FIRNTEC BUILDING COMPLIANCE

VISUAL STRUCTURAL APPRAISAL

Kynaston House

Rydding Lane
West Bromwich
B71 2HD

3rd September 2025

Unit 3,
Endeavour House,
Longbridge Road,
PL6 8LR

Document Issue

Revision	Date	Purpose of Issue
A	25.07.25	Issued for Review
B	28.07.25	Final Issue
C	03.09.25	Issued with floorplan

Prepared By

Ruben Wood , Meng CEng MIStructe

Dated: 03/09/2025

Reviewed by

Firntec Project Consultant

Dated: 03/09/2025

Competency Statement

Ruben has a thorough range of experience on projects both in the UK and globally, centered on the construction industry but also in public art installations and bespoke structures. His extensive experience in a wide range of projects afford him the contacts to bring together the right team for a job. Ruben has worked on a range of projects where he has developed the ability to perform conceptual and detailed designs, analysis of complex structures and element design. He has extensive experience in structural design and construction methods and has successfully completed projects with values in excess of £10m.

Ruben has also been engaged in structural appraisals of high rise residential structures across the UK and has been assisting Principal Accountable Persons in assessing their housing stocks and compiling information to be included in the golden thread as a requirement of the Building Safety Act.

QUALIFICATIONS, MEMBERSHIPS & TRAINING

First Class MEng Civil Engineering - University College London

Chartered Engineer and Member - Institute of Structural Engineers CEng MIStructE

Knowledge of the Engineering Council's competency register for HRB's as set out in UK-SPEC for HRB

Continued professional and development through industry led courses and webinars on The Building Safety Act, fire in structural engineering and assessments of existing HRB's.

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1. Executive Summary

The building survey revealed that the overall structural integrity of the property is sound. No major structural issues were identified that pose an immediate threat to the occupants or the building's stability.

Is the Building Structure Safe to Occupy	**Yes but further investigations required**

Kynaston House is a Large Panel System(LPS) building. The primary concern in LPS structures is their susceptibility to disproportionate collapse in the event of a gas explosion. The resistance to disproportionate collapse is achieved through a combination of key element design under accidental loadings and adequate tying provision between structural wall and floor panels. Note that Kynaston house does not have a piped gas supply.

Intrusive investigations are required to properly assess the building's health, condition and fire resistance. The actions for the building are:

Ref no	Observations	Risk Rating	Mitigation	Timescale
Kyn 001/ 2024	Concrete material tests for carbonation, presence of chlorides and HAC	Low	Engage a qualified specialist contractor to complete the investigations	within the next 24 months
Kyn 002/2024	The fire resistance of the structural elements can't be confirmed without knowing the depth of reinforcement within the panels.	Due to the LPS construction and fire being a primary cause of loss of strength in a structural member this poses a Medium/High risk	Concrete cover tests to the wall panels are required. Engage a qualified specialist contractor to complete the investigations	Within next 12 months
Kyn 003/2024	We have been provided with statements on a BRE report which states that the robustness tying requirements are met and that the remedial ties	As this relates to disproportionate collapse, a known issue in LPS buildings, this poses a medium/high risk	Engage a suitably qualified contractor and engineer to complete checks to the ties in at least 25% of the units	Within next 12 months

	have been installed. However, no evidence is supplied to support their form and presence. Note multiple requests have been made to the BRE by the PAP requesting the report. It is not available.			
Kyn 004/2024	Regular maintenance is crucial to preserving the property's condition and preventing future issues. The building is to be surveyed at regular 3 year intervals to inspect for signs of movement, structural decay and/or defects.	Low	Engage a qualified specialist contractor to complete the surveys	Every 3 years

2. Introduction

2.1.Brief

Firntec were appointed by Sandwell to perform a visual structural appraisal of the purpose built residential block known as Kynaston House, B71 2HD. The instruction was to identify the key structural elements of the building and is prepared to be included as part of the Building Safety Case Report for the housing block. The inspection relates to the main building only and any external fences/walls, sheds and outbuildings/garages have been excluded from our scope. An inspection of the property was undertaken by Firntec on the 12th June 2025. The scope of the inspection included limited, visual observations of the interior and exterior of the structure. The inspection was limited to accessible areas which included the external elevations (viewed from the ground), communal areas, stair core, plant rooms and the roof.

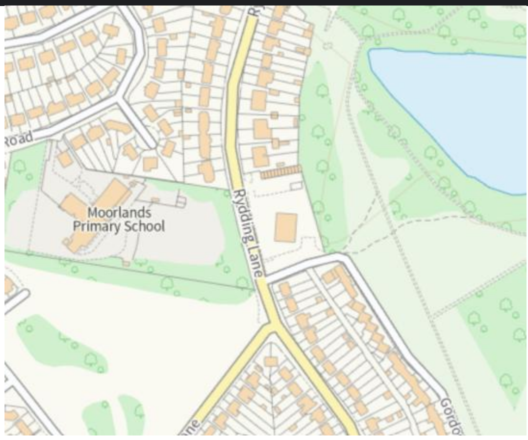

Figure 1: Site location

2.2 Limitations

This report is based upon a visual inspection of the property, as described above. This report describes the findings and draws conclusions of a general nature and is not an intrusive survey of the building and no calculations were carried out with regards to load transfer and verification of structural members.

Whilst comments are made to satisfy the requirements of the brief, the report has, of necessity, not been exhaustive and cannot therefore constitute a warranty as to the soundness or otherwise of the property in areas hidden from visual inspection or not seen during the inspection. No internal finishes were disturbed during our inspection, and no opening up works or invasive inspections were undertaken. We were not able to observe any structural elements where hidden under finishes.

This report has been prepared for the sole use of the client Sandwell.

Disassembly or removal of any portion of the structure, mechanical equipment, plumbing equipment, or electrical equipment is beyond the scope of this inspection. There is no warranty or guarantee, either expressed or implied, regarding future performance, life, insurability, merchantability, workmanship, and/or need for repair of any item inspected. Please note that any liabilities associated with Asbestos including surveys, identification and removal have been excluded.

The components of the property included in the scope of the inspection, if present and applicable, include: structural foundations, primary load-carrying framing members, roof surface, façade structure, and miscellaneous items related to the property. The scope of the inspection is specifically in relation to structural elements and hence general building defects which are not structurally significant may not be included.

2.3 Survey Methodology

Initially a desk top study was completed through research of online records and available archive information. The desk study considers site-specific historical, geological and environmental characteristics such as the expected soils condition and if the site is within a flood zone. It also includes a review of the existing available information provided by the client. From this we are able to determine the expected construction type and materials used and plan the survey accordingly.

Once the nature of the site and surrounding areas has been recorded the more detailed inspection of the building can take place. This includes an appraisal from the inside and outside of the building. A visual appraisal is completed on every level with a more detailed assessment completed at intermittent floors, always including roof level, top floor and ground floor.

If archive drawings are unavailable the structural form of the building is determined by searching for the areas where the underlying structure is visible such as plant rooms, bin stores or service cupboards.

The integrity of the building is assessed searching for signs of structural movement, material decay and inspecting the condition of the structural elements.

Below is a list of completed actions for each area inspected:

ACTION	POINTS CONSIDERED
INSPECT ROOF AREA	• Check for signs of cracking at junctions • Check levels • Check for obvious defects
CHECK FINISHES IN FLAT/COMMUNAL AREAS	• Review each area for damages to finishes which may be caused by structural movement
SEARCH FOR CRACKS IN STRUCTURE	• Search for cracks in structure, in particular at junctions between elements • Are cracks penetrating structure • Follow the trail of the crack to assess cause
CHECK FLOOR/CEILING LEVEL	• Sagging ceiling or unlevel floors may indicate deficient design or overloaded floor plates
INSPECT CUPBOARDS	• Inside cupboards underlying structure often visible • Finishes may not have been updated and therefore provide insight into original features
ASSESS CLADDING FIXINGS	• Search for access to view cavity • Search for defects of failed wall ties such as bulging walls, cracking and separation of window reveals • Check for lifting/sagging of lintels
CHECK OPERATION OF DOORS/WINDOWS	• Search for windows and doors which are out of square, binding, cracks in the architraves
SEARCH FOR MATERIAL DEFECTS	• Identify materials used and known issues with material
INSPECT PLANT ROOMS AND SERVCE ROOMS	• Areas often without finishes revealing underlying structure • Often structural form and key elements visible
INSPECT EACH ELEVATION EXTERNALLY	• Search for cracks and/or signs of structural movement
IDENTIFY OVERHANG STRUCTURES	• Search for overhang structures • Inspect for visible defects
ASSESS STAIR CORE	• Often shear core providing stability • Stair form (precast/in-situ)
INSPECT GROUND FLOOR	• Search for vents to determine if floor is ventilated / suspended • Check levels

Table 1: Site Survey Actions

Reference shall be made to Table A of BRE Digest 292 "Cracking in Buildings" 2nd Edition (2016)

Table A: Classification of damage and repair for different crack widths

Category of damage	Description of typical damage (*Ease of repair in italics*)	Approximate crack width
0	Hairline cracks of less than about 0.1 mm width are classed as negligible.	Up to 0.1 mm
1	Perhaps isolated slight fracturing in building. Cracks rarely visible in external brickwork. *Fine cracks of up to 1 mm width can be treated easily using normal decoration.*	Up to 1 mm
2	Cracks not necessarily visible externally. Cracks can be filled easily. *Redecoration probably required. Recurrent cracks can be masked by suitable linings.* Some distortion to doors and windows, which may stick slightly. *Some external repointing may be required to ensure weathertightness.*	Up to 5 mm
3	Doors and windows sticking. Service pipes may fracture. Weathertightness often impaired. *The cracks will require some opening up and can be patched by a mason. Repointing of external brickwork and possibly a small amount of brickwork to be replaced.*	5–15 mm or several, each up to 3 mm
4	Window and door frames distorted, floor sloping noticeably*. Walls leaning or bulging noticeably*. Some loss of bearing in beams. Service pipes disrupted. *Extensive repair work involving breaking-out and replacing sections of walls, especially over doors and windows.*	15–25 mm, depending on number
5	Beams losing bearing, walls leaning badly and requiring shoring. Windows broken with distortion. Danger of instability. *This requires a major repair, involving partial or complete rebuilding.*	Usually greater than 25 mm, depending on number

* Local deviation of slope, from the horizontal or vertical, of more than 1 in 100 will normally be clearly visible. Overall deviations in excess of 1 in 150 are undesirable.

Table 2 – BRE Digest 292 Table A

2.4 Potential Material and Typology Defects

Material defects in concrete and LPS buildings can have various causes and can affect different components of the structure.

<u>LPS Defects:</u>

LPS blocks utilize mechanical fastenings, typically comprising a blend of embedded and exposed reinforcement, crafted into pins and loops that are interwoven during construction. These joints were formed on site and it was unclear to designers of the time how well the panels needed to be tied together. Poor workmanship and lack of supervision created a situation where the effectiveness of the connections between panels was not known and often deficient. This is a major issue for LPS structures, which unlike brickwork and cast-in situ RC, the integrity of the building was dependent on those joints.

These joints are later shielded with a concrete infill to offer a measure of resilience against unforeseen local failures. To resist lateral pressure, dry pack, consisting of unreinforced concrete, is commonly positioned beneath wall panels. Nevertheless, the absence of these fastenings and dry pack is attributed to inadequate workmanship and supervision.

After the disproportionate collapse of the LPS building Ronan Point, in 1968, it was recommended that all LPS buildings be modified to improve panel fixity between the floors/walls against the effects of a local explosion and/or fire. This involved the removal of piped gas supplies and the installation of restraint angles to the inner faces of the façade walls where the panels abut floors.

Recent checks on LPS buildings (Refer to IStructE Compendium of HRB's) indicates that many of the buildings were not strengthened for disproportionate collapse, therefore didn't follow the recommendations from the Ronan Point Enquiry. These same buildings may also have

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inadequate slab cover for fire resistance, due to a poor understanding of concrete fire resistance in the codes at the time. This combined weakness to disproportionate collapse and fire, which could precipitate a collapse, is considered carefully on a case-by-case basis.

Concrete Defects:

1. Honeycombing: Incomplete filling of the concrete mix, leading to voids or honeycomb-like structures within the concrete.

2. Cracks: These can occur due to various factors, such as shrinkage, settlement, overloading, temperature changes, or inadequate reinforcement.

3. Spalling: Concrete surface peeling or chipping off due to freeze-thaw cycles, corrosion of embedded reinforcement, or poor quality concrete.

4. Efflorescence: White, powdery deposits on the surface caused by the migration of soluble salts to the surface and their crystallization.

5. Alkali-Aggregate Reaction (AAR): Also known as "concrete cancer," it's a chemical reaction between alkaline cement paste and certain reactive aggregates, causing cracking and deterioration.

6. Corrosion of Reinforcement: Exposure to moisture and chloride ions can lead to corrosion of the steel reinforcement, weakening the structure.

7. Shrinkage Cracks: These cracks occur as the concrete dries and shrinks, especially if curing was not adequately done.

8. Carbonation: Carbon dioxide reacts with the alkaline compounds in concrete, lowering the pH and potentially affecting the passivation of steel reinforcement.

