxiv–xxv, 1, 5n4
failure in bending 32
peeling off after explosion 12, 13
sliding failure at base of 8, 31, 81
floor overpressure tests
Leeds Reema Conclad LPS dwelling block 134, 135, 137, 138–142
method of analysis 138
floor slabs
assessment of resistance to overpressure loading 74, 75
classes 73
cement strength parameters 39, 40, 41
direction of span 38
fragility of LPS dwelling block 69
Fram Russell LPS dwelling blocks 2, 3, 84, 174
assessment case study 180–182
number in UK 129
Type A blocks 180–181
Type B blocks 181–182
friction/adhesion parameters [at base of wall panels] 42
determination of 184–188
FRP (fibre-reinforced polymer) composites, strengthening by 169, 170, 172
full-scale structural load tests xxix, 3, 4, 19, 82, 133–136
Leeds Reema Conclad LPS dwelling block 4, 34, 134, 135, 138–145
Liverpool Bison Wallframe LPS dwelling block 4, 34, 155–163, 164–168
overview 34–36, 137
protection measures in event of premature failure 133
Sandwell Bison Wallframe LPS dwelling block 4, 34, 133–134, 135, 145–154
structural performance 34–36

gale damage 117
gas boilers, positioning of 46, 47
gas detectors 46–47
gas explosions xxiv, 10, 21, 22–23, 106–110
classification of 22, 106–107
risk issues 29
see also external gas explosions; internal gas explosions
GLC (Greater London Council) building by-laws 97
global hazard xxv
glossary xxii–xxviii
H
hazard environment
  accidental aircraft impacts 111–112
  accidental external gas explosions 110
  accidental internal gas explosions 106–110
  accidental road vehicle impacts 110
  accidental train impacts 111, 112
  fire 112–113
  and probability of occurrence 114–115
hazards xxv
  assessment process 21
  fire 21, 24–25, 112–113
  gas explosions 10, 21, 22–23, 106–110
  probability of occurrence 121
  severity/consequence of 120
  sources 21, 22, 106–113, 119
  vehicle impacts 10, 21, 23–24, 110–112
Health and Safety at Work, etc. Act 1974 103
heating (and subsequent cooling), effects of 57
historical perspectives
  on development of regulatory requirements 96–99
  on LPS dwelling blocks 81, 128
  structural assessment 81–82, 128–130
  horizontal ties, strengthening by 169, 170, 174
HSE (Health and Safety Executive) statistics [on probability of death] 116, 117, 177, 178
Hulme Point, Manchester, very severe piped gas explosion 107, 108
I
individual risk levels 16, 102, 178
industrial accidents, deaths caused by 116, 117
ingress xxv
inhibiting new developments near airport 123, 124, 125
inspection xxv
Institution of Structural Engineers
  guidance
    on security of cladding 9
    on structural robustness 16
  reports 8, 9, 84, 97, 98
internal gas explosions 19, 22–23, 106–110
factors influencing behaviour during 45
multi-room explosions 23, 32, 110
overpressure loading associated with 23, 31–33, 45, 61
risk issues 28–29, 62, 82
single-room explosions 23, 32, 33, 109
statistics 108, 109, 117
see also external gas explosions
intervention xxv
investigation xxv
ISO 2394 [on reliability] 29, 115, 177
J
Jesperson LPS dwelling block(s)
  assessment case study 178–179
  number in UK 129
  joints xxv, 2
    at base of wall panels 41–42, 43–44
  classes 73
  engineering evaluation of 74
  inspection of 48, 64, 75, 98
  strengthening of 169, 171, 174
K
key elements xxv, 61, 65, 100n35, 105
key structural members [in demolition] 59–60
keywords xxi
L
large panel system
  meaning of term xxv–xxvi, 5n4
  see also LPS...
Larsen Nielsen LPS dwelling block(s) 7n10, 81
levelling dowels, contribution from 31, 134
limit of admissible local failure 102
linear elastic analysis, deterministic xxx, 20, 67, 72–73, 73–74
linear finite element analysis xxvi, 137, 164
local collapse xxvi
local damage xxvi
local failure xxvi
local hazard xxv
local road safety, cost benefit analysis 124, 125
London County Council LPS dwelling blocks 81
LPG (liquid petroleum gas) cylinders
  density of gas 32, 47
  explosions involving 32, 33, 107
  risks of storing 77, 97
LPS Criterion 1 xxix, 5, 66, 177
LPS Criterion 2 xxx, 5, 66, 177
LPS Criterion 3 xxx, 5, 66, 177
LPS dwelling blocks xxv–xxvi, 1, 5n4
  configuration of panel construction 1, 2, 38
  number in UK 3, 128
  structural parameters 37–45
  types 1, 128, 129
  see also Bison Wallframe...; Fram Russell...; Jesperson...; Reema Conclad LPS dwelling blocks
M
maintenance xxvi
malicious acts or attacks xxvi, 3, 21, 106, 127
MHLG (Ministry of Housing and Local Government)
  Circulars 62/68 and 71/68 iii, 3, 8–9, 81, 96
Milan, Pirelli building, aircraft crash 111, 112
moderate explosion(s) 22, 106
  examples 77, 107
  statistics on 25, 108, 109
  moderate impact 110
  monitoring xxvi
NHBC (National House Building Council), definition of basements xxiii, 38
nominal length 5n7, 45, 66n26, 100n38
non-destructive evaluation xxviii
non-destructive testing xxviii, 65, 71
non-invasive testing xxviii
non-linear finite element analysis xxvi, xxx, 20, 75
non-piped gas supply
assessment overpressure used xxix, 5, 20, 23, 32–33, 35, 61
explosions 4n3, 32, 35, 62, 107, 108, 110
normal loading, assessment of block under xxx, 67, 71
normal loads and actions xxvi
notional risk of death
in fires 62
in severe explosions 29, 62, 115, 116

