

Management of ageing assets - Multi-storey dwelling blocks

Assessment of Large Panel Buildings Under Accidental Loading

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INTRODUCTION

There are still over 700 high rise LPS blocks ($\approx 50,000$ dwellings) in the UK and block owners have an ongoing responsibility to manage them. This activity requires periodic inspection and assessment of the blocks concerned. In addition there are also in excess of 1000 low- and medium-rise LPS blocks in the UK.

The assessment requirements stem from the collapse in the UK of the south east corner of Ronan Point, a 22 storey large concrete panel system dwelling block, in 1968. This followed a piped-gas explosion in an 18th floor flat which displaced load bearing wall elements and led to a vertical progressive collapse (see Figure 1). The incident resulted in changes in the UK Building Regulations, the introduction of 'robustness', as well as an ongoing need for the assessment and management of this particular class of UK building.

Traditionally assessments of robustness 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 blocks by other dwellings. This situation was creating unnecessary expenditure, social upheaval, environmental impacts etc.

BRE has believed for some years that LPS blocks should be stronger than the 'simplified' structural assessment calculations could demonstrate, particularly in relation to the forces due to certain types of gas explosions. However, there were no practical or experimentally verified analytical methods available to demonstrate this.



Figure 1: Damage to Ronan Point [1968]

BRE has gained experience and knowledge about the behaviour of LPS blocks from programmes of full-scale testing within three LPS blocks (these being 9, 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 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 (e.g. 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). A state-of-the art report on the methodology for assessing existing LPS blocks has been developed. This includes several advanced methods based upon BRE LPS load testing experience. A high level of expertise and knowledge has been developed to correctly implement these methods. These have been found to be valuable in justifying the retention of existing block, typically with a substantially smaller degree of strengthening and related works than contemporary procedures are able to justify.

Keywords: Concrete Panel Building Testing, LPS Buildings, Finite Element Modelling, Full-Scale Structural Building Test, Accidental Loading, Gaseous Explosion, Alternative Load Paths, Structural Assessment.

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BACKGROUND AND PREVIOUS WORK

Following the publication of the recommendations of the Tribunal of Inquiry into the Ronan Point collapse [1], the then Ministry of Housing and Local Government issued advice to UK Local Authorities owning LPS blocks via Circulars 62-68 and 71-68 [2]. The advice was to appraise all their LPS blocks of 7 storeys and above for their susceptibility to progressive collapse. Owners were required to consider whether strengthening was necessary. This was to be assessed on the basis of either:

- assuming the removal of a critical section of the load-bearing wall (the length between returns) and being able to demonstrate that alternative paths of support could be mobilised to carry the load, or by
- ensuring that the form of construction was sufficiently strong to resist the forces liable to damage the load-supporting members.

Engineers were required to check whether the structure was able to accommodate a static overpressure of 5lb/in² where a piped gas supply was present, or 2½ lb/in² where it was not. The metric equivalents of these overpressure criteria are 34 and 17 kN/m², respectively. It appears no consideration was given at that time (1968) to the risk or implications of failure of one or more floor panels (i.e. vertical progressive collapse of the floors or removal of lateral support to the vertical load-bearing walls). The Inquiry also warned about the risks of storing explosive substances, such as liquefied petroleum gas (LPG), and this warning was repeated in Circular 62-68. Following appraisal a number of LPS blocks were subsequently selectively strengthened and / or had their piped gas supplies removed.

Figure 2 presents the changes that have taken place since 1968 with respect to UK requirements for assessment of robustness in relation to the height of LPS blocks. In 1970 provisions to resist progressive collapse, based on a 5lb/in² overpressure criterion, were introduced into the UK Building Regulations for buildings of 5 storeys and above (then each basement level was counted as one storey). It appears that the 1970 changes and the need to appraise 5 and 6 storey LPS blocks for accidental loads were not acted upon by all owners and their consultants at the time. Although this requirement was reiterated some years later when the BRE LPS report was published in 1987 [3], it was not until recently [circa 2004] this unfortunate situation was discovered and action taken to address the issue.

BRE has believed for some years that LPS blocks were stronger than the 'simplified' structural assessment calculations could demonstrate. This was based partly upon evidence from various series of tests undertaken by others within existing LPS blocks or upon components or assemblies of components carried out in the late 1960's. For example, 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 fear that appeared to exist at that time that all LPS systems in the UK might be at a similar risk of collapse. In this period BRE also constructed a quarter-scale LPS block within one of its structural laboratories to investigate tolerance to damage and progressive collapse behaviours.