9. Presence of High Alumina Cement(HAC): HAC was widely used in the manufacture of structural pre-cast concrete in the 1960’s as it accelerated curing. However HAC is found to be prone to a conversion process which could result in the concrete being reduced in strength.

3 Desk Top Study

3.1 Existing Information

To date we have been provided with schematic floorplans for the building and a Fire Risk Assessment.

We have also been provided with a BRE report for a different Sandwell LPS block dated 13th March 2009 titles "Assessment of Structural Adequacy and Durability, Russell House". This is a comprehensive study into the resistance to disproportionate collapse and is attached here for reference. Within the report the following statement can be found on page 11.

> BRE was invited by Sandwell MBC in early 2008 to undertake an assessment of Russell House in line with a previous assessment carried out by BRE on a sister block, Kynaston House (refer BRE report GI281) in 1997.

Following extensive conversations between the PAP and the BRE who searched their archives extensively in 2025 for BRE Report GI281, we understand that the above mentioned similar study conducted for Kynaston was not available. Note we are including the Russell study here not as evidence, but as guidance to scope the intrusive works which are required at Kynaston.

After requesting historical evidence that the building has had the remedial ties installed we were provided with the following statement by Sandwell on 27/09/24, which was written following queries raised by a government body in 2018, post-Grenfell:

"We have only have 2 remaining LPS blocks. Russell House and Kynaston House, both are of Bison Construction. Both blocks have also undergone major external refurbishment in recent years (2012 & 2016 respectively) .

Structurally, the blocks have had full external remedial tie installations between outer concrete panels and full external concrete repairs prior to over cladding.

Previously, both blocks were also assessed for progressive collapse in 1970s further to the Ronan Point collapse and the floor to wall connections were strengthening in response . Additionally, further to the structural assessment to Russell House carried out by the BRE in 2008 (listed below) strengthening was added between wall and floor connections to the internal spine wall between kitchen and lounge to flats due to potential progressive collapse under a gas explosion where identified by the BRE, this work was carried out in 2006, this was not required to Kynaston House due to a different configuration of the blocks layout.

We have carried out a number of structural surveys to both blocks over the years as listed below:-

<u>Kynaston House</u>

1. Bison Wall frame Panels- Wall Tie Location, Kynaston House, Wednesbury, 19th August 1985. (Industrial Research Laboratories, City of Birmingham, Engineers Department).

2. Kynaston House Phrase I study, Report on Desk Study of Structural Adequacy and the Energy Efficient Refurbishment of the Block, August 1997. (Building Research Establishment.)

3. . Report on The Visual Inspection and Concrete Testing of Kynaston House, February 2000. (High Rise Services Ltd)

4. Inspection and Investigative Testing Report on the External Elevations of Kynaston House, Ryddings Lane, August 2007,2012. (CAN Structure Ltd)"

Russell House

1. Bison Wallframe Panels- Wall Tie Location, Russell House, Wednesbury,19th August 1985. (Industrial Research Laboratories, City of Birmingham,Engineers Department)

2. Report on The Visual Inspection and Concrete Testing of Russell House, February 2000. (High Rise Services Ltd)

3. Inspection and Investigative Testing Report on the External Elevations of Russell House, Holyhead Road, August 2007& 2016. (CAN Structure Ltd)

4. Assessment of Structural Adequacy and Durability– Russell House, Wednesbury, Sandwell, 11th August 2008. (Building Research

Establishment Ltd).

We understand that the reports referred are not available at this time and thus far we have not located photographic evidence that the ties have been installed at Kynaston as stated above. We were provided however of photographic evidence of the ties at installed at Russell House, see below. We recommend these are spot checked within Kynaston also in say 25% of the units.

Figure 2: Photographic evidence of ties installed at Russell House. From archive information and historical communications we understand similar stiches between floor slabs and external walls are installed at Kynaston also, however we have no evidence. Therefore some intrusive investigations required to evidence their presence.

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3.2 Desktop Study: Ground Conditions

Our scope of works did not include intrusive ground investigations or soil testing. In absence of such data, we have undertaken a desktop study to identify the likely founding strata of the property. The British Geological Survey (BGS) produces maps which highlight the general ground conditions around Britain.

The bedrock is the 'Etruria Formation' which is a mudstone, sandstone and conglomerate. Superficial deposits sit on the older deposits referred to as bedrock.

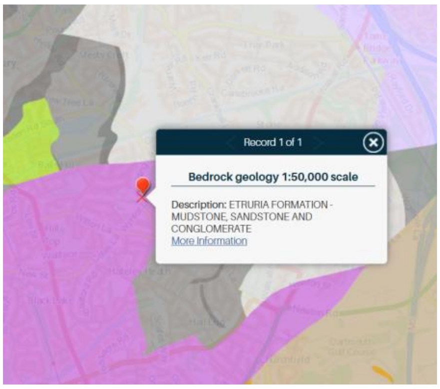

Figure 3. Bedrock Geology

For the site, the superficial deposits are an unspecified 'till'.

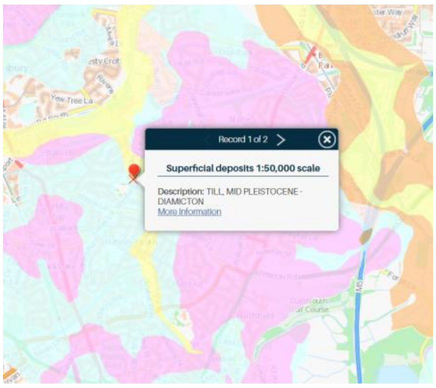

Figure 4. Superficial Geology

3.3 Desktop Study: Flood Zone .

Flood zones have been created by the Environment Agency (EA) as a starting point in determining how likely a location is to flood.

There are 3 flood zones as defined by the EA; Flood Zone 1, 2 and 3. The flood zones are based on the likelihood of an area subject to flooding, with Flood Zone 1 areas least likely to flood and Flood Zone 3 areas more likely to flood. The property is in Flood Zone 2, indicating a 0.1% to 1% annual chance of river flooding or a 0.1% to 0.5% chance of sea flooding.

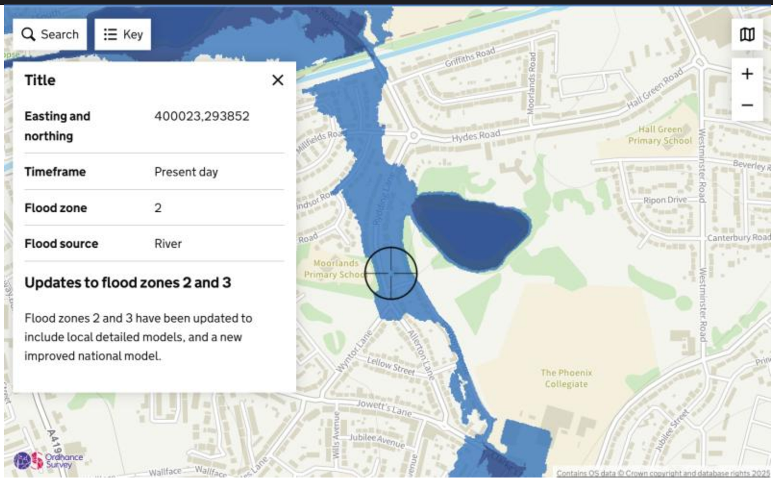

Figure 5. Flood Risk from Environmental Agency

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4 Structural Form

4.1 Description of Building

The existing building is an 11 storey purpose built residential housing block containing 82 dwellings understood to be built in circa 1961. It is therefore a pre Ronan Point block.

The building is a Large Panel System (LPS) building, and is thought to be a Bison wall frame construction system that was developed by Bison Manufacturing Ltd in the 1960's The typical structure is precast concrete panels forming the walls and floors of the structure. All internal and external walls are loadbearing, with thinner 6 inch walls used internally.

It is a Structural typology 6-Large Panel System, reference IStructE: Assessing higher-risk buildings under the Building Safety Act: a compendium of structural typologies, relevant section attached in Appendix for reference.

Given the scale of the existing structure we would expect that the main foundation system is either a series of reinforced concrete pile caps and ground beams or a reinforced concrete raft foundation. Both of these systems will have reinforced concrete piles extending into the bedrock to a significant depth.

Vertically, the building is stabilised in two orthogonal directions by the reinforced concrete wall panels. The floor slabs, which are supported off the wall heads, act as a rigid diaphragm to provide horizontal stiffness. We were unable to investigate whether there are ties present between panels, which is a critical robustness detail. We are in possession of information which states that the joints are satisfactory for the internal panels and were remediated to the gable ends with stitches between the external panels and internal floor slabs. However, evidence is missing. This is discussed further in section 4.4 and section 6.

The fire resistance of the structural frame is provided by the concrete cover of the steel reinforcement elements. The cover is unknown and requires confirmation with further investigations. According the FRA the required fire resistance is 60mins in vertical and horizontal elements.

The building has an original flat roof structure with lightweight steel framed overoof installed.

The facade is appeared to be constructed from RC panels which were overclad during refurbishment works in 2012.

Figure 6 and 7. Archive of the estate photograph from before the overclad

4.2 Structural Design Type

In order to comply with the Building Safety Act 2022 the block owner will need to register the structural type with the Building Safety Regulator. As part of the registration process the building material information is required which is indicated in Table 3.

Structure design type	
Composite steel and concrete	
Concrete large panel system - 1960's	X
Concrete large panel system - 1970's onwards	
Modular - concrete	
Concrete - other	
Lightweight metal structure (aluminium)	
Masonry	
Modular - steel	
Steel frame	
Modular - other material	
Modular Timber	
Timber	
None of these	

Table 3: Structure Design Type

4.3. Structural elements

Figures 8 indicates the assumed structural layout for a typical upper floor level.

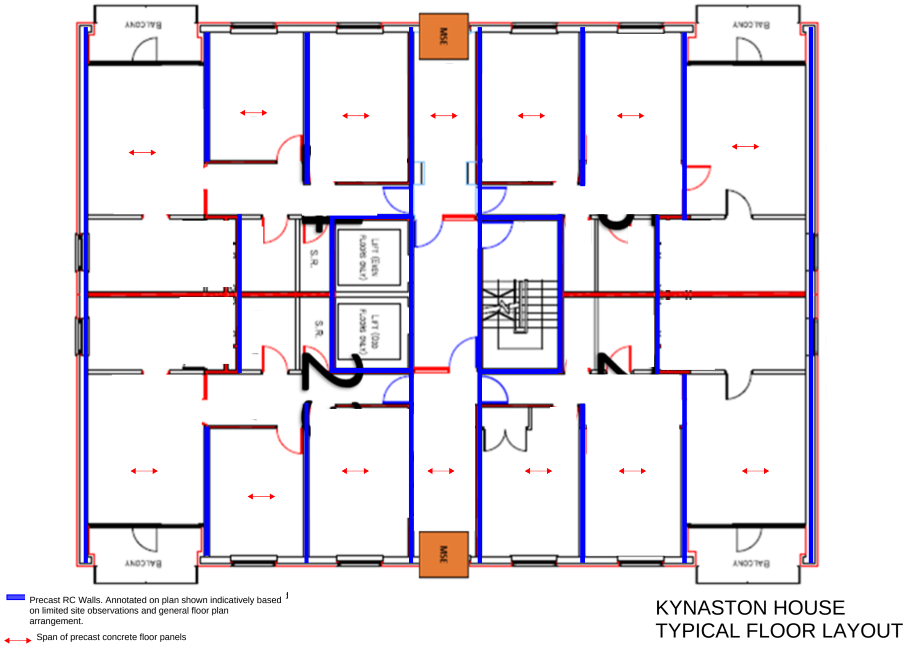

Figure 8: Diagram of typical floor plan

Table 4 Below is a summary table of the key structural elements

Item	Notes
Original and Current design intent	The building is purpose-built for residential use.
Key Materials (Are there any Precast Concrete Elements)	The building is a LPS (Large Panel System Construction) with the main structural material as precast concrete. Strutural Typology 6 from the IStructE's guide to assessing Higher Risk Buildings
Design Loading Parameters	We assume that the building was designed to the current British Standard Code of Practice of the time which was: British Standard Code of Practice 3, Chapter V 1952: Imposed Loading Residential Floors = 40lb/ sqft (equivalent to 1.92kN/sqm) Flat Roof with access = 30lb / sqft (equivalent to 1.44kN/sqm) Stairs and Landings = 60lb/sqft (equivalent to 2.88kN/sqm) British Standard Code of Practice 114 1957 The structural use of normal reinforced concrete in buildings
Have there been any refurbishments	Last refurbishment works carried out in 2016 to the external elevation where the facade was upgraded with cladding. A light weight metal frame has also been added to the roof.
Primary Load Bearing System (Beams, columns, wall?)	A large Panel System Building. Precast floor planks on precast walls.
Secondary Load Bearing System (floors)	Suspended in-situ reinforced concrete floor slabs.
Stability System - Horizontal and Vertical	The vertical stabilty is achieved via RC shear walls
Claddings and fixings	Brickwork to external elevations on ground floor level. Upper levels consist of a mixture of render and cladded façade.
Roof Build-Up	Flat roof which consists of an RC concrete slab and insulated PIR/PUR board with felt roof coverings. Light weight metal frame structure with pitched roof constructed above original flat roof.
Description of any basements	N/A
Description of any cantilevered elements - Type(Juliet, cantilever, hung, stacked)	Cantilevered balconies at each floor level.
Fixing method for cantilevered element	Fixed to floor slab.
Cantilevered element structural and decorative materials	Handrails fixed to balcony floor with cladded panels.
Dimension Cantilevered element	1m
Current Structural Condition (Poor, fair, good, excellent)	Fair
Signs of structure not performing correctly (movement or failure)	None observed
Finishes in flat	No access to apartments
Structural Cracks Visible?	None observed
Floors and Wall Levels	Fairly level
Condition of Elevations	Fair

Table 4: Summary of Key Structural Elements

The loads distribute from the RC floor plates to the supporting load bearing RC walls. The walls are supported off the foundations which disperse the loads into the ground. The current foundation system is unknown but is presumed to be a reinforced concrete piled system. Figure 9 describes a schematic diagram of the various joints between flank wall and floor panels in a typical LPS building.