Occupiers Liability Acts 96, 105
occupiers of LPS dwelling blocks 83, 84
overpressure xxvi
overpressure assessment criteria
for building with basement 5, 61
for building with piped gas supply xxix, 5, 20, 23, 31–32, 46, 61, 96, 97
for building without basement and without piped gas supply xxix, 5, 20, 23, 32–33, 35, 61, 97
origins 23, 31–33
for sliding failure at flank wall 8, 31
overpressure tests see combined wall and floor overpressure tests; floor overpressure tests
owners of LPS dwelling blocks 83
advice given to 8, 96–97

'pancake' form of progressive collapse 11, 12
permanent action xxii
petrol, explosions involving 33, 77
PII (Partners in Innovation) research project 3, 4, 155
piped gas supply
and ability to mobilise alternative load paths 46
assessment and 69–70
density of gas 32, 47
disconnection/removal of 20, 69, 70, 76, 96, 180
overpressure criterion used xxix, 5, 20, 23, 31–32, 46, 61
risk reduction measures 77
safety issues 46–47
very severe explosions involving 29, 62, 69, 107, 108
potential/kinetic energy in progressive collapse 12, 81
preventive measure 102
proactive approach to through-life management 19, 49, 65
probabilistic based assessment methodology 6, 45, 75
probabilistic risk assessment 176–178
probability-based structural assessment calculations xxx, 20, 67, 75, 76
probability of building collapse 29, 35, 115
compared with other hazards and risks 116–117
replacement load-bearing structural partitions, 169, 170, 173
residents in LPS dwelling blocks 7, 83, 84
resilience xxvii
resistance measures 102
risk xxvii, 27, 118
risk acceptance criterion 29, 63
risk acceptance levels 16, 102
risk analysis 83, 119, 120
risk assessment
  matrix for profiling 119, 121
  methodology 6, 16, 119–127
  use in decision making 27
risk issues 28–30, 62–63, 118–127
risk levels, compared with risks from other hazards 116–117
risk management 27, 63, 118–119
risks
  benchmarking of xxix, 116–117
  reduction and management of 28, 63, 76–77, 83
road vehicle impacts 21, 23–24, 110
risk issues 29, 62
ways to prevent 23
robustness xxvii–xxviii, 14, 61, 65, 97
definitions xxviii, 14–15
Ronan Point Inquiry/Tribunal 7–8, 96–97, 106
Ronan Point LPS block 7, 81
partial collapse iii, 7–8, 81
MHLG Circulars issued after iii, 3, 8–9, 81, 96
see also Taylor Woodrow-Anglian LPS dwelling blocks room overpressure tests, hydraulic loading rig for 157, 158
SCOSS (Standing Committee on Structural Safety)
definition of ‘robustness’ 14
on sources of hazards 119
segmentation of structure 14
seismic actions xxii, xxvi
severe explosion(s) 22, 107
examples 19, 23, 32, 47, 61, 77, 82, 107
factors affecting 32, 107
probability of occurrence 25, 28, 62, 108, 109, 110, 114
statistics on 25, 108, 109
severe fire
repair procedures after 59
structural assessment after 57, 59
severe impact 110
SFARP (so far as is reasonably practicable) principle ivn1, xxviii, 28
application 76, 78
socially acceptable risk to human life 16, 102, 178
cost benefit analysis 123–125
F–N curves 119–123, 178
solidity, workplace regulations and 103
speed cameras, cost benefit analysis 124, 125
spine wall xxviii, 5n4
spine wall test iv, 35, 160–163
spreadsheet-based linear elastic analysis xxx, 20, 67, 72–73, 73–74
stability xxviii, 61, 65, 98