BRE published its report on the structural adequacy and durability of LPS dwellings in 1987 [3]. This report defined the requirements for an appraisal, 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

Block Height (Storeys)	Ronan Point Inquiry / Local Government Circulars: 1968	UK Building Regulations: 5th Amendment: 1970	BRE LPS Report: 1987
<i>Upwards</i>	↑	↑	↑
19			
18			
17			
16			
15			
14			
13			
12			
11			
10			
9			
8	N+A	N+A	N+A
7	N+A	N+A	N+A
6	N	N+A	N+A
5	N	N+A	N+A
4	N	N	N
3	N	N	N
2	N	N	N
1	N	N	N

Notes :

N Assessment required with respect to normal loads only

N+A Assessment required for normal and accidental loads

Change of assessment requirement over time

Figure 2: Change in LPS assessment criteria

the accidental loads which might be applied to an LPS block, the possible forms of progressive collapse and the significance of connections / tying reinforcement between precast structural components. Full appraisal of a complete LPS block was recommended every 20 years. For buildings of 5 storeys and more (including basement storeys) the appraisal was to be made in respect of both normal and accidental loads.

Explosion testing [4] was undertaken by BRE in several rooms of a 3-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 previously obtained test laboratory results. 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 suggested that venting and failure of non-structural walls determined the maximum overpressures reached within each test room. The magnitude of the overpressures generated using 200ml and 750ml canisters varied, even for the same size canister and identical test set-ups. The pressures generated ranged from 2.6kN/m^2 to a maximum of 9.0kN/m^2 . The maximum lateral deflection of a wall panel recorded was 0.074mm (associated with an overpressure of 9.0kN/m^2). There was a degree of spatial variation in overpressure, which was different for each test explosion. No structural damage was reported, although damage to the non-load bearing timber partitions was significant. The explosions generated during the tests were classed as 'moderate'¹. This form of testing was useful in demonstrating the likely magnitude of 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 buildings (without a piped gas supply) when subjected to overpressures of the magnitude experienced.

Later BRE undertook a programme of static and dynamic testing [5] linked to an evaluation of the accuracy of a range of analytical techniques for calculating structural response of reinforced concrete panels to actual gas explosions. The work indicated that there were significant problems in predicting actual behaviour under dynamic loading even of simple uniform reinforced concrete panels. It was concluded that similar or greater problems would be experienced when seeking to analyse complex structures, i.e. LPS buildings.

Of particular relevance were strain rate effects, demonstrated by the different behaviours exhibited by the reinforced concrete test panels under static loading compared to that exhibited under dynamic (gas explosion) loading. Whilst the researchers appeared to have had leakage problems with the test rig during the static load tests, which limited the ultimate pressure that could be applied, the static strength was estimated to be approximately half that sustained by notionally similar panels under dynamic loading. The concrete test panels also appeared to be appreciably stiffer under dynamic loading.

OVERVIEW OF RECENT PROJECT WORK PROGRAMMES AND ASSOCIATED ACTIVITIES

However the above work did not resolve all concerns and there were no practical or experimentally verified analytical methods to demonstrate satisfactory performance available for use in the 1990's when there was a requirement for the assessment many existing LPS blocks where they were required to remain in-service.

The majority of the work reported in this paper was carried out as part of a recent project entitled "*Improving the Management of Ageing Assets by Advanced Techniques for Assessing Existing Multi-Storey LPS Blocks*". This project was carried out by a consortium brought together in 2003 by BRE including six key consulting engineering practices, a leading analytical software development company, fourteen local authorities and housing associations, a non-destructive test equipment manufacturer and a material test laboratory. The group was able to secure financial support from the UK's Department of Trade and Industry.

The paper also makes reference to the results of previous full-scale structural tests upon LPS blocks situated in Sandwell and Leeds in the UK during the late 1990's. In addition this paper also draws on work undertaken in a number of research projects by various researchers, including some dating back to the investigations carried out in the late 1960's following the Ronan Point collapse.

The main tasks undertaken during the recent projects were :

- Liaison with building owners and consultants to develop a test programme and for case studies.
- Forensic investigation of the form of construction and of build quality in the test blocks.
- Development of analytical models including the construction of 3D finite element computer models.
- Full-scale load testing of the test buildings.

¹ Structurally significant explosions in the UK are classed as moderate, severe and very severe (worst).

- Comparison of analytical models / computer modelling outputs with the load test results.
- Further development of the assessment methodology for LPS blocks.

The LPS dwelling blocks used were made available for the research prior to them being demolished as part of the redevelopment of the city areas concerned. Accordingly they were not inhabited at the time of testing. Table 1 summarises the various full-scale structural tests undertaken by BRE within existing LPS blocks. The initial tests carried out in 1997 involved separate loading of wall and floor units to assess their strength and performance, prior to undertaking room tests involving simultaneous loading of wall and floor elements.

Table 1: Nature of the various full-scale structural tests undertaken by BRE.