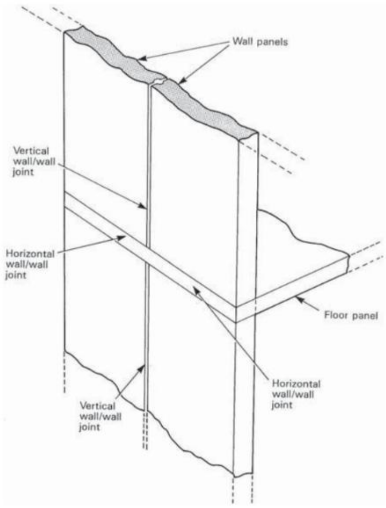

Figure 10 describes the vertical load path for the buildings

4.4 Robustness and Consequence Class

Avoiding disproportionate collapse is a critical concept for the design of new buildings and checking existing ones. It has to be accepted that explosions, usually gas, do occasionally occur and can cause severe damage to residential properties. Avoiding disproportionate collapse ensures that the structure of a building is designed or strengthened to ensure that damage is relatively limited.

The requirement was a consequence of the Ronan Point Large Panel System (LPS) building disaster of 1968. The Ronan Point enquiry produced recommendations (MHLG Circular 62/68) requiring all LPS blocks to be tied together. The initial recommendation referred to buildings with piped gas and required the structure to withstand a force of 34 kN/m^2. Another MKHG circular followed a few months later (71/68) which required a similar approach to buildings where gas was not supplied or had been removed but the pressure which the structure had to withstand was 17 kN/m^2.

The current building regulations (Part A – Structure) are regularly revised and include requirements for robustness to be included, specifically the horizontal tying requirement, to apply to all residential blocks, regardless of height. This emphasises the need when assessing existing buildings with regards to the building regulations, for judgement, research and experience to be applied. Regulation 8 of the regulations places a limit on all requirements such that they "shall not require anything to be done except for the purposes of securing a reasonable standard of health and safety for persons in and about the building".

The requirements for buildings depend on it's "consequence class" as described in Table 11 of Part A of the building regulations 2010.

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Class	Building type and occupancy	Summary requirements
1	• House not exceeding 4 storeys. • Agricultural buildings. • Buildings into which people rarely go.	• No additional measures are likely to be necessary.
2A	• 5 storey single-occupancy houses. • Hotels, apartments and other residential buildings not exceeding 4 storeys. • Offices not exceeding 4 storeys. • Industrial buildings not exceeding 3 storeys. • Retailing premises not exceeding 3 storeys of less than 2000 m^2 floor area in each storey. • Single-storey educational buildings. • All buildings not exceeding 2 storeys to which members of the pubic are admitted and which contain floor areas exceeding 2000 m^2 at each storey.	• Horizontal ties, OR • Effective anchorage of floors to walls, as described in the codes of practice.
2B	• Hotels, apartments and other residential buildings exceeding 4 storeys, but not exceeding 15 storeys. • Educational buildings greater than 1 storey, but not exceeding 15 storeys. • Retail premises greater than 3 storeys but not exceeding 15 storeys. • Hospitals not exceeding 3 storeys. • Offices greater than 4 storeys but not exceeding 15 storeys. • All buildings to which members of the pubic are admitted and which contain floor areas exceeding 2000 m^2 but less than 5000 m^2 at each storey. • Car parking not exceeding 6 storeys.	• Horizontal ties and vertical ties as described in the codes of practice, OR • Show that the removal of a wall or column will cause only limited damage, OR • Design as 'key elements'.
3	• All buildings defined above as Class 2A and 2B that exceed the limits on area and/or number of storeys. • All buildings, containing hazardous substances and/or processes. • Grandstands accommodating more than 5000 spectators.	• Systematic risk assessment.
Note Basement storeys may be excluded provided they meet Class 2B criteria		

Table 5: Building Type and Consequence Class

Kynaston House is classified as a Class 3 building due the fact it's an LPS construction. There are effectively three approaches to determining compliance.

1. Compliance with tying rules
2. That the removal of a wall or column will cause only limited damage
3. Showing that "key elements" and "non-removable" – i.e. can withstand the accidental loading requirements discussed above.

In this case, we have been provided with a BRE report dated 13th March 2009 titles "Assessment of Structural Adequacy and Durability, Russell House" which is a comprehensive study into the resistance to disproportionate collapse and is attached here for reference. Within this report there is reference to a similar study completed by BRE at Kynaston.

Whilst this report does not extend to calculations it is their opinion that based on the findings of their study, and considering ***the building does not have a gas supply***, then the panels can, once remediated to gable ends, withstand 17 kN/m2 in all locations, satisfying point 3.

The findings of the study and investigations by the BRE state that the joints were in accordance with the design requirements and the embedded remedial steel tie provides lateral tying in plane of the floor slab and gable walls.

We do not have evidence of the remedial ties. Based on the BRE study and archive communication listed within section 3 we are of the opinion that the building has a moderate risk of not satisfying the robustness requirements. However, as a mitigation to the risk we suggest spot checking the remedial ties have been installed, therefore strengthening confidence in the buildings ability to resist disproportionate collapse. We also recommend a cover meter survey to assess the fire resistance of the elements as a means of assessing one of the primary hazards that could cause a loss of structural strength in an element.

In addition a HAZID risk assessment was compiled to determine whether there are any hazard scenarios that have an unacceptable level of risk and if so to identify steps to mitigate those risks.

5 Site Inspection

The inspection took place on 12/06/25 and the findings have been recorded in this report. The scope of the inspection included limited, visual observations of the interior and exterior of the structure and was viewed from ground level. The report contains a limited amount of photographs taken during the inspection.

5.1 External Walls Elevations

The exterior elevations were generally in ok condition and no significant structural defects were noted to the cladding panels. There were no significant cracks or tell-tale signs of any significant structural movement noted at the time of our inspection.

The external cladding system is an overclad system which is formed of wraparound brickwork at ground floor, rendered panels to the flank walls and High-Pressure laminate panels to the front and rear.

The exterior walls were noted to be plumb with no signs of significant bowing, bulging or leaning noted to the cladding indicating failed fixings. Archive photographs show the installed vertical railing system.

There are cantilevered stacked balconies at every level. These have steel railings with glass infill panels. No signs of significant movement noted.

Figure 11 and 12: Building Elevations and photographs of Re-clad in progress

Unit 3,
Endeavour House,
Longbridge Road,
PL6 8LR

5.2 Roof

The flat roof structure is a flat slab with an over roof installed built from a steel frame supporting a lightweight pitched steel sheeting. The sheeting provides a waterproofing cover which appeared in sound condition, with no known leaks and water tightness issues.

There were no tell-tale signs of any significant structural movement noted at the time of our inspection. The drainage system should be checked and cleared as part of the normal regimen of maintenance.


Figure 13 and 14: Roof Level

5.3 Typical Floor Plan

The stair core and landing areas of every level were inspected with more detailed assessments made at third points over the height of the building. Refer to Appendix A for site records.

The floors in the communal landings were noted to be solid concrete floors. The floors were checked with a spirit level and were considered level and within tolerance. The doors and windows to the communal landings from the stair core all appeared to be level and opened and closed without difficulty. There were no signs of structural movement detected.

There were vertical concrete walls supporting the floor plates at every level.

The interior of the stair core walls were checked and were plumb and in line with observations made to the exterior – no significant signs of leaning or movement noted. The concrete appeared in fair condition with few BRE category 1 cracks but no structurally significant defects were noted.

The wall panels are measured at 130-150mm thick. A cover test is required to check the resistance of the concrete section in the event of a fire.

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Figure 15 and 16: RC Structure

Figure 17 and 18: RC Elements visible and communal areas

5.4 Summary Key Elements

To assist in compiling the Building Safety Case Report a summary table of the key structural elements which may be included as suggested in the Building Safety Act 2022 has been compiled with highlighted cells indicating areas of missing information which require further investigations.

ELEMENT	FINDING
PRIMARY MATERIAL OF CONSTRUCTION	• Reinforced Concrete
TYPE OF BUILDING	• Large Panels System
PRIMARY LOAD BEARING SYSTEM	• Concrete Walls
STABILITY SYSTEM	• Shear Walls
CLADDING SUPPORT SYSTEM AND FIXINGS	• Precast panels supported off precast RC walls
ROOFING MATERIALS	• NA
FOUNDATIONS	• Presumed Piled System but unknown
BALCONIES	• Stacked cantilevers

OTHER OVERHANG STRUCTURES	• NA
BUILDING CONSEQUENCE CLASS	• Class 3 as per Part A of Building Regulations

Table 6: Key Structural Element Summary

Refer to Appendix A for detailed photographic record of the salient structural elements recorded.

6 Structural Risk Assessment

In line with government guidance one of the objectives of the Higher-Risk Building ("HRB") safety regime is to identify the building's safety risks and describe how they being managed.

The risks included are for structural failure. For the purpose of this report the HAZID technique has been adopted. This is an analysis of the hazards that may impact structural safety of the property, and is included in the Appendix A.

The HAZIDs considered are:

- Non-piped gas explosion
- Fire within a building
- Minor material deterioration
- Sever Material Deterioration
- Water ingress from roof or damaged pipes
- Poor construction/Design Inadequcies
- Overloading
- Underground leakage / flooding causing foundation settlement
- Vehicle impact
- Unauthorised Structural Alterations
- Accidental Aircraft Impact

Each risk associated to a hazard is the combination of:

• the probability of an incident (how likely is it to happen?)

• the severity of an incident (what the consequences could be?)

Severity may be categorised as: low, Localised damage (1) medium, localised failure leading to risk of life/serious injury (2) high, Significant collapse or potential progressive collapse (3)

Probability may be categorised as unlikely (1) probable (2) or certain (3).

The risk is multiplied with the consequence and use a 3x3 matrix to categorise the risks, refer to table under. Recommended mitigation measures are included to either maintain existing control measures or, if possible, adding to them. Suggested timeframes for carrying out the recommendations are added to each item which should form part of the building Action Plan.

		Consequence		
		Localised Damage	Isolated local failure - Life threatening or serious injury possible	Significant failure leading to major catastrophic event. Life threatening or serious injury possible to multiple people
Likelihood	High	3	6	9
	Medium	2	4	6
	Low	1	2	3

Table 4 - Categorisation of Risk using 3x3 Matrix

Green = Tolerable risk

Yellow = Medium risk to be managed and if reasonable mitigated

Red = Intolerable Risk to be mitigated ASAP

7 Conclusions and Recommendations

The findings of this report revealed that the building is built using the Large Panel System (LPS) form of construction, a method which involved assembling precast concrete panels on site to form the structure of the building. Due to LPS buildings susceptibility to disproportionate collapse, as discussed in detail in section 4.4, it is essential to perform intrusive investigations into the remedial ties installed between the wall and floor panels, as there is a risk the building does not satisfy the requirements and could therefore pose a risk to the residents and users of the block. We have been provided with archive correspondence which states that the tying requirements are met and that the remedial ties have been installed however no evidence is supplied to support their presence. Therefore we suggest completing spot checks to the ties in at least 25% flats of the property.

These should include opening up works to assess the form and presence of the remedial ties. Once their presence and form is confirmed it is possible to use a GPR to check the remaining joints. We are expecting ties similar to the ones installed to Russell House, shown below.

Unit 3,
Endeavour House,
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It is not possible to confirm the fire resistance of the structure without performing a cover test to the panels. We therefore recommend a cover meter survey to assess the depth of reinforcement within the wall, and hence the fire resistance of the elements, as a means of assessing one of the primary hazards that could cause a loss of structural strength in an element.

Due to the age and construction type of the building it is also essential to perform durability checks to the concrete elements. This should include tests for carbonation, chlorides and HAC. It would appear that these tests have never been performed to this building and therefore we recommend engaging a specialist contractor as soon as possible who will be able to perform the above tests and provide an interpretive report.

We have not been provided with floorplans or any information, construction details, product specifications and warranties on the building or re-cladding. We suggest these are sourced from the original contractors and designers and kept on file by the Duty Holder as part of the Golden Thread for future reference.

To effectively manage the risks we suggest completing the actions from the risk assessment included in Appendix A.