workplace regulations and 103
Starossek, Uwe
definitions
  collapse resistance 15
  robustness 14
on forms of progressive collapse 12
steel plate bonding, strengthening by 169, 170, 172
steel reinforcement 2
corrosion of
  factors affecting 49–50, 50–53, 172
  risk categories 53, 56
depassivation of
  by carbonation 50–51, 52
  by chlorides 51–53, 54, 55
passivation of 50
properties 70
stoichiometric mixture 22n14, 109n42
Storebælt project, Denmark [risk assessment curve] 122, 123, 178
strengthening xxviii, 20, 78, 79, 169–174
of components or panels 169, 170, 172–173
by dry pack replacement/enhancement 169, 170, 173
by supplementary load-bearing structural elements 169, 170–171
by supplementary tying 169, 170, 173–174
vertical extent of 169, 171
structural assessment
  after severe fire 57, 59
  case studies 175–183
  consulting engineer’s perspective 83–84
  contemporary guidance on 82–83
durability inspection and monitoring regime 64, 75–76
hierarchical approach xxx, 20, 67
historical development of requirements 96–98, 128–130
implications to be taken into account 6
owner’s perspective 83
structural assessment calculations
  assessment level 1 [deterministic linear elastic analysis] xxx, 20, 67, 72–73, 73–74
  assessment level 2 [deterministic non-linear finite element analysis] xxx, 20, 67, 75
  assessment level 3 [probability-based calculations] xxx, 20, 67, 75, 76
structural assessment methodology
assessment stage 1 [review of existing technical information] xxx, 67, 68–70
data requirements 69
material properties 70
and presence of piped gas supply 69–70
assessment stage 2 [collection of new technical information] xxx, 67, 70–71
assessment stage 3 [assessment of block under normal loading] xxx, 67, 71
basis of 19–20, 67–68
structural assessment process, steps in xxi, 68
structural diversity xxiv

Structural Eurocodes definitions
‘accidental action’ xxii, xxvi, 10
‘robustness’ xxviii, 14, 15
on disproportionate collapse 16
UK national standards replaced by 5n9, 16, 72, 100n36
see also BS EN 1990; BS EN 1991-1-7
structural functionality xxviii
structural integrity xxviii
structural stability, effect of fires 25
supplementary bearing steelwork, strengthening by 169, 170, 172
supplementary load-bearing structural elements, strengthening by 169, 170–171, 174
supplementary tying, strengthening by 169, 170, 173–174
survey xxviii

T
Tacoma Narrows Bridge collapse 12
Taylor Woodrow–Anglian LPS dwelling blocks 4, 7, 41, 81
technical information
review of existing [in structural assessment methodology] xxx, 67, 68–70
Tehran, Iran, aircraft crash 111
testing xxviii
through-life management
durability inspection and monitoring 48
methodology 66
proactive approach 19, 49, 65
reactive approach 19, 49, 65
risk assessment and reduction measures 83
strategies for 82
through-life performance 17–19
through-life risk management strategy 78, 80
‘time of wetness’, corrosion affected by 49–50
tolerability of risk 28, 123
tolerable risk xxvii, 28
TPWS (train protection and warning system), cost benefit analysis for 124, 125

train impacts 21, 24, 111, 112
transport-related safety measures, cost benefit analysis for 123, 124, 125
TWMV (two-wheel motor vehicle) users, local road safety for 124, 125