Nature of Test	Overpressure Floor Panel	Overpressure Wall Panel	Overpressure Room – Lounge (Long floor span)	Overpressure Room – Kitchen (Short floor span)	Element Removal
Sandwell (1997) Bison Wallframe	4	4	4	4	
Leeds (1997) Reema Conclad			4		
Liverpool (2004) Bison Wallframe			4		4

INVESTIGATIONS TO ASSESS BUILD QUALITY - LIVERPOOL BISON WALLFRAME LPS

The quality and consistency of workmanship within the structural joints opened up and examined by BRE were, in the main, found to be reasonably good. Localised investigations of some kitchen and lounge floor slabs indicated that the quality of workmanship achieved within the spine wall / floor slab joints was variable. In these locations significant areas of in-situ concrete were missing or poorly compacted and levelling dowels had been cut off and / or bent to aid construction. Similar examples of poor workmanship have been observed in other Bison blocks previously inspected by BRE. In summary, the quality of construction appeared to broadly comparable with that observed in other Bison Wallframe blocks generally and, more specifically, with that in the Sandwell Bison Wallframe block load tested by BRE in February 1997.

COMBINED WALL AND FLOOR (ROOM) TESTS - LIVERPOOL BISON WALLFRAME LPS

Two combined wall and floor load tests were undertaken in the lounge of a flank wall flat situated two storeys down (i.e. 14th storey) from the roof of the 15-storey block. The first test was conducted with all the mechanical connections within the cross and flank wall / floor panel joints intact. Following the successful completion of the first load test, two of the mechanical connections within the cross floor / wall panel joint were cut. The floor and wall test loads were reapplied up to the maximum achieved during the first load test. The aim of this phase of the first test was to replicate the situation during construction where the projecting floor loops had not been correctly placed over the associated wall dowels (i.e. a construction defect).

The loading and instrumentation systems were identical to those used for the previous load tests undertaken in the Sandwell Bison and Reema Conclad blocks. Figure 3A presents a schematic layout of the hydraulic wall loading rig, including the associated safety systems. Figure 3B shows a picture of part of the assembled horizontal and vertical loading system forming the rig used for a combined wall and floor test. This rig, with associated instrumentation for measuring the response of the building, took about one week to assemble within the test room. The instrumentation was mounted on an independent reference frame to ensure a picture of global structure movements was obtained. All pre-existing (1960's) strengthening steelwork local to the test sites was removed prior to the BRE tests being conducted.

The principal goal for the loading system was to replicate, as far as was practical and economic, the maximum mid-span bending moments & shear forces arising from a uniformly distributed pressure produced by a gas explosion on the boundary of the enclosing volume. For safety reasons it was decided to adopt a loading scheme utilising hydraulic jacks, adjustable lightweight loading shores and hollow rectangular steel box beams (for load distribution) fixed to the elements under test. The applied loads were applied simultaneously to the wall and floor elements in the test room. Separate hydraulic circuits were employed to allow different loads to be applied to the shear and bending load distribution beams. Thus in a combined wall and floor test four separate hydraulic circuits were used. The magnitude and rate at which the loads were applied was controlled by four manually operated hydraulic pumps, again for safety reasons. The

loads applied were proportioned primarily on the basis of one-way spanning elastic behaviour. However, the applied shear load was uprated to compensate for the anticipated degree of two-way spanning behaviour.