As part of the structural safety risk management for the building an inspection regime which includes a combination of superficial , regular and benchmark investigations should be implemented. These are described as follows:

Superficial Inspection: Personnel visiting the structure and noting visible indications of potential durability issues. The building manager to have an adequate system for keeping up to date with structural changes to the building and an engagement strategy in place for residents to raise concerns.

Regular inspections: The building is to be surveyed at approximately 5 year intervals by a competent engineer to inspect for signs of movement, structural decay and/or defects.

Benchmark Inspections which will include durability intrusive investigations: Every 15-20 years, when there is a functional change, or when a regular inspection notes a particular issue, whichever comes fists. Note these have just been completed.

Unit 3,
Endeavour House,
Longbridge Road,
PL6 8LR

Appendix A – HAZID Risk Assessment

Item	Scenario	Hazard	Consequence	Existing Controls	Original Risk			Mitigation Measures	Administrative Mitigation Measures	Residual Risk			Recommended Action	Timeframe
					Probability	Severity	Rating			Probability	Severity	Rating		
1	Gas Explosion	Structural failure to walls	Partial or total collapse if the impacted element is a key element	The block does not have a piped gas supply. The block underwent a study by the BRE and was found to be (according to archive information) adequate at resisting non-piped gas loads (17kPa) to the internal panels but requiring stiches between the external and internal floor slab. The presence of these ties is unconfirmed and therefore requires further investigation work to evidence.	2	3	6	Perform checks to strucutre to demonstrate tying between panels	Ensure storage constraints are applied, such as banning gas cannisters. Install signage in prominent places warning residents of dangers of storing gas canisters. Brief all residents to explain the risks of storing and using bottled gas within a large panel system. Where residents are required to have oxygen cylinders in the home for medical purposes, ensure they are aware of where the oxygen cylinders are located and incorporate this information in their emergency plans so that local FRS are aware.	1	2	2	PAP has confirmed that gas storage constraint is within tenancy agreement and that suggested signs are within lifts. Demonstate ties existing between gable panels and inner floors	Design measures within 12 months. Admin measures are in place and ongoing
2	Fire within a building	Significant damage to a structural element due to loss of load bearing capacity	Single member experiences plastic deformation causing structural collapse. As the building is LPS a fire could lead to disproportionate collapse	Concrete elements have an inherent fire resistance. The prescirbed fire resistance at the time of construction (CP114) would have required 90 minutes however this relies on the cover to the reinforcement which is currently unknown.	2	3	6	Cover checks in representative locations of the strcuture with section analysis to ensure prescribed fire resistance is met. A collaborative risk assessment with the fire engineer for the building	Regular FRA's undertaken. Management of storage units to the ground floor	2	1	2	Cover meter tests to determine structural fire resistance of frame with result shared and assessed collaboratively with fire engineer	12 months or sooner
3	Minor Material Deterioration	Spalling elements cause cover to burst.	Damage to areas/people below spalled element.	Concrete elements have been designed for a prescribed design life and exposure class with sufficient cover to reinforcement steel.	3	1	3	Perform visual structural surveys on a regular basis to inspect critical signs of deterioation.	Monitoring of signs of deterioration within floor plank soffits to be included in superficial inspections	1	1	1	An inspection regime which includes a combination of superficial , regular and benchmark investigations should be implemented. As per BRE Digest 444.	Ongoing
4	Severe Material Deterioration	Damage to a structural element causes a loss of load bearing capacity	Single member experiences plastic deformation causing structural collapse	Concrete elements have been designed for a prescribed design life and exposure class with sufficient cover to reinforcement steel.	2	3	6	Concrete durability material tests for strength, carbonation, presence of chlorides and HAC.	Monitoring of signs of deterioration within floor plank soffits to be included in superficial inspections	1	3	3	Undertake the concrete durability material tests for strength, carbonation, presence of chlorides and HAC.	24 months
5	Water ingress from roof or damaged pipes	Damage to structural elements from exposure to water causes RC elements to spall	Damage to areas/people below spalled element	Concrete elements have some inherent protection from localised short term water ingress provided by the concrete cover	3	1	3	None	Perform visual structural surveys on a regular basis to inspect for signs of water ingress. Ensure there is an adequate resident engagement strategy to report leakages.	2	1	2	An inspection regime which includes regular visual imspections by a competent person. Implement an adequate resident engagement strategy for reporting leaks / dampness	6 months
6	Poor construction / design inadequacies	Significant damage to a structural element due to insufficient load bearing capacity	Single member experiences plastic deformation causing structural collapse	LPS are known to have poor quality workmanship issues. Lateral stability from Wind was not always considered adequately at time of construciton	2	3	6	Critical construction details should be checked. Concrete cores tested. Consider an overall stability check is required if not included in the BRE check previously commissioned.		1	2	2	Spotcover checks to critical details and concrete strength tests to RC frame. Consider further analysis into the global lateral stability	18-24 months

7	Underground leakage / Flooding	Corrosion to soils under foundations causing subsidence. Altered exposure class to concrete elements expediates deteriation (refer to item 6)	Building suffers differential settlement	Building may subside due to differential settlements across the footprint of the building causing damage to finishes and services	2	2	4	None	Underground drainage survey and regulat 5 year visual surveys to inspect for movement	1	2	2	An inspection regime which includes regular visual impsections by a competent person. Complete an undergruond drainage survey for drains adjacent to properties	12 months
8	Vehicle Impact (external)	Structural failure to columns/walls which are subject to impact	Partial or total collapse if the impacted element is a key element	Builiding is primarily set back and protected from traffic and estate car parking area and therefore risk of impact is low. However as there are known robustness issues the impact of a vehicle to a load bearing panel could precipitate disproportionate collapse	1	3	3	Perform checks to strucutre to demonstrate tying between panels	None	1	2	2	Perform checks to strucutre to demonstrate tying between panels	12 months
9	Unauthorised Structural Alterations	The load path within the structure changes	Single member is overloaded and experiences plastic deformation causing structural collapse	Building manager permission is required when renovating units within the building	3	2	6	Work applications are to be reviewed by a competent structural engineer	Ensure that the residents are aware of the conseuquences of unauthorised alterations.	3	1	3	PAP to put up notices within the building lobby notifying residents of dangers of unauthorised alterations.	6 months
10	Overloading of structure	Significant damage to a structural element due to insufficient load bearing capacity. Overstressing of the elements that contribute to lateral stablity	Single member experiences plastic deformation causing structural collapse. Excessive lateral deflection of the building leading to cracking of structural elements	We presume that the building was designed to the loadings specified in the British Standard Code of Practice of the time which was: British Standard Code of Practice 3, Chapter V 1952: Imposed Loading Residential Floors = 40lb/ sqft (equivalent to 1.92kN/sqm)	2	3	6	None	Ensure storage and usage constraints are applied within blocks to avoid overloading. Corridor and staircase are unlikely to be subject to unusual loadings as they are monitored by PAPs personnel	1	3	3	PAP to confirm storage and usage constraints are applied, Monitoring of building cracks due to horizontal loadings to be included within the regular inspections	Ongoing
11	Accidental Aircraft Impact	Structural failure to columns/walls which are subject to impact	Partial or total collapse if the impacted element is a key element	Robustness relies on the tying between floors and wall panels. However, no evidence is supplied to support the form or presence of the ties within the block.	1	3	3	None	Building is <150m limit for aircraft warning lights. The risk of aircraft impact is improbable and therefore overall risk is considered as low as reasonably possible wihtout further mitigation measures required.	1	3	3	Perform checks to strucutre to demonstrate tying between panels	12 months

Unit 3,
Endeavour House,
Longbridge Road,
PL6 8LR

Appendix B – Survey Photographs

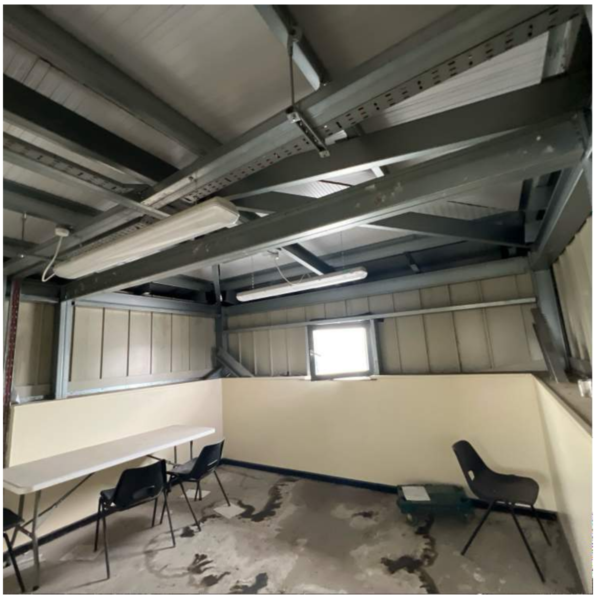
Kynaston House

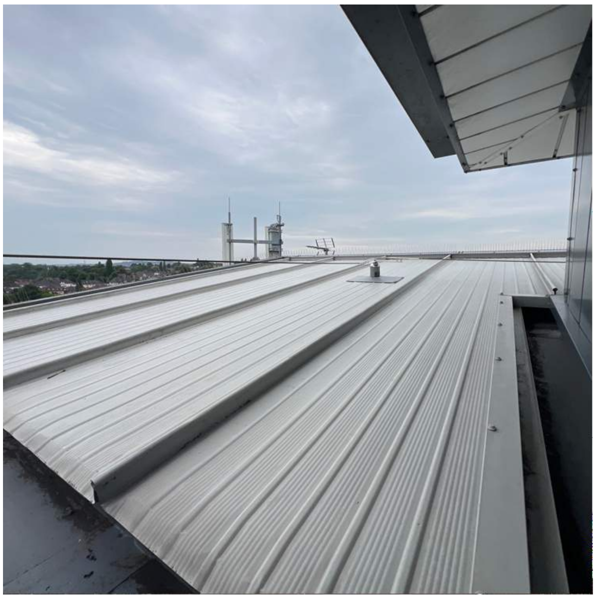
Firntec
12 Jun 2025

Project Details

Name: Kynaston House

Main Roof

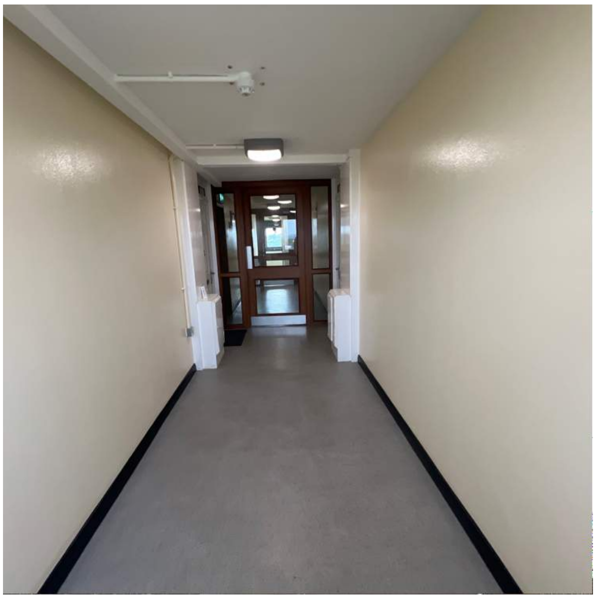


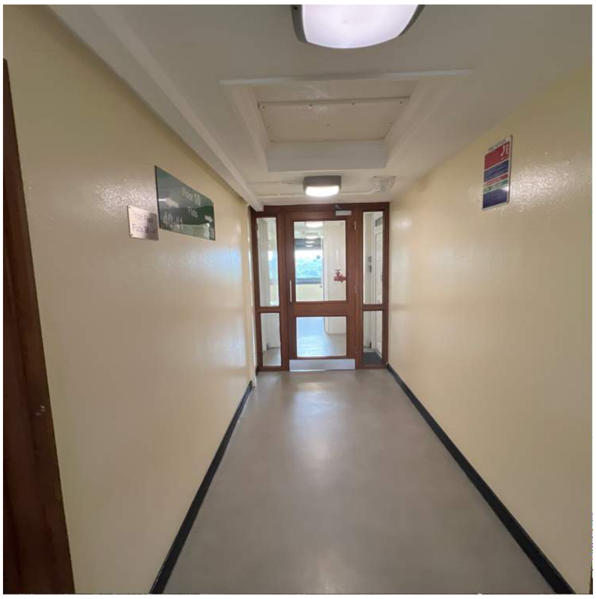


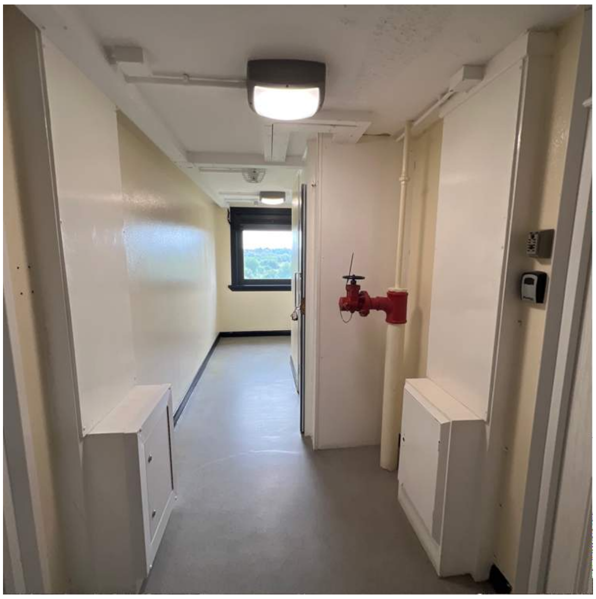


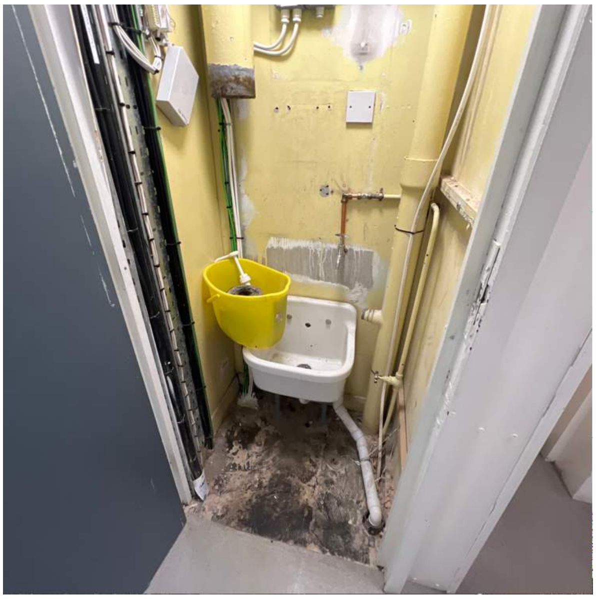

Comments: A lightweight pitched roof constructed above original flat roof structure utilising beams and purlins with aluminium standing / mineral wool core profiled panels.
Lift concrete shear core structure noted up to roof level.