U
ultimate limit state design, target life-time reliability index 29–30, 115
uniformly distributed loads, ‘equivalent
Leeds Reema Conclad LPS dwelling block combined wall and floor overpressure tests 142, 145
floor overpressure test 139
methodology for calculating 137–138
Sandwell Bison Wallframe LPS dwelling block, combined wall and floor overpressure tests 146, 150, 153

value of preventing a fatality 16, 123, 126–127
application to LPS dwelling block 127

variable action xxii

vehicle impacts 21, 23–24, 110
venting mechanisms 22, 45, 176
venting panels xxviii, 45
vertical ties, strengthening by 169, 170, 173, 174
very severe explosion(s) 22, 107, 110
examples 19, 23, 65, 107, 108
probability of occurrence 25, 29, 62, 109
risk 69
statistics on 25, 108, 109
very severe impact 110
visual inspections 48, 64, 71, 75, 98
in assessment case studies vulnerability xxviii, 3

W
wall panels
assessment of resistance to overpressure loading 74, 75
classes 73
concrete strength parameters 39, 40, 41
joints at base of 41–42, 43–44
Wates LPS dwelling block system 129, 131
windows, failure of 22, 45
Workplace (Health, Safety and Welfare) Regulations 1992 96, 103
World Trade Center [New York], progressive collapse of 24, 111

Z
‘zipper’ form of progressive collapse 11, 12
Dynamic comfort criteria for structures
A review of UK codes, standards and advisory documents
Understand all the acceleration comfort criteria associated with the vibration of buildings. Aimed at building designers, consultants, architects and structural engineers, this BRE Trust report reviews vibration comfort criteria in Eurocodes, UK and ISO Standards and other sources, for three areas where vibration is an important recurring issue: buildings, floors and grandstand risers.

Ref. FB 33, 2011

Concrete structure management
Owners’ guide to good practice
Maximise the benefits to be gained from your concrete structures, while minimising through-life cost and sustainability impacts. This BRE Digest gives building owners an insight into their responsibilities, what they should do and seek to achieve in the context of concrete structure management. It describes stages in the life of an asset, potential deterioration mechanisms to be avoided and actions to be taken. The advice given is based on ‘Concrete structure management – Guide to ownership and good practice’ (fib Bulletin 44).

Ref. DG 510, 2009

Concrete repairs (2-volume set)
Explore the performance of concrete repairs and a performance-based intervention strategy with these reports derived from the European CONREPNET network on concrete repair.
Concrete repairs: performance in service and current practice (EP 79) assesses case histories and reviews the problems of concrete durability, current issues of sustainability, and the differing expectations of what concrete repairs should achieve.
Achieving durable repaired concrete structures: adopting a performance-based intervention strategy (EP 77) offers a new way forward for achieving durable and long-lasting concrete repairs.

Ref. EP 81 (2-Volume set), 2007

All titles are available in print and pdf format.

Order now @ www.brebookshop.com or phone the IHS Sales Team on +44 (0) 1344 328038.
HANDBOOK FOR THE STRUCTURAL ASSESSMENT OF LARGE PANEL SYSTEM (LPS) DWELLING BLOCKS FOR ACCIDENTAL LOADING

This handbook presents new guidance on the structural assessment and strengthening options for large panel system (LPS) dwelling blocks, focusing primarily on their resistance to accidental loading associated with gas explosions, and supported by extensive background information.

The progressive collapse of part of Ronan Point tower block in east London in 1968 was a significant event in structural engineering in relation to the understanding of disproportionate damage to structures. Extensive research and investigations since then, including full-scale structural load tests on a block in Liverpool, are taken fully into account.

This handbook:
• defines the requirements to be met and the criteria against which the results of a structural assessment of this particular class of building should be judged. These are seen to effectively supersede the previous guidance set down in the Ministry of Housing and Local Government Circulars dating back to 1968
• gives guidance on how to undertake the structural assessments which are required, drawing on previously unpublished technical information
• details the historic background to these requirements, with these being brought up to date and set in the contemporary philosophical context of the requirements of the recently introduced structural Eurocodes
• explains the risk environment which applies to this class of building
• provides an overview of durability assessment/intervention and strengthening options.

RELATED TITLES FROM IHS BRE PRESS

STRUCTURAL APPRAISAL OF EXISTING BUILDINGS, INCLUDING FOR A MATERIAL CHANGE OF USE
DG 366 (4 Parts), 2012

DYNAMIC COMFORT CRITERIA FOR STRUCTURES
A review of UK codes, standards and advisory documents
FB 33, 2011

STRUCTURAL FIRE ENGINEERING
AP 283, 2011

CONCRETE REPAIRS
EP 81 (2-volume set), 2007

CONCRETE STRUCTURES IN FIRE
Performance, design and analysis
BR 490, 2007