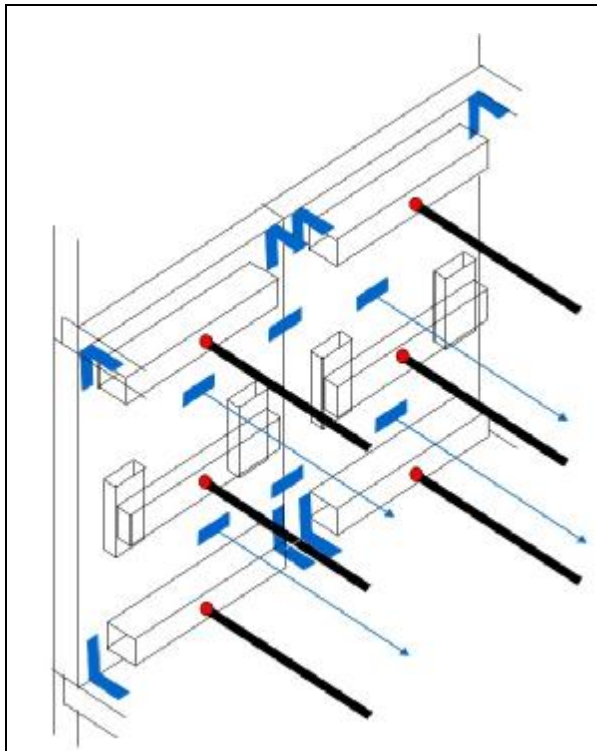


Figure 3A: Schematic of hydraulic wall loading rig



Figure 3B: Hydraulic room (wall & floor) loading rig

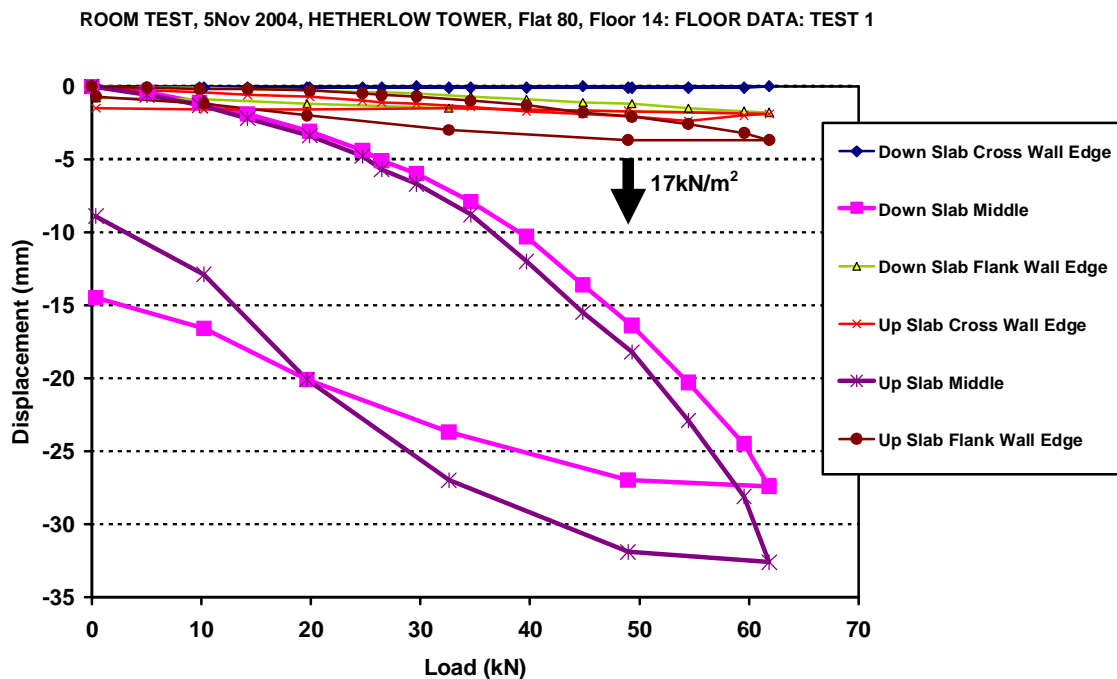


Figure 4: Liverpool Bison Wallframe overpressure room test: Floor panel displacements: Test 1

Several forms of safety protection (refer Figure 3) were provided which could be utilised during or after the overpressure test to prevent excessive movement of the (flank) wall panels or floor slabs in the event of a premature failure of any of the components or joints under test. These included items such as:

- Temporary scaffolds (with adequate clearance) to arrest excessive deflection of the floor slabs or walls.
- Horizontal tie rods (loose during test) to prevent excessive lateral displacement of the flank wall.
- Steel fixing angles and plates (loose during the test) shown shaded in Figure 3 - the fixing bolts were tightened up after the load test to allow panel & wall / floor joints to be left in a state of known connectivity.

Figure 4 presents the results obtained from the first room overpressure test (with mechanical connections between floor and wall panels intact). It will be seen that a load in excess of an equivalent uniformly distributed pressure of 17kN/m^2 was sustained. Both tests demonstrated that the wall and floor panels were able to accommodate the minimum specified notional overpressure loading of 17kN/m^2 without gross distortion of the panels or the accompanying joints. This was with all the cross wall / floor joint mechanical connections intact (test one) and subsequently (test two) with half the connections cut away by the test crew.

The results of the Liverpool combined wall and floor panel tests are broadly consistent (the floor spans were different in the two Bison Wallframe blocks tested) with those obtained from nominally identical tests undertaken in the Sandwell Bison Wallframe block. In reviewing the results from the tests in the two Bison blocks we are of the view that unless there are gross deficiencies in the floor panel reinforcement / tying provision and / or a very low concrete compressive / tensile strength, that Bison Wallframe blocks should in general be sufficient strong to accommodate the forces applied to the wall and floor panels bordering the site of a single room non-piped explosion (with a maximum notional overpressure of 17kN/m^2).

SPINE WALL TESTS - LIVERPOOL BISON WALLFRAME LPS

The second form of load test undertaken was aimed at exploring the response of the LPS block to the simulated loss of a load-bearing spine wall. This wall was located between the kitchen and lounge in a flank wall flat situated at first floor level. This test was included in the test programme to enable the modelling / calibration of a different form of behaviour to that examined during the first set of load tests, and because of existing concerns over the potential behaviour of the structure in the event of the loss of this type of wall. Such a loss, due to base shear failure and / or flexural failure of the kitchen / lounge spine wall, is predicted to occur during a severe non-piped gas explosion.

The spine wall supports one end of each kitchen floor slab in the flat 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. This tie normally comprised a relatively short square steel bar welded to steel channels which in turn were welded to the tension reinforcement embedded in the two floor slabs.