10th Floor Hallway





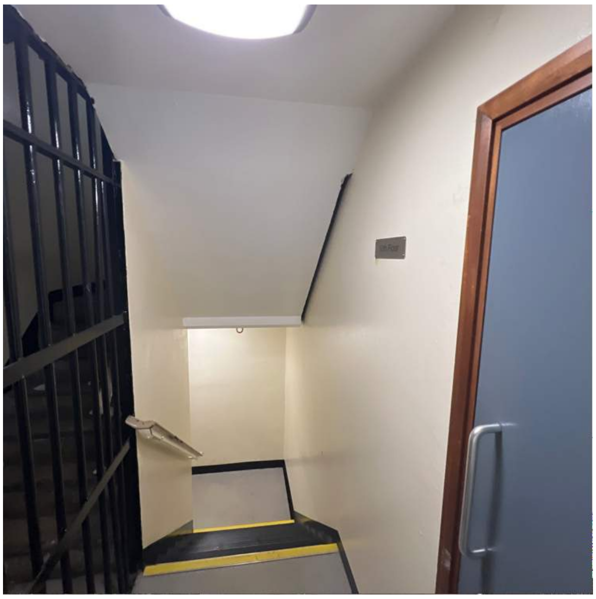
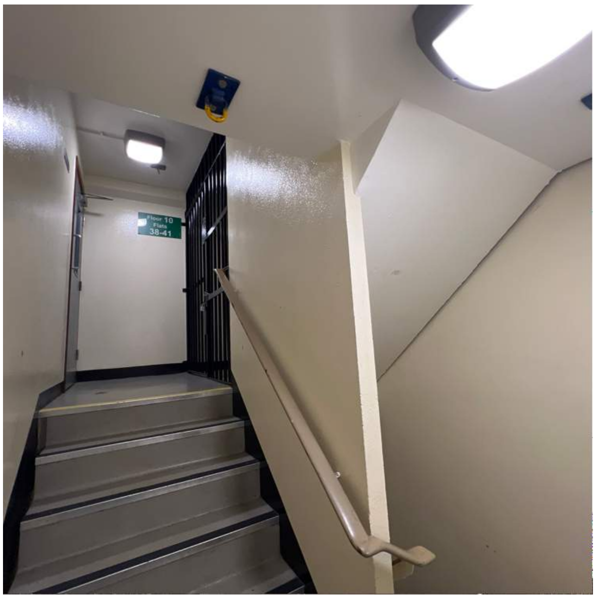
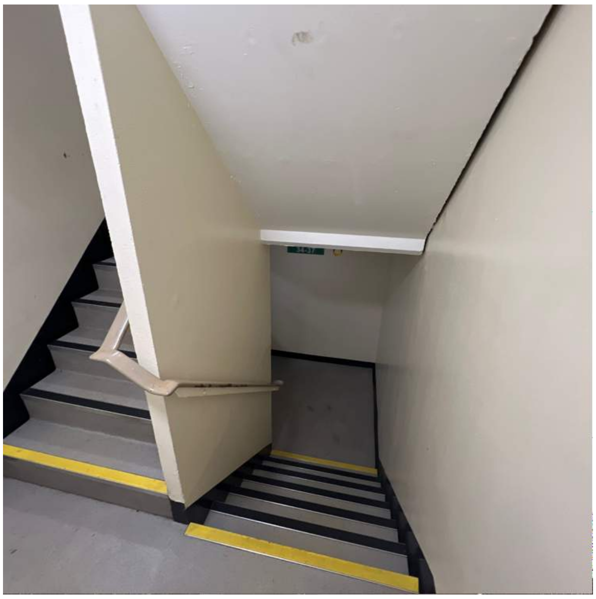
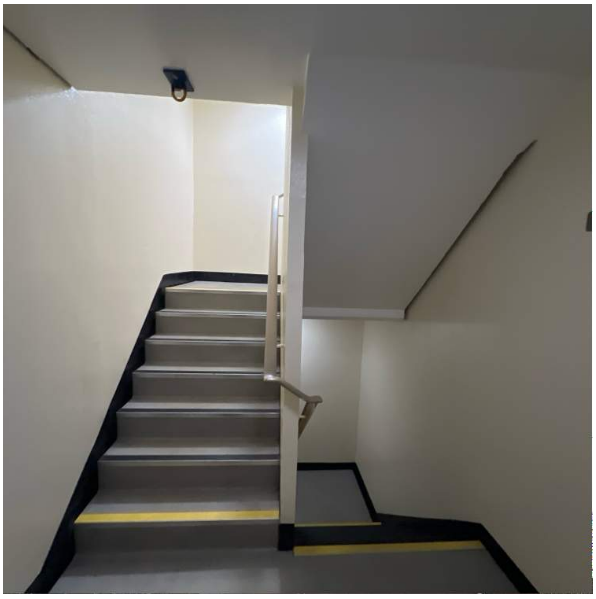




Comments: No jams or distortions noted to hallway doors to indicate any serviceability issues. Generally walls covered with finishes in hallway, likely to consist a mixture of concrete shear walls, embedded columns and masonry infill walls. Limited structure exposed. Concrete walls and infill masonry (non-load bearing) walls noted in riser cupboard.

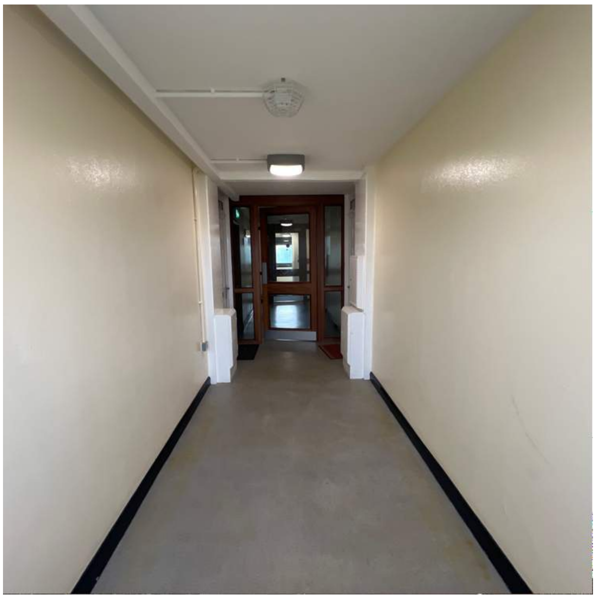
01752 977147 info@firntec.com www.firntec.com

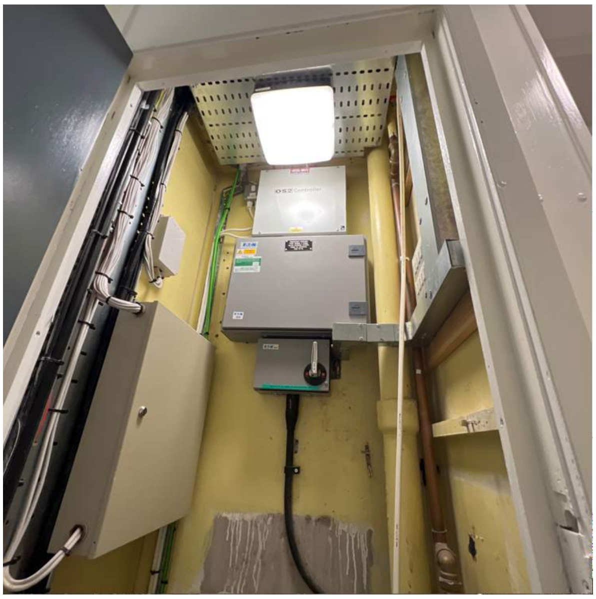
10th Floor Staircase

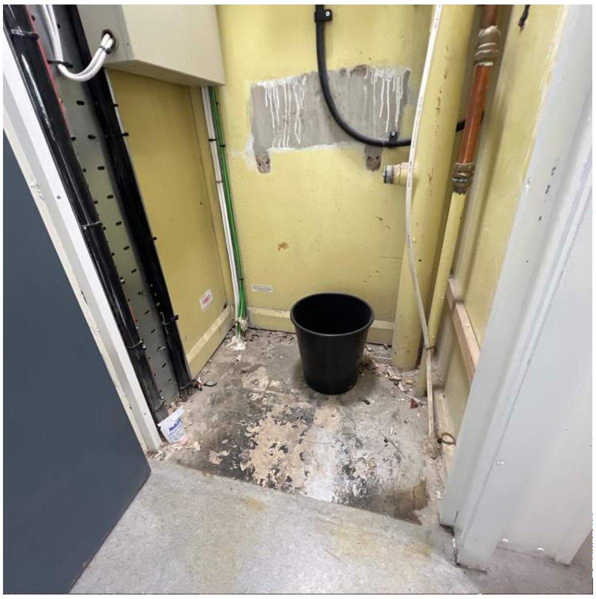





Comments: Insitu concrete stairs.
Walls and floors appear to be faily level.
Stair walls covered with finishes however these would likely form the main concrete shear core walls.

5th Floor Hallway

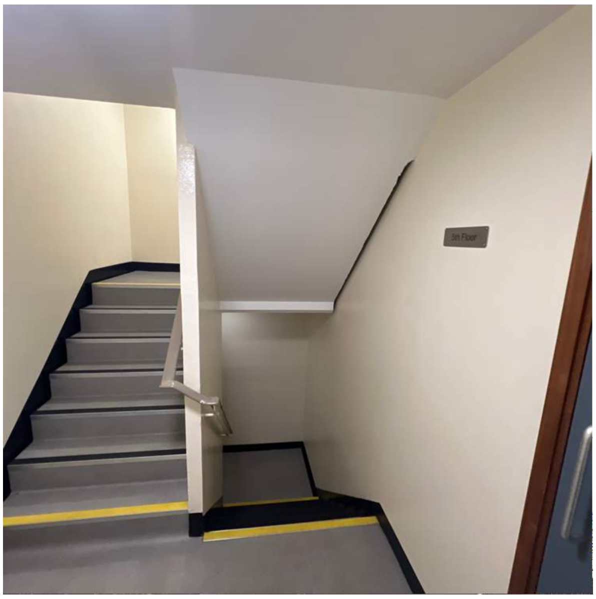
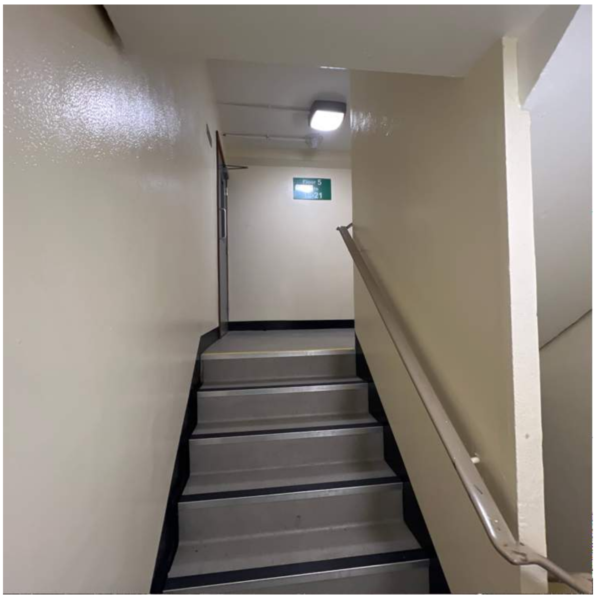
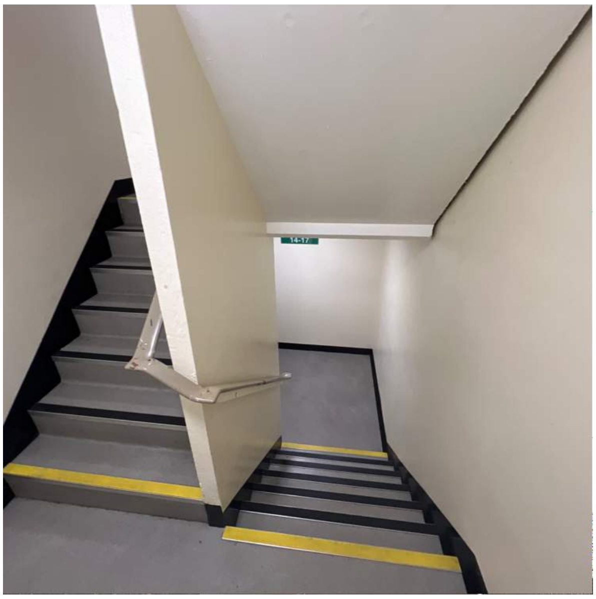
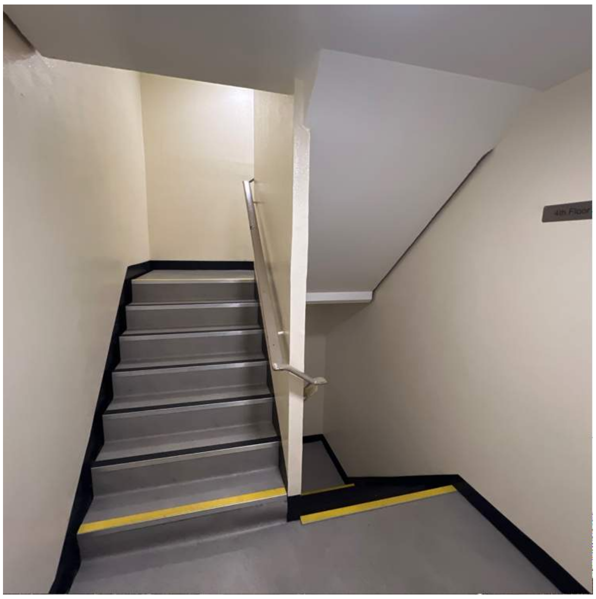




Comments: No jams or distortions noted to hallway doors to indicate any serviceability issues. Generally walls covered with finishes in hallway, likely to consist a mixture of concrete shear walls, embedded columns and masonry infill walls. Limited structure exposed. Concrete walls and infill (non-load bearing) walls noted in riser cupboard.

5th Floor Staircase

Comments: Insitu concrete stairs.
Walls and floors appear to be faily level.
Stair walls covered with finishes however these would likely form the main concrete shear core walls.

Ground Floor Hallway

Comments: No jams or distortions noted to hallway doors to indicate any serviceability issues. Generally walls covered with finishes in hallway, likely to consist a mixture of concrete shear walls, embedded columns and masonry infill walls.
RC Columns/walls noted in lobby area and in electrical room.

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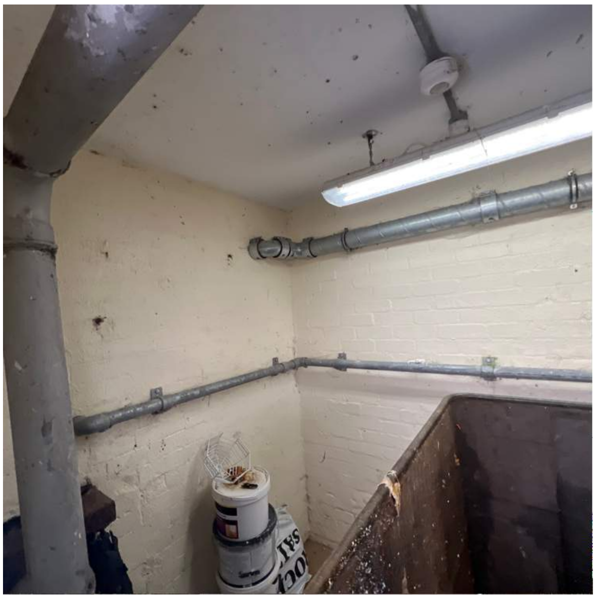
Bin Store




Comments: RC concrete walls noted in bin store.

Elevations




Comments: Brick work at ground floor level all sides. Cladded panels to upper floors, installed as part of last refurbishment works.
Cantilevered balconies noted on each level.

Unit 3,
Endeavour House,
Longbridge Road,
PL6 8LR

Appendix C – BRE Report on "Assessment of Structural Adequacy and Durability – Russell House, Wednesbury, Sandwell"

Assessment of Structural Adequacy and Durability - Russell House, Wednesbury, Sandwell

FINAL

Prepared for:
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CV2337

13th March 2009

Client report number 246-866

Prepared by

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Executive Summary

ES1 Background

Sandwell Metropolitan Borough Council (MBC) is responsible for a number of precast concrete high-rise large panel system (LPS) built residential blocks. Amongst these is Russell House, a 11-storey block constructed using the Bison Wallframe large panel system (LPS) of construction.

BRE have been invited to undertake a feasibility study into the viability of overcladding the external envelope to Russell House. The study is being undertaken in two phases. Phase I of the study consisted of a review of existing documentation, which included a limited number of construction drawings and two technical documents, together with an assessment of the ability of the block to accommodate the forces associated with a severe non-piped gas explosion. The structural assessment included a limited invasive investigation of selected areas of the structure and a programme of materials testing of selected reinforced concrete components exposed to internal environments. BRE's assessment also included a review of existing materials test data for the external reinforced concrete balconies.

It is intended that Phase II (which is currently in abatement) will comprise a critical review by BRE of an overcladding scheme submitted by Sandwell MBC.

This report presents the results of the Phase I study.

A sister building, Kynaston House, will be appraised by BRE as part of a separate, but related, commission.

ES2 Structural Appraisal Against Accidental Loading

BRE has previously undertaken full-scale static load testing of two type types of LPS blocks to establish their structural performance. The LPS types tested were Reema Conclad and Bison Wallframe. The three blocks tested by BRE were built in the mid to late 1960's (i.e. pre-Ronan Point designs). The 10-storey Reema Conclad block was located in Leeds, whilst the two high-rise Bison Wallframe blocks were situated in Sandwell and Liverpool.

The experimental work undertaken by BRE demonstrated that the buildings tested would have been able to withstand an accidental load over-pressure of $17kN/m^2$ without gross deformation or failure of the structure. The load test programme undertaken is considered to have been very successful, providing a basis on which to evaluate the prospective performance of other LPS blocks of similar (or better) design and standard of construction.

Since it has been reported to BRE that **<u>no gas supply has been provided to Russell House</u>**, then it was considered appropriate to evaluate the robustness of the block should a severe non-piped gas explosion occur within the building. That is, there was a requirement to consider the likely behaviour of the structure should the wall and floor panels bounding the site of a prospective explosion be subjected to a $17kN/m^2$ over-pressure (i.e. assessment criterion for a non-piped gas explosion).

The *'deterministic'* evaluation using 'simplified' calculations undertaken by BRE has indicated that the majority of the wall and floor panels (and associated joints) within Russell House should be sufficiently strong to accommodate the forces arising from a severe non-piped gas explosion generating an overpressure of $17kN/m^2$.

This opinion is supported by the fact that the long span floor slabs in both the Bison Wallframe and Reema blocks previously load tested by BRE were able to accommodate an overpressure loading of this magnitude (and more) albeit being damaged at the end of the load test.

However, it has been calculated that the two lounge floor slabs (Floor 1-1) in Russell House have an overload factor of 2.3 under upward accidental loading (these floor slabs are expected to be able to accommodate the forces under downward loading associated with an overpressure of $17kN/m^2$ even though the overload factor was calculated to be approximately 1.3). In considering the high value of the overload factor it was therefore difficult (on the basis of the simplified calculations) to justify that these floor slabs could survive an overpressure loading of $17kN/m^2$ under upward flexure.

In an attempt to resolve this impasse Sandwell MBC subsequently commissioned BRE to carry out a non-linear finite element analysis[Σ] of a representative long span floor slab under upward loading. A very limited parametric study was carried out to assess the influence of concrete tensile strength on the load/deflection response of the floor slab being modelled. Non-linear finite element analyses (NLFEA) is able to examine the influence of additional and more complex forms of structural and material behaviour than can be considered in a 'simplified' analysis. The NLFEA was expected to provide an insight into the ability of the long span floor slabs to survive the applied uniformly distributed upward loading.

The NLFEA undertaken by BRE has indicated that the long span reinforced concrete floor slabs fabricated from concrete with a tensile strength in excess of $2N/mm^2$ should be able to accommodate an upward overpressure of $17kN/ m^2$, albeit suffering from significant damage in the form of gross cracking/final set.

All the floor slabs assessed by BRE are expected to perform adequately in shear.

However, the evaluation based upon 'simplified' calculations did indicate that the spine wall panels (Wall Type 1-4) between the kitchen and lounge in the flank wall flats at most storey levels are expected to fail in flexure and/or sliding under accidental loading. This situation is further compounded by the identification of shortcomings in at least some of the welded tie connections that should exist between the end of the kitchen floor slab and side of the adjacent lounge floor slab within the flank wall flats. Defects in this type of connection could potentially lead to disproportionate damage of the building in the event of the loss of a kitchen/lounge spine wall panel.

Accordingly, it is strongly recommended that invasive investigations should be undertaken within selected flank wall flats to establish whether this tying defect is likely to be common within the building. If this tying defect is found to be commonplace then each missing/defective tie should be reinstated/correctly formed.

[Σ] In previous structural assessments of LPS blocks undertaken by BRE we have been able to show using non-linear finite element modelling, that the floor slabs that fail the simplified deterministic evaluation, should be able to survive a severe non-piped gas explosion, albeit in a damaged condition.

A proposal for two alternative evaluation methodologies (i.e. invasive and non-invasive) aimed at defining the extent of this defect is included in this report.

ES3 Assessment of Future Concrete Durability

Internal Floor Slabs

The chloride ion content of the floor slabs tested was low in each instance. Accordingly, chlorides are not expected to influence the long term durability of the concrete floor slabs tested.

It is calculated that in the areas tested it will take approximately 40 years from now for the carbonation front to reach approximately 10% of the reinforcement embedded adjacent the soffit to the floor slabs. It is also predicted that it will probably take a further 40 years (i.e. approx. 80 years from now) for the carbonation front to reach about 40% of the reinforcement located adjacent the soffits to these components.

Accordingly, the risk of corrosion of the reinforcement located adjacent the slab soffits is expected to be ‘negligible’ for the foreseeable future.

Nevertheless, it should be noted that water leaks from defective pipework or seals around baths and/or showers will increase the risk of corrosion locally once the carbonation front reaches the embedded reinforcement.

Internal Wall Panels

Chloride ion contents were generally low apart from two out of the 14no. dust samples which had a chloride ion content of 0.54%Cl^- and 0.68%Cl^- (by weight of cement), respectively. Accordingly, chlorides are not expected to influence the long term durability of the majority of the internal concrete wall panels tested.

In the areas tested we calculate that it will take in excess of 45 years from now for the carbonation front to reach some sections of the embedded reinforcement.

On the basis of a simple statistical analysis of the concrete cover and carbonation data obtained by BRE, it can be shown that it could take in excess of 150 years from now for the carbonation front to reach about 10% of the reinforcement. It will take a further 50 years or so for this percentage to increase to 40%.

Accordingly, the risk of corrosion of the reinforcement in the internal wall panels is expected to vary from ‘negligible’ to ‘moderate’ for the foreseeable future depending on the chloride ion content. This risk could, however, increase to ‘high’ in areas where a wall panel containing a level of chloride in excess of about 0.4%Cl^-, becomes ‘damp’ due to water leaks as a result of defective bath seals or as a result of leaking pipes.

In-situ Concrete

Whilst no concrete dust samples were obtained from the in-situ concrete joints or material tests undertaken to determine carbonation depth and chloride ion content, the reinforcement embedded in the in-situ concrete joints appeared, in the main, to be sound and free from significant levels of corrosion. Accordingly, currently the risk of corrosion of the reinforcement embedded in the structural joints examined by BRE is considered to be low in areas where the concrete remains dry.

The risk of corrosion is, however, expected to increase by one category (refer **Figure 6**) where the concrete becomes ‘damp’ due to water leaks or ingressing water.

Balcony Floor Slabs

Some of the slabs were found to contain significant levels (>0.4%Cl^- by weight of cement) of cast-in chlorides. However, since calculations have indicated that the carbonation front could take well in excess of 100 years or more from now to reach the reinforcement, BRE anticipate that the concrete balcony slabs should generally remain serviceable over this time span (*It should be noted that with the level of chlorides present in the slabs tested, significant corrosion of the embedded reinforcement is only expected to occur once the carbonation front has reached/passed the reinforcement).*

External Cladding Panels

Since Sandwell MBC is planning to overclad the external envelope to the block in the near future BRE considered that there was little merit in undertaking a materials testing programme upon the external cladding panels. On the assumption that the external cladding panels will eventually be surrounded by what is expected to be a ‘dry’ internal environment, the future risk of corrosion of the reinforcement embedded in the external cladding panels is therefore expected to be broadly comparable to that of the internal wall panels.