The spine wall test set-up consisted of several elements. A load-bearing safety structure installed adjacent the spine wall comprising twinned SGB steel soldiers, steel box (RHS) 'needles' and flat jacks. The purpose of this structure was to carry the dead load of the structure above in the event of failure of the primary jacking system or the structure failing to mobilise an alternative load path when the primary path through the spine wall was disrupted by the deliberate removal of the upper part of the spine wall by the test crew. The safety structure also allowed the dead load of the structure to be safely transferred into the primary jacking system. 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 test spine wall. An instrumentation system similar to that used during the room tests was installed within the kitchen to monitor the behaviour of the surrounding structure.

Once the safety and instrumentation systems had been installed, the top 100mm or so of the spine wall was gradually broken away using hand held concrete breakers. This then enabled the full load of the structure above to be taken by the primary jacking system. The rams of the jacks forming the primary loading system 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 top of the spine wall had been removed and the primary and safety system hydraulic jacks retracted.

Figure 5 illustrates the form of the finite element analysis (FEA) models developed by BRE together with the predicted response of the wall and floor elements bordering the spine wall under test. Considerable effort went into this activity and into risk assessment to ensure the development of rigorous protocols to control the test and allow the work to be carried out safely. As with the overpressure tests, load transfer and associated

actions had to be undertaken in a manner which ensured the safety of the test crew and could be assured of leaving the building in a known and safe condition for the following demolition contractor.

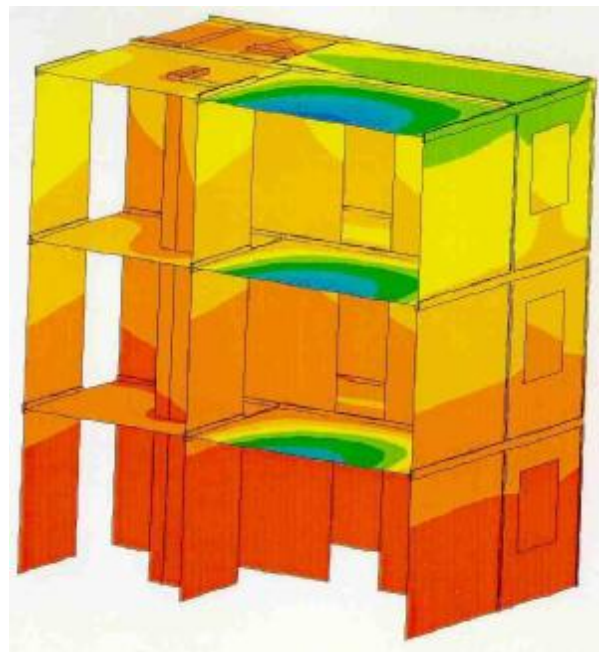


Figure 5A: Vertical deflections - spine wall present

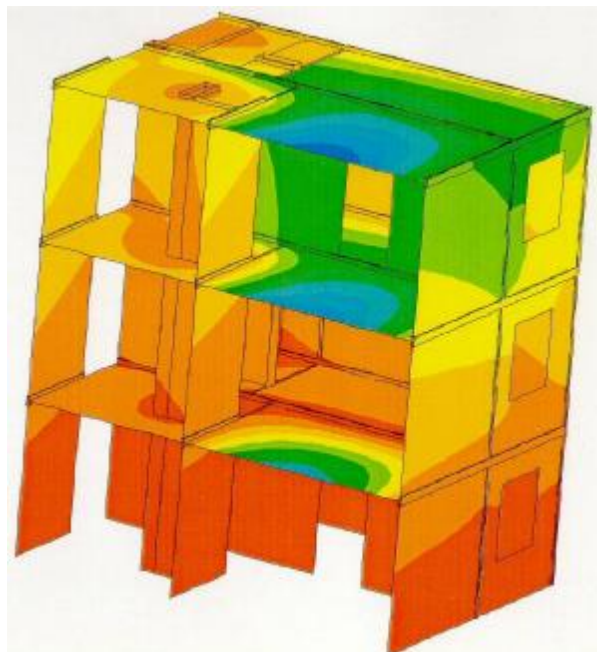


Figure 5B: Vertical deflections - spine wall removed

The test clearly demonstrated that the section of the building situated above a damaged or 'missing' spine wall was able to develop alternative load paths. This mechanism should enable the structure to bridge over areas of damage which might occur during a severe gas explosion within the lounge or kitchen to the flank wall flats in the Liverpool Bison Wallframe block.

Accordingly, we concluded that that building would not be expected to suffer disproportionate damage in the event of the loss of a spine wall panel between the kitchen and lounge in a flank wall flat, arising from a non-piped gas explosion. This assumes that there would not be too much disturbance to the remainder of the adjoining structural elements from such an explosion. The results of previous overpressure tests suggest that this should be a reasonable assumption.