However, it is recommended that the overcladding system and external cladding panels should be subjected to a detailed hygrothermal thermal analysis/condensation risk analysis to determine that the existing cladding panels will in fact remain dry. This work should be undertaken as part of the feasibility study.

Accordingly, assuming that the outcome of the hygrothermal thermal analysis/condensation risk analysis is favourable then the external cladding panels are expected to remain serviceable for a period in excess of 50 years from now.

Definitions

Cross wall	The main load bearing walls lying perpendicular to the main building axis.
Flank wall	The load bearing end walls lying perpendicular to the main building axis.
Spine wall	The walls bordering the central corridor running parallel to the main building axis.
Elevation wall	The external walls running parallel to the main building axis.
Levelling dowel	A 25mm diameter steel dowel projecting from the head of a wall panel.
Squat dowel	A short small diameter dowel projecting from the head of a wall panel over which a loop projecting from the end of a floor slab should be placed.
Assessment criterion	An overpressure loading applied to all walls and floors bordering the site of an explosion. This is taken to be 17kN/m^2 for a non-piped gas explosion and 34 kN/m^2 for a piped gas explosion.

Contents

1 Introduction

Sandwell MBC contacted BRE in November 2007 to enquire whether we were able to undertake a feasibility study into the viability of overcladding Russell House - a high-rise block built using the Bison Wallframe form of large panel construction. It was intended that the feasibility study would be carried out in two phases. Phase I would involve a review of existing documentation which included a limited number of construction drawings together with consideration of the technical report included as reference [1]. A limited invasive investigation of selected areas of the structure together with a programme of materials testing of the reinforced concrete components exposed to internal environments was also to be undertaken under Phase I. Finally, BRE undertook an assessment of the ability of the block to accommodate the forces associated with a severe non-piped gas explosion.

It was intended that Phase II (which is currently in abatement) will comprise a critical review by BRE of an overcladding scheme submitted by Sandwell MBC.

BRE subsequently received a verbal instruction to precede with Phase I of the study on 25th January 2008. This was supported by a formal written instruction on 4th August 2008.

The site works undertaken in Flats 5[Π] and 50 Russell House and externally at first floor level as part of Phase I commenced on 2nd April 2008 and were completed on 4th April 2008. A second flat (Flat 30) was investigated by BRE on the 2nd and 3rd July 2008.

This report presents the results of a review of a limited number of documents together with the results of a structural and durability assessment of Russell House undertaken by BRE under Phase I of the feasibility study.

[Π] Since Flat 5 was located at 1st floor level the detailing of the connections of the precast wall panels to the in-situ podium was atypical of the flats from the 2nd floor level upwards. Accordingly, only coring of the wall panels was undertaken within this flat.

2 Background

Sandwell Metropolitan Borough Council (MBC) is responsible for a number of precast concrete high-rise large panel system (LPS) built residential blocks. Amongst these is Russell House, a 11-storey block constructed using the Bison Wallframe large panel system (LPS). A sister building, Kynaston House, will be appraised as part of a separate, but related, commission.

Russell House was located on a level site in the Wednesbury area of Birmingham. It is understood that the block was constructed in the mid 1960's prior to the partial collapse of Ronan Point in 1968. The block was rectangular in plan form with narrow re-entrant areas in the West and East facing elevations (**Plates 1** & **2**).

From our inspection of the building it appeared that the ground floor storey was of in-situ concrete cross and spine wall construction with an in-situ reinforced concrete first floor slab.

The North and South facing elevations were clad with plain non-load bearing precast concrete sandwich cladding panels. The precast wall panels forming the West and East facing elevations were load bearing.

Precast concrete floor panels spanned between the intermediate load bearing cross walls and at each end of the building, the respective flank wall panels. Non-load bearing internal walls were of plasterboard faced timber studwork or 62mm thick concrete panels

The flat roof comprised precast concrete floor panels spanning West-East. The roof deck was finished with a silver coloured reflective membrane. Numerous telecommunication aerials and repeaters had been attached to the parapet panels, flat roofed lift motor/plant room and adjacent single storey hut.

Plates 1 to **4** show the appearance of external envelope to Russell House. **Plates 38** to **40** show various views at roof level.

The block had been previously assessed by consulting engineers after the partial collapse of Ronan Point in 1968 and as a result of this study, subsequently strengthened by the provision of steel angles at the base of the flank wall panels at all floor levels. The piped gas supply to the building is understood to have been terminated at that time. It was deemed at that time that the block should perform satisfactorily under normal loads (dead, imposed and wind loads) and would be unlikely to suffer disproportionate collapse under accidental loads specified for a building without a piped gas supply.

BRE understand that an evaluation of the distribution of the cast-in mechanical ties used to secure the outer and inner leafs to the external cladding panel provision (between) was carried out by Birmingham City Laboratories in 1985 [1]. As a result of this study remedial works were commissioned by Sandwell MBC to retro-fit remedial bolts between the inner and outer leafs to rectify perceived shortcomings in the original tying provision. BRE understand that no other structural appraisals have been undertaken of this building since this time.

BRE was invited by Sandwell MBC in early 2008 to undertake an assessment of Russell House in line with a previous assessment carried out by BRE on a sister block, Kynaston House (refer BRE report GI281) in 1997.

The current assessment is intended to form part of the feasibility study into whether the building is still expected to be able to accommodate both normal and accidental loads arising from a non-piped gas explosion, and also whether it is technically feasible to overclad the building.

2.1 Full-Scale Load Testing of LPS Buildings

BRE have documents that relate to a programme of experimental work that was undertaken in the 1960's to evaluate the ability of a pre-Ronan Point Reema block to resists the overpressures associated with a gas explosion. The report upon the experimental studies [2] indicated that the wall/floor joint and associated load bearing wall components would be expected to perform satisfactorily under a lateral loading of 17kN/m^2 at all levels apart from the top storey.

BRE has been involved in the investigation and assessment of many large panel system built blocks and other forms of housing for a number of local authorities and building owners. These studies have generally been concerned with evaluating the potential behaviour of the blocks under either normal or accidental loading or determining durability/maintenance requirements. An aspect of the former work involved the full-scale testing of three LPS blocks, a Reema Conclad and two Bison Wallframe buildings [3,4].

The testing has helped in the assessment of the ability of LPS blocks of similar design to resist an accidental loading of 17kN/m^2 on wall and floor components, and in evaluating how a LPS block might behave when certain[¥] load bearing walls are removed. Such forces/actions might arise as a result of a severe non-piped gas explosion. This work has provided important background data for the current BRE assessment of Russell House.

2.2 Structural Assessment Methodology and Criterion

2.2.1 Accidental Loading : The Risk Environment

Notionally LPS buildings are susceptible to two principle forms of accidental loading, namely :

- Vehicle impacts, and
- explosions.

A review of the occurrence of damage to buildings within the United Kingdom caused by vehicle impact over the periods 1971-1981 and 1985-1995 showed that there were no incidents reported causing structurally significant damage to LPS buildings of 5 or more storeys. Overall vehicle impacts appear to pose an appreciably smaller risk of causing disproportionate collapse of high-rise LPS blocks than gaseous explosions. Consequently we have only considered accidental loading associated with a non-piped gas explosion in our assessment of Russell House.

[¥] Spine wall panel between the kitchen and lounge in a Bison Wallframe block.

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, as well as the type and nature of the building concerned. Research has indicated that the pressures generated during explosions involving cylinder gas (LPG) and aerosols accord fairly well with the 17kN/m^2 overpressure used to appraise buildings without a piped gas supply. Theoretically the pressures generated could be much larger. However, venting via windows and other relatively weak elements of the construction typically reduce the peak pressure generated to less than that the recommended overpressure of 17kN/m^2. Pressures generated during more complex explosions associated with the larger volume of gas from piped gas supplies can be much larger and involve more than one room (i.e. they may be of a cascade type, rather than a single localised incident).

Although the total number of reported incidents within the United Kingdom has shown a considerable decline over the past two decades or so, there has been a noticeable increase in the number of explosions causing significant structural damage which are attributed to cylinder gas and aerosols. However, the number of structurally significant explosions in all types of dwelling buildings of 5 storeys and over remains very low, generally being less than 3no. incidents per year. This includes buildings with and without a piped gas supply. Most fatalities associated with explosions in buildings (all types) occur in circumstances where structural damage was minor. Thus the risk of any 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. The behaviour of high-rise LPS blocks over the last 30 years or so suggests that the risk of injury or death to their residents arising from structural failures or damage due to accidental loading is not appreciably different to that of the general population of UK buildings.

2.2.2 Russell House : Our Approach

The process of assessment adopted by BRE has involved :

1) A review of technical information available for Russell House.
2) Comparison of this information with that available for the Reema and Bison LPS blocks subjected to load testing by the BRE
3) Consideration of the structural evaluation calculations undertaken by BRE for the blocks which were load tested and also for other LPS blocks more generally.
4) Consideration of the tying provision [5] in the structural joints investigated in Russell House by BRE and whether the tying was broadly in accordance with the limited information given in the construction drawings for pre-Ronan Point Bison Wallframe blocks. The reader should refer to section 4.2 and Appendix A for further comments on this aspect.

When undertaking an assessment of pre-Ronan Point LPS blocks it has been normal practice for BRE to undertake calculations which seek to explore the way in which selected load-bearing elements of the structure might behave under an accidental loading of 17kN/m^2 (criterion for a non-piped gas explosion). The calculations do not follow closely the specific requirements of a Code of Practice, but are intended to improve insight into potential behaviour. As such the calculations are not considered to be suitable as an analytical justification of behaviour in terms of such code / guidance requirements. An example of a contemporary guidance document is given as reference [6]. The intention is that calculations of this type might provide the basis for judging whether it is probable that the structures would survive an accidental loading of the specified nature, and in the case of buildings to be subjected to load testing, whether it might be possible to demonstrate this by experiment or proof testing procedures.

The 'realistic' calculations normally undertaken by BRE for pre-Ronan Point LPS buildings explores the following behaviours:

- Floor slabs - upward loading
- Floor slabs - downward loading
- Floor slabs - shear
- Cross wall panels - shear and flexure
- Spine wall panels - shear and flexure
- Flank wall panels - shear and flexure (assuming no strengthening steelwork was present).

In addition, it is also necessary to make an evaluation of the 'general condition / form' of a number of the connections (joints) between the above components.

Due to the general lack of comprehensive tying within LPS blocks designed prior to 1968, there is a requirement for the walls and floors (and associated joints) within LPS buildings of this age to be able to accommodate the forces associated with a severe non-piped gas explosion without the loss of a key load bearing component.

Accordingly, due to the construction of Russell House (i.e. pre-Ronan Point Bison Wallframe) BRE considered that it is appropriate to carry out simplified calculations to evaluate the potential behaviour of the building under accidental loading **on the assumption that Russell House does <u>not</u> contain a piped gas supply to any part of the building.**

The results of BRE's assessment of the likely behaviour of the building under accidental loading are discussed in section 5.2 of this report.

3 Review of Pertinent Technical Documentation

3.1 Documents Provided to BRE

Only a very limited number of documents pertaining to Russell House have been provided to BRE. These comprised a report prepared by City of Birmingham – Industrial Research Laboratories, entitled *'Bison Wallframe Panels – Wall Tie Location, Russell House, Wednesbury',* Ref BC97/117/W31/9304B, dated 19th August 1985, and a limited set of construction drawings.

The drawings included the following :

- Russell Street Scheme 2, Typical upper floor plan, Ref 0041506, Bison Gregory.
- Russell Street Scheme 2, 1/8" elevations, Ref 0038369, Bison Gregory.
- Wall tie survey (showing remedial bolt locations), Panel layout, Russell House, 4no. elevation, Ref: 10871 (4no drawings).

A brief discussion of subject matters that have a direct bearing on the current assessment is contained in section 5.5 of this report.

4 Scope of Visual Inspection, Invasive Investigations and Materials Testing Programme

4.1 Visual Inspection of Building

A visual inspection of the components bordering the central corridors located at each floor level and the circulation areas such as stairwells, was undertaken as part of the evaluation of the current condition of the building. The external surfaces of the cladding panels to all elevations were also examined from ground level with the unaided eye and binoculars (upper levels). The roof top structures and upper surface of the flat roof were also inspected briefly by Mr. Barry Reeves of BRE as part of a general walk around inspection. However, it is proposed to undertake a more detailed inspection of the external elevations as part of the overcladding/over-roofing feasibility study.

The results of the general visual inspections undertaken by BRE have been summarised in section 5.4.3 of this report.

4.2 Invasive Examination of Structural Joints

BRE undertook invasive investigations of examples of selected structural joints located in two void flank wall flats situated at 4th and 7th floor levels. The invasive investigations were limited to these two levels as no other void flats located at 2nd floor[±] level or above, were made available to BRE.

Selected examples of several joint types were broken out using hand held concrete breakers. The standard of construction of the structural joints together with the reinforcement and tying provision was checked against any construction drawing that was available for the joint being examined. Where no drawing was available then a judgement on quality / tying provision was made on the basis of previous investigations of other Reema blocks assessed by BRE.

The quality of the in-situ concrete joints and dry packing and condition of the reinforcement was noted in each of the joints broken out. The nature and condition of the screed was also noted where broken-out.

A summary of the results of these investigations has been included as Appendix A. Annotated photographs of each area of the joints exposed have been included in this report as a record.

4.3 Assessment of Concrete Durability

In taking a holistic approach to the future management of LPS blocks BRE consider that it is generally necessary to make an evaluation of the current and future durability of both the internal and external reinforced concrete panels (i.e. internal wall panels and floor slabs and external cladding panels). In

[±] The detailing of the structural connections at 1st floor level are not typical of those found elsewhere in the block since the wall panels are founded on an in-situ podium. Accordingly, there is no merit in investigating the details of the connections at this level. Consequently, only materials testing and coring of the wall panels for strength determinations can be undertaken at 1st floor level.

undertaking such an evaluation it is necessary to consider the rates of carbonation, concrete cover and the presence (or otherwise) of chlorides, whether these be cast-in or ingressing. The results from such a study provide an indication of the likely future maintenance burden for the block with respect to concrete deterioration.

Accordingly, BRE undertook on-site testing for depth of carbonation and depth of cover to embedded reinforcement as well as sampling for chloride content on selected reinforced concrete components such as the loading bearing internal wall panels and the soffits of floor slabs. It was decided not to carry out materials testing of the external cladding panels since we understood that Sandwell MBC intended to overclad the building in the near future. Once this work is completed the external cladding panels will, in effect, be surrounded by an internal environment.

The concrete dust samples were obtained from selected internal components by a single depth drilling and later tested for the presence of calcium chloride.

BRE obtained concrete dust samples from a total of 14no. internal wall panels and the soffits to 13no. floor slabs.

The depth of carbonation of the concrete components tested was determined by incremental drilling.

The concrete cover to the embedded reinforcement was determined by drilling down from the concrete surface to expose a short section of the main reinforcement and measured directly using a depth gauge, or determined indirectly using a cover meter.

Since it was often impractical to locate the exact position of the wall panel reinforcement due to the relatively large depth of concrete cover, it was decided to use a cover meter set to a 12mm bar diameter to gauge the approximate depth of cover in these units.

The results of the on-site measurements made upon the internal concrete elements and details of the subsequent material testing undertaken, are discussed in section 5.3.

4.4 Concrete Compressive Strength Determinations

Over the past several years BRE have become aware of sometimes appreciable deficiencies in the compressive strength of cores obtained from some wall and/or floor panels in a number of the pre- and post-Ronan Point LPS blocks previously assessed by BRE. Such deficiencies are expected to have a direct bearing upon the behaviour of the wall and/or floor panels under accidental loading. That is, the concrete panels are expected to be less able to accommodate the tensile forces that develop during a non-piped gas explosion.

Accordingly, BRE obtained 10 no. concrete cores from selected wall panels bordering the two void flats in which investigations were conducted and in a 3rd void flat located at 1st floor level, as well as in adjacent circulation areas. A further 12no. cores were obtained from the edge(s) of the voided floor slabs and solid stair landing slabs at or above the 4th floor level.

A further 6no. squat cores (i.e. length << diameter) were removed from the outer leaf of the external cladding panels to provide a qualitative indication of the compressive strength of the concrete use to fabricate the outer leaf. It was envisaged that these strength data might be used during the development of a future overcladding system.

The cores were obtained by wet coring and subsequently prepared and tested in a laboratory for compressive strength. The respective characteristic compressive strengths were then calculated for the population of wall and floor panels sampled.

The results of these tests together with consideration of the associated implications are presented in section 5.1 of this report.

5 Discussion of Results

5.1 Concrete Compressive Strength Determinations

The characteristic[ϖ] compressive strength calculated for the cores obtained from the internal wall panels, outer leaf of external wall panels and floor slabs sampled within the building are 26.5N/mm^2, 52.9N/mm^2 and 27.3N/mm^2, respectively.

These values together with the mean values calculated (where available) for the internal wall and floor panels within the population of post-Ronan Point LPS buildings previously assessed by BRE, are shown for comparison in the table included below. Cores have not, however, been obtained from the external leaf to the external cladding panels in other LPS buildings assessed by BRE. Accordingly, it has not been possible to include a characteristic compressive strength for the outer leaf to the wall panels to these buildings in the table below as a comparison.

Component Type	Calculated Average Characteristic Strength For Other Post Ronan Point LPS Blocks Assessed by BRE (N/mm^2)	Calculated Characteristic Compressive Strength (N/mm^2) For Components in Russell House
Internal wall panels	**25.0**	**26.5**
Outer leaf of external wall panels (flank and elevation walls)	Not sampled on previous blocks assessed by BRE.	**52.9¥**
Floor panels	**24.6**	**27.3**

¥ Note that only 6no. squat cores (c71mm diam, c52mm to 74mm length) were obtained from the outer leaf of selected external wall panels. The compressive strength values varied from a minimum of 55.5 to 68.6N/mm^2. Since the cores had a length/diameter ratio of less than 1.0 the above test results and calculated characteristic compressive strength should be taken as being indicative values only. Compressive test undertaken upon squat cores tend to result in compressive strengths that are greater than those obtained from cores with a length/diameter ratio of => 1.0 [7].

Please refer to section 5.5 of this report for clarifying comments in relation to the predicted characteristic compressive strength of the concrete used to fabricate the external leaf to the cladding panels.

ϖ Defined as the mean of individual compressive strength values minus 1.64 times the standard deviation.

The characteristic compressive strength values calculated for the wall and floor panels sampled in Russell House are greater than the mean values calculated for the corresponding elements within the population of post Ronan Point LPS buildings previously assessed by BRE.

Accordingly, the strength of the wall and floor panels within Russell House in flexural and under compression is expected to be slightly greater than that found in other designs of pre-Ronan Point LPS buildings that have been assessed by BRE situated elsewhere within the UK.

5.2 Structural Assessment

The process of assessment adopted by BRE has involved :

1) A review of information available for Russell House [derived from structural investigation of selected areas in two void flats within the block]

2) Comparison of this information with that available for the Reema and Bison LPS blocks subjected to load testing by the BRE

3) Consideration of the structural evaluation calculations undertaken by BRE for the blocks which were load tested and also for Russell House.

5.2.1 Structural Connections/Quality of Construction

The details of a total of 23no. structural joints located in flat 30 (4th floor) and flat 50 (7th floor) were determined following the localised removal of internal finishes and the encapsulating in-situ concrete/ structural screed.

Details of the invasive investigations undertaken as part of the structural assessment are summarised in Appendix A. The annotated photographs included as **Plates 5** to **37** illustrate the position and nature of the joints examined.

The structural connections examined by BRE may be classified into four generic types. These include :

1. Cross wall / floor

2. Transverse floor / floor

3. Longitudinal floor / floor

4. Spine wall / flank wall.

The joints between the flank wall panels, adjacent floor slabs and elevation wall panels bordering the re-entrant areas have not been examined by BRE since the junction between the base of these wall panels and the adjacent floor slabs have been strengthened. Accordingly, the strength/connectivity of the associated wall/wall joints is not expected to be utilised in the event of a non-piped gas explosion due to the presence of the strengthening steelwork.

Consideration of the quality of construction of the above joint types listed together with the associated structural assessment is given in the following table; the structural assessment being based upon the results of the series of load tests undertaken previously by BRE on a Reema block and two Bison Wallframe blocks.

Joint Type	Summary of Condition and Structural Assessment
Cross wall / floor slab	The joints were in the majority of instances correctly formed and therefore in accordance with the design requirements. However, in three cases (Joints 1, 8 & 9) one or both of the projecting floor loops had not been correctly placed over the dowel bars projecting upwards from the head of the wall panel below. The in-situ concrete was generally well compacted. The reinforcement was found to be free from significant corrosion. The dry pack was generally well distributed along the length/across the width of the base of the panels and well compacted. Nevertheless, in view of the general good quality of workmanship with regard to the formation of the tying within these joints they are expected to be able to accommodate the forces associated with the 17kN/m^2 overpressure. The presence of vertical levelling dowels between 'upper' and 'lower' wall panels is expected to provided additional restraint to failure though base sliding. **12no. connections correctly formed and 3no. defective.**
Floor slab / floor slab (longitudinal joint)	The detailing of these joints was in accordance with the design requirements. The in-situ concrete was well compacted in the areas inspected. Therefore the embedded steel tie is expected to provide longitudinal tying in the plane of the floor slab between the end lounge and adjacent bay. **4no. correctly formed.**
Kitchen floor slab / lounge floor slab	The welded connection between the side of the one of the lounge floor slabs and the end of the kitchen floor slab within the flank wall flats, is expected to be mobilised in the event of the loss of support provided by a kitchen/lounge spine wall (Wall Type 1-4) in the flat below. The presence of a correctly formed welded connection between the two adjacent floor slabs is considered to be necessary to enable the building to develop alternative load paths should one of the kitchen/lounge spine walls be lost during a non-piped gas explosion. **1no. correctly formed and 1no. defective. Further invasive investigations should be undertaken within kitchens in the remaining flank wall flats to establish whether defective joints are likely to be common within the block.**
Spine wall / flank wall	Only one of these types of joints was excavated. The joint detail was in accordance with the design requirements. The in-situ concrete was generally well compacted. The reinforcement was found to be free from significant corrosion. As such this joint is expected to be able to enable shear transfer between the spine and flank wall panels to occur should vertical support be lost at a corresponding position within one of the storeys above/below. **1no. correctly formed.**

Cross wall / elevation wall	Only one of these types of joints was excavated. The joint detail was in accordance with the design requirements. The in-situ concrete was generally well compacted. The reinforcement was found to be free from significant corrosion. Since the elevation wall panels are expected to be lost in the event of a severe non-piped gas explosion any shortcomings in workmanship that exist within these particular joints are not expected to be of structural significance. **1no. correctly formed.**

Details of the original reinforcement provision for the wall panels and floor slabs within Russell House were not available. Accordingly, BRE undertook a covermeter survey of the soffits to selected floor slabs supplemented by localised breakouts to expose short lengths of reinforcement. The nature of the tying and reinforcement provision identified with Russell House was found to be broadly consistent with that found in other pre-Ronan Point LPS buildings previously assessed by BRE.

5.2.2 Accidental Loading

It should be noted that the simplified analysis of the wall and floor elements undertaken as part of the structural assessment of this block have incorporated various assumptions and limitations such as one-way spanning behaviour and linear elastic behaviour. The analysis procedure used as part of this assessment was developed as part of BRE's programme of work involving full-scale testing of large panel system built dwelling blocks. The outcome of each analysis was compared to the previous experimental findings and an engineering judgement made about the ability of the element being assessed to resist the forces arising from an accidental loading overpressure of $17kN/m^2$.

This assessment was made on the basis that Russell House did <u>not</u> contain a piped gas supply to any part of the building. BRE understand that this remains the case.

Consideration was given to the potential performance of the flank wall (assuming no strengthening steelwork was present), the cross and spine walls and the floor slabs under accidental loading.

A summary of the findings of the accidental loading ($17kN/m^2$ over-pressure) analyses (refer Appendix C) is given below for the convenience of the reader.

Area/Element	**Summary and Comment**
Floor slabs (Types 1-1, 1-2 & 1-3] (Refer Appendix C)	The simplified evaluation of the floor slabs indicates that the three generic types should all be satisfactory for the downward loading scenario. Floor Types 1-2 and 1-3 are also expected to be able to accommodate the applied accidental loads under upward loading. **The 'simplified' calculations, however, indicate that the overload factor for floor Type 1-1 under upward loading is approximately 2.3. On this basis this floor type fails the assessment criterion for upward loading.** However, a non-linear finite element analysis undertaken upon floors with similar overload factors manufactured using concrete with a lower characteristic compressive strength have been shown to be structurally adequate under upward loading albeit suffering from appreciable damage. In addition, the Bison floors load tested by BRE, which were slightly shorter at 3.06m and 3.21m, respectively, were able to accommodate an overpressure in excess of 17kN/m^2. **The non-linear finite element analyses undertaken by BRE upon the long span floors (assuming a minimum tensile concrete strength of 2N/mm^2) indicates that these elements should be able to survive an overpressure of 17kN/m^2 under upward loading, albeit suffering from gross cracking.**
Cross walls	Generally satisfactory under accidental loading on all but the top or upper two storeys[Π]. However, those cross walls that are predicted to be at an increased risk of failure do not provide primary support to adjacent floor slabs above. Accordingly, their loss is not expected to result in disproportionate damage.
Re-entrant elevations walls	Generally satisfactory under accidental loading on all but the top storey[Π]. Existing strengthening steelwork is expected to prevent base shear failure.
Flank wall	Likely to be satisfactory under accidental loading regardless of the presence of the steel strengthening angles at the base of the panels. The strengthening steelwork and anchor bolts were found to be in good condition and are therefore expected to perform satisfactorily.
Spine wall between kitchen and lounge in flank wall flats	**Expected to fail in flexure at all floor levels.** **In the event of the failure of a kitchen/lounge spine wall then the transverse welded connections between the lounge and kitchen floor slabs are expected to come into play enabling the structure to mobilise alternative load**

[Π] Components on this level are normally regarded as being sacrificial in the event of an explosion on the top storey.

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