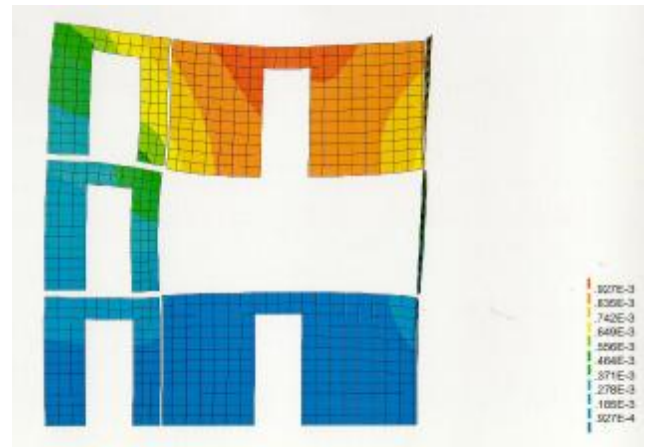


Figure 5C: Vertical deflections - spine wall removed
Note this plot uses a different legend to previous plots.

FINITE ELEMENT ANALYSES (FEA) & CALIBRATION EXERCISES - LIVERPOOL LPS

Non-Linear Finite Element Analysis and Calibration Exercise: Lounge Floor Slabs

Both 'shell' and 'solid' elements were used for modelling the Bison lounge floor slabs. Whilst 'solid' elements are the most appropriate type for accurately modelling the complex geometry of the floor slabs, their use produces large systems of equations which required appreciable processing power. Due to restrictions in the computing resources available, it was necessary to use 'solid' elements for a limited number of analysis runs. Some runs were repeated to facilitate comparison of the accuracy of models constructed using 'shell' element and those constructed with 'solid' elements.

'Layered' shell elements, as opposed to 'Normal' shell elements, were employed as they are better able to describe the ultimate behaviour of the reinforced concrete components. The reinforcing bars were modelled as embedded, i.e. reinforcement elements that lie within the boundaries of the basic elements. Both

smearing and discrete reinforcement methods were evaluated. The discrete bars were positioned in accordance with our understanding of the layout of the lounge floor reinforcement established through both invasive and non-invasive investigations undertaken previously by BRE.

Material Behaviour and Material Properties: Lounge Floor Slabs

The most likely non-linear effects were taken into consideration when building the FE models of the reinforced concrete floor. These included multi-axial compressive response and tensile cracking of concrete, together with 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 was found to be dominated by tensile cracking. Table 2 below presents a summary of the range of material properties used and assumptions made during the calibration exercise.

Table 2: The range of material properties used during the calibration exercise.

Material Properties	Value or Lower / Upper Bound Values Adopted
Characteristic compressive strength of concrete	30 MPa
Young's modulus	31.8 GPa
Compressive failure strain	3.5 mm/m
Tensile strength of concrete	2 to 4 MPa
Tensile ultimate strain	0.002
Shear retention factor	0.2 – 1.0
Reinforcement yield stress	355 to 470 MPa

Summary of Results of Calibration Analyses Exercise: Lounge Floor Slabs

BRE carried out in excess of 50No. modelling runs using the two element types and a range of input parameters representative of material properties and behaviours. Clearly it was impractical to explore the effects of all possible combinations of input parameters and the two selected element types on the predicted structural response. Engineering judgment was used to select the most appropriate combinations of input values and assumptions with respect to material behaviour for those modelling runs that were undertaken. Once the first analysis was completed we reviewed the changes required in the input parameters / assumptions to reduce the error between the measured and predicted structural behaviours. Appropriate adjustments were made to selected parameters and the model re-run.

The side elevation wall to the lounge in which the full-load test was carried out comprised timber studding faced with plywood with full width half-height glazed windows. Access to the adjoining balcony was gained via a fully glazed timber door. It was therefore important to explore what effect, if any, a non-load bearing glazed timber partition would have on the predicted behaviour of the floor slab under accidental loading (see Figure 6). An idealised version of the partition was then introduced into the model during some of the calibration runs.

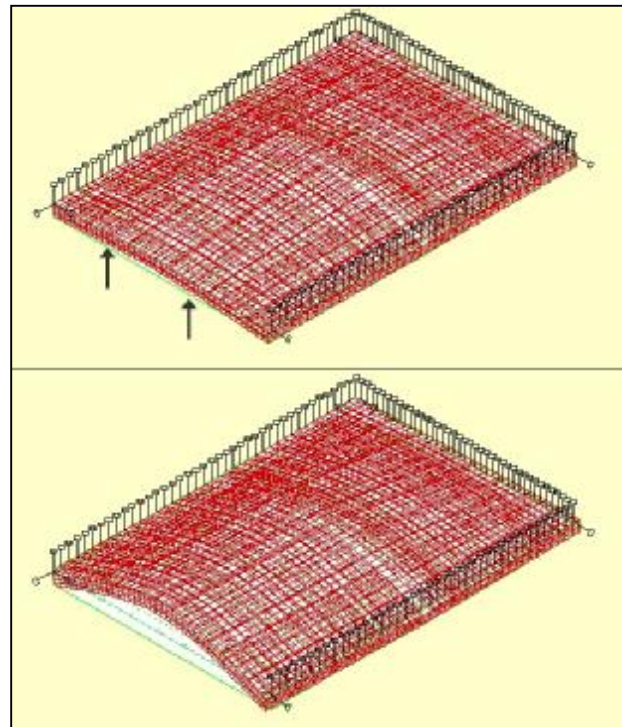


Figure 6: Influence of edge support on performance of a lounge floor slab

Summary of Results of Calibration Analyses Exercise: Spine Wall

Linear FEA was undertaken prior to the actual load tests to provide guidance on the likely behaviour of the structure. In particular whether the building was likely to develop alternative load paths and hence bridge

over the missing spine wall, or to suffer some form of failure and gross downward displacement / collapse.

The results of the modelling runs, which employed a number of different element configurations and sought to replicate expected variations in joint behaviours, indicated that the downward vertical displacement of the spine wall was likely to be less than 1mm. This appeared to be regardless of the variations in the elements or their respective parameters, that had been adopted. Accordingly, the FEA suggested that the structure should be able to bridge over the 'missing' spine wall, which it did with deflections of the order predicted.

PROCESS OF ASSESSMENT FOR ACCIDENTAL LOADS

Appraisal of Existing LPS Blocks for Accidental Loading or Damage

The assessment of a structure is 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 current and 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. Various definitions related to these activities are given in the Annex.

For LPS blocks the approaches given in the guidance for assessment of structures for accidental loads is initially to review compliance with recommendations for the provision of vertical and horizontal ties in structures. Only if these requirements are not satisfied, to then try and demonstrate that the LPS components and joints are strong enough to resist the forces associated with the anticipated accidental loads. In most cases these relate to the forces created by gaseous explosions. A series of increasingly sophisticated analysis techniques may potentially be utilised to try and demonstrate that the LPS components are likely to be sufficiently strong. In most instances the performance of joints between components can only be evaluated by reference to the performance of similar joint details during load tests.

The overall process of assessment might involve a number of steps from the following outline procedure:

- Undertake a review of existing documents and information; record findings and other outcomes such as the need to obtain additional information. This is desk study, but would involve an initial walk-around inspection of the buildings concerned. An important aspect is to understand how the Owner has managed the building, what works may have been undertaken and what future plans there may be in this regard. It will be necessary to work closely with the Owner / Owner's team to establish these details.
- Consider the hazards to which the particular buildings might be exposed. Seek to make an evaluation of the risks and the requirements for the assessment, such as normal and accidental loads or other potential threats:
 - Piped gas supply ?
 - Prospect of vehicular impact (road vehicles, trains, planes etc) ?
 - Nearby chemical manufacturing or storage site ?
- Undertake required inspection and testing (site investigations, material strengths, etc) to gather any additional information required or deemed necessary by hazard identification. The scope of these works will depend upon findings of the initial review of available documents. As a minimum this stage is likely to involve some degree of on-site validation of the available technical information / records.
- Undertake evaluation of construction of selected joints etc. Expensive and disruptive physical investigation activities are often carried out in a phased manner for a number of reasons. This may be so that the focus of later stages can be adjusted to account for the findings of earlier phases, or perhaps to control or minimise cost and disturbance to residents, perhaps in order that a hypothesis driven approach can be adopted instead of a random sampling methodology, etcetera.

Having gathered the required information, it has then to be evaluated in order that decisions can be made about the adequacy of the structure. This may require consideration of various factors including normal and accidental loads or damage, deterioration of structural and other components, current and / or future performance, as well as durability & service life aspects. Evaluation and calculation methodologies could include a wide range of techniques. Usually evaluations are undertaken initially using a simplistic approach, with increasingly sophisticated procedures only being adopted as required to resolve particular aspects or to verify that the performance of specific elements / components is satisfactory. This is a common engineering methodology and could involve any of the following:

- Spreadsheet calculations (Simplified one-way spanning for each member type).
- Summary engineering judgement of potential behaviours based upon experimental performance.
 - Floors & walls : Upward and downward loading / Reversal of loading

- Shear & flexural behaviour of components
- Advanced non-linear finite element analysis (typically investigating performance under circumstances envisaged to be reasonably representative of upper and lower bound conditions).
- Probabilistic assessment of loadings and evaluation of safety risk.

Usually it is only when the relevant assessment avenues have been exhausted that consideration is given to the adoption of structural strengthening measures. These are usually local in nature and designed to resist specific forces associated with a gaseous explosion or other accidental load / damage. Otherwise post-damage or post-failure behaviour has to be considered and usually this is rather difficult to quantify with any degree of confidence and is generally a zone of structural behaviour that engineers will seek to avoid.

Strengthening: Various Approaches, Options and Philosophies

A first objective is usually to avoid the initiation of failure, such as stopping the potential lateral translation of the foot of a wall panel under lateral forces due to a gas explosion. If this is not adopted consideration might then be given to the creation of alternative load paths assuming that some form of failure mechanism has been initiated. For example:

- Floors might fail : Either under upward or downward loading actions, which might result in:
 Debris loading.
 Lateral disturbance of the alignment of load bearing walls.
 Removal of lateral restraint to load bearing walls.
- Walls might fail : With a need to route gravity loads around lost members, noting that:
 Horizontal ties tend to engage more of the structure & so spread damage laterally.
 Vertical ties or supports (columns) can provide support, holding up lower elements.
 The upper levels in an LPS building are most at risk due to the lower gravity loads.

The presence or not of a piped gas supply clearly has a major influence upon hazard, consequences and risk.

Way Forward: Issues of Risk and its Perception

A rational risk or probabilistic approach is thought to be desirable and the most promising way forward, which will require a revision of current UK assessment guidance for LPS dwelling blocks. This will require a better understanding of engineering aspects, psychological influences and also cultural - society & press

In engineering situations the concept of risk is commonly expressed by an equation in the following form:

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

However what levels of risk are acceptable to society and what value system / data applies to judgements which have to be made by informed persons, and what baseline criteria and values does society assign for these judgements? It is also true that perceptions change with time. It is clear that the influence of events such as the World Trade Centre (WTC) collapse (Sept 2001) following the terrorist attacks will last for many years to come, as do other significant events such as major explosions or fires at chemical manufacturing or storage sites. Indeed the collapse at Ronan Point led to the definition of new structural performance requirements in the form of 'robustness'.

A new BRE Report currently in preparation and having the working title of *State-of-the-Art Report on the Assessment of Large Panel Buildings Under Accidental Loading* [6 - in press] has sought to draw together the related technical aspects within the wider societal context to provide a more rational approach than that offered by MHLG Circulars 62-68 and 71-68 [2]. These documents although published almost 40 years ago are still in general use by the structural engineering profession as the basis for undertaking assessments of LPS blocks. The new document seeks to recognise contemporary thinking on matters of hazard identification and related risk, making a review of the various sources of risk and the means by which they may be managed, and modern technical approaches to the assessment of existing structures. Some of these are still in the process of development and research is ongoing in respect of probabilistic methods.

The currently available assessment techniques, decision processes and potential outcomes are illustrated in Figure 7, which is taken from the forthcoming BRE report [6].



Figure 7 : Part A: Flow diagram for assessment methodology decision process and associated alternative outcomes and potential actions



Figure 7 : Part B: Flow diagram for assessment methodology decision process and associated alternative outcomes and potential actions

CONCLUDING COMMENTS

The BRE tests involved detailed forensic investigation of the buildings, prior to programmes of structural load tests in which elements / joints between elements were loaded to failure. Loading was applied by a hydraulic test rig to walls and floors simultaneously to simulate an overpressure from a gaseous explosion. In some locations repeat tests were performed with damage to connections being introduced between different tests in the series. Although the load transfer had to be carried out slowly to ensure the safety of the test crew and the following demolition contractor, 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.

The current understanding of LPS performance under a simulated overpressure has been built up from tests in three LPS blocks of pre-Ronan Point construction, two being of Bison Wallframe design and one being a Reema Conclad design. Although the blocks were found to be constructed to a reasonable standard of workmanship, they were all found to contain various forms of construction defects. Overall the blocks tested are thought to be reasonably representative of the wider population of LPS blocks.

Structural assessments by BRE of Bison Wallframe LPS blocks had consistently shown that certain types of load-bearing 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 remaining above the test location.

Advanced structural assessments were undertaken using sophisticated analytical methods, including non-linear finite element analyses, to guide the experimental programme, to ensure that the proposed activities could be undertaken safely, and to assist in the interpretation of the results obtained.

In summary, this work successfully demonstrated that the blocks concerned should be able to:

- resist the specified 17 kN/m² overpressure loading for a building without a piped gas supply, and
- mobilise an alternative load path after the removal of a particular type of load-bearing wall element.

A revised framework for the assessment of existing LPS dwelling blocks has been prepared. The new state-of-the-art document [6] seeks to recognise contemporary thinking on matters of hazard identification and related risk, making a review of the various sources of risk and the means by which they may be managed, and modern technical approaches to the assessment of existing structures. Some of these are still in the process of development and research is ongoing in respect of probabilistic methods.

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ANNEX: DEFINITIONS

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 current and future service may be established against specified performance requirements, loadings, durability or other criteria. This activity may include prognoses about future condition and performance.

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.

Deterioration 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.

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.

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 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 of the structure.

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 functionality at the level anticipated by the designer.

Monitoring To keep watch over, recording progress and changes with time; possibly also controlling the functioning or working of an entity or process. Structural monitoring typically involves gathering information by a range of possible techniques and procedures to aid the management of an individual structure or class of structures. It is often taken to involve the automatic recording of performance data for the structure and possibly also some degree of associated data processing. Strictly this does need to be so, there being a variety of means of gathering appropriate data including physical testing and non-destructive techniques.

Robustness The ability of a structure subject to accidental or exceptional loadings 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 (number of potential alternative load paths) which could be mobilised within the structural system concerned.

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 Procedures whereby 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.).